Near-Field Soil-Structure Interaction Analysis Using Nonlinear Hybrid Modeling

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INTRODUCTION

The hybrid modelling method (Gupta and Penzien, 1980) and associated analysis procedure for solving a three-dimensional soil-structure interaction problem was developed by Gupta and Penzien (1981) and Gupta et al.(1982). Subsequently, successive modifications have been made to the original modelling method and analysis procedure allowing more general treatment of the SSI problem (Penzien, 1988). Through many correlation studies of field test data obtained under forced-vibration and earthquake-excitation conditions, it has been shown that the HASSI programs can effectively predict the dynamic response of a soil-structure system, if realistic soil parameters are adopted (Katayama et al., 1987(a) and 1987(b); Penzien et al., 1987; Chen et al., 1988). In the above, the entire structure-foundation system is considered to respond in a linear fashion. Since the reflected three-dimensional waves at the soil-structure interface decays very rapidly with distance away from the structure (Katayama, 1987(a)), the response of the soil close to the base of the structure may greatly affect its response; therefore, proper modelling of the nonlinear soil behavior characteristic is essential. The nonlinear behavior of near-field soil has been taken into consideration in HASSI-7 by the standard equivalent linearization procedures used in programs SHAKE and FLUSH.

These procedures as used in the hybrid modelling method, including efficient substructuring and condensation procedures, and the results of correlation studies of real earthquake response of two different concrete model structures at different sites with analytical results predicted by HASSI-7 are presented herein.

CONCEPT OF HYBRID MODELLING AND HASSI-4

Our hybrid modelling approach basically partitions the system (Fig. 1(a)) into the near- and far-field soil regions, using a hemispherical interface which cuts through the soil under structure. One substructure (the near-field) consists of the structure to be analyzed under prescribed loading conditions and a finite portion of soil encompassing its base as shown in Fig. 1(b). The second substructure (the far-field) is a

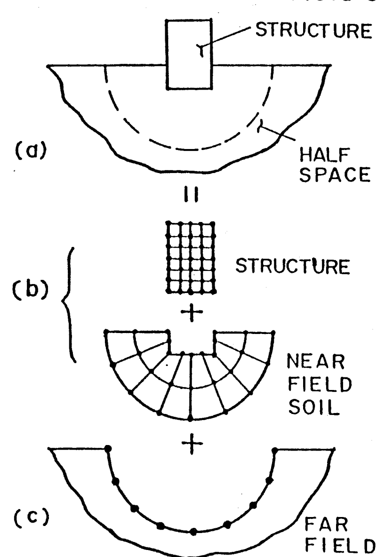


Fig. 1 Modelling concept of HASSI-4

semi-infinite half-space which shares a common interface with the near-field as in Fig. 1(c). This interface is hemispherical in form so that it provides a smooth surface along which mathematical boundary conditions to permit radiation of energy away from the foundation of the structure into the half-space can easily be satisfied. This modelling uses a far-field impedance matrix, relating far-field forces at the interface nodes to the corresponding nodal displacements, which was obtained by the by the system identification technique(Gupta and penzien, 1981). Modelling of the near-field makes use of the following types of linear finite elements: (1) three-dimensional beam element, (2) three-dimensional spring element, and (3) three-to-nine-node axisymmetric solid element. The stiffness and damping properties of each element are modeled using the complex modulus method; thus element damping is represented in hysteretic form. The HASSI series have reached a practical application level at the stage of HASSI-4 (Katayama, 1988) by permitting the following calculations based on linear-homogeneous soil properties; (1) response analysis of the soil-structure system subjected to the prescribed triaxial free-field acceleration input motions at the structure-soil interface, and (2) time-domain or frequency-domain response analysis of the complete soil-structure system subjected to simultaneous forced-vibration forces prescribed at multiple locations.

NONLINEAR HYBRID MODELLING AND HASSI-7

Controlling strain of soil

In applying the standard equivalent linearization technique, the effective strain of a soil element, the shear strain to control the strain dependent stiffness and damping properties of the near-field soil as experienced during three-dimensional seismic response, is represented by its octahedral shear strain at the middle point of the element.

First, the displacement due to soil-structure interaction, u^i, is calculated and the total displacement of a soil element, u^t, is obtained by adding the displacement, u^f, induced by the free-field ground excitation; i.e.

$$u^t = u^f + u^i$$

By using the constitutive law for the given displacement and shear strain, the octahedral shear strain can be expressed in terms of the non-zero strain components e_{zz}, e_{yz}, and e_{xz} as:

$$r_{oct} = \sqrt{\frac{8}{3}\left[\frac{1}{3}e_{zz}^2 + e_{yz}^2 + e_{xz}^2\right]},$$

where x, y, and z denotes two orthogonal horizontal axis, and vertical axis, respectively.

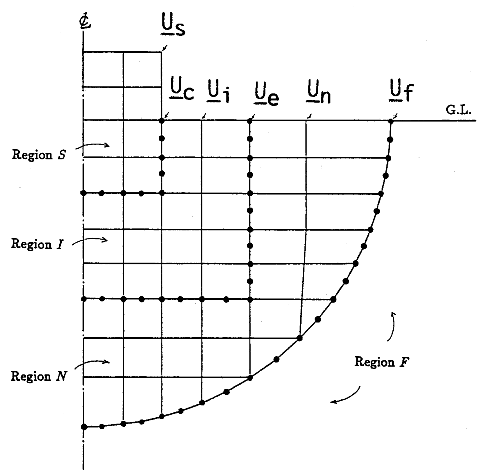


Fig. 2 Substructuring scheme of Near-Field soil region

Substructuring of Near-field Soil

Since the hemispherical interface is chosen to pass through the soil region at a distance of approximately three to four times the radius of the structure, the ground motions outside this interface will be nearly the same as the free-field motions produced by the earthquake; however, the motions within the region very close to the structure will be altered considerably by the effects of soil-structure interaction. Therefore, the near-field soil region is subdivided into two subregions I and N as shown in Fig. 2. Region I, having $\underline{U_e}$ degree of freedom at its interface with region N, is that region which experiences nonlinear soil behavior due to the large strains produced by soil-structure interaction. The interface I/N can be of arbitrarily axisymmetric shape but it must be located between the S/I and the hemispherical N/F interface. By the way, the region S is an arbitrary linear-structure of three dimension except its axisymmetric foundation.

CORRELATION STUDIES

Response produced by the May 20, 1986 earthquake, Lotung, Taiwan

The May 20, 1986 earthquake of magnitude 6.5 occurred near Hualien (epicentral distance; 66 km, focal depth; 16 km). The peak ground accelerations recorded at the test site as well as at the SMART-1 array were about 0.22 g. The horizontal acceleration components of ground motion at station FA15 and the response motion at F4UE of the model are shown in Fig. 3(a) with the location of the measuring points in Fig. 3(b).

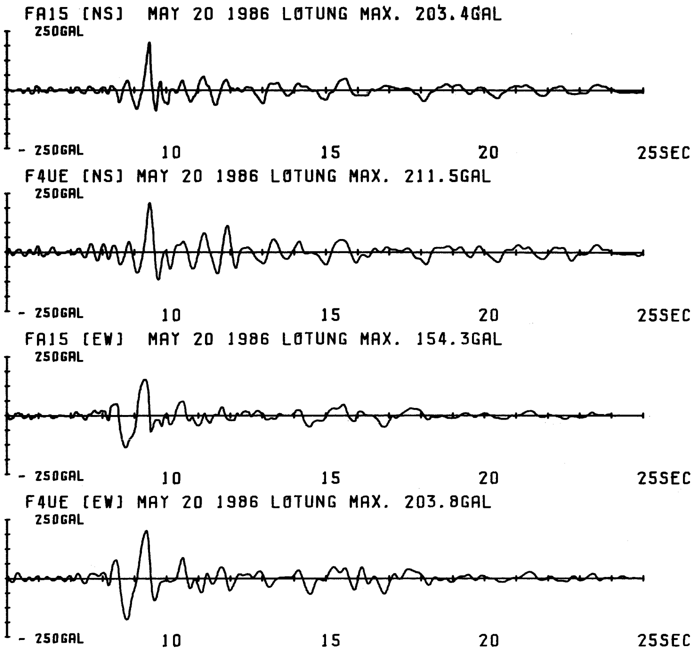


Fig. 3(a) Accelerograms at station FA15 and at F4UE during the May 20, 1986 earthquake

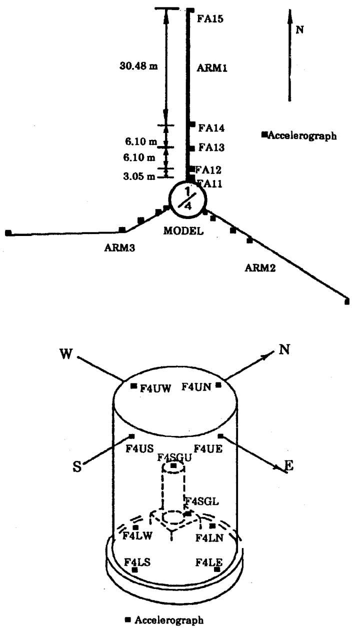


Fig. 3(b) Location of accelerographs

Following the validation studies requested by EPRI/TPC for the workshop held at Palo Alto in December 1987, the recorded ground surface accelerations at station FA15 were specified as the control motions.

For the site response analysis, the low strain shear modulus and a damping ratio of 2 % were used as the initial soil properties for each layer. The final effective strain of each layer was estimated by 60 % of the maximum octahedral shear strain of the layer. The convergence limit of shear moduli in iteration was set 5 %.

The deconvolved free-field ground motions at the depth of 4.7 m obtained by the site response analysis were used as the input ground motions at the foundation-soil interface. The associated shear wave velocity and damping ratios shown in the far-field layer of Fig. 5 were used as the initial soil properties to the hybrid model.

The acceleration time-histories at the containment roof (F4UE) by the HASSI-7 and the associated response spectra of 5 % damping are shown in comparative form with corresponding recorded motions and spectra in Fig. 4.

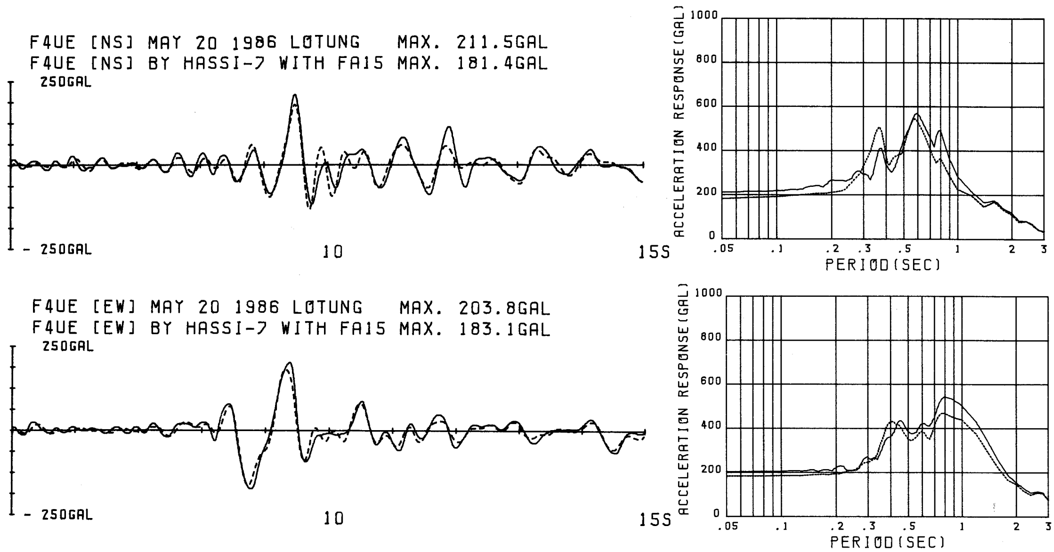


Fig. 4 Predicted response of the roof of the model and corresponding response spectra of 5 % damping

The converged soil properties in nonlinear soil region by the analysis are shown in Fig. 5. From these figures, it can be said that the acceleration time-histories at the containment roof (F4UE) are well reproduced by the nonlinear hybrid analysis and also that, since the converged soil properties in Region I are comparable at the presumed I/N interface to those of the site response analysis, a fairly small region surrounding the base of the structure is significantly affected by soil-structure interaction.

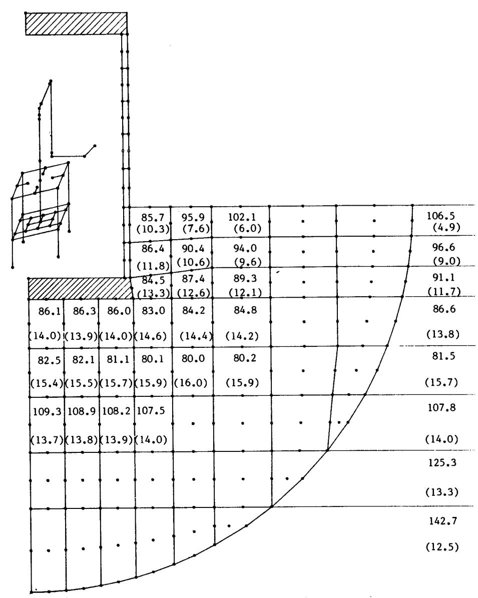


Fig. 5 Converged soil properties in non-linear soil region (Region I of Fig. 2)

Response produced by the December 17, 1987 earthquake, Kazusaminato, Japan

The natural unit weight and in-situ shear wave velocity of the surface layer consisting of a stiff and clean medium sand at Kazusaminato site were 1.8 t/m^2 and 300 m/sec within the near-field soil region modelled in the analysis (Katayama et al, 1987(a) and 1988). The strain dependent soil properties are given by the laboratory data in Fig. 6.

The horizontal components of measured response motions of the roof top of the model structure and the free-field ground motions at station S1, and the locations of them are shown in Fig. 7, for the 1987 Off Bousou Pennisula Earthquake. The maximum acceleration at the roof top and the ground surface were 301.2 and 73.5 gals. The surface ground motions at S1 were designated as the control motions and used as the input ground motions at the structure-soil interface.

The modelling conditions and procedures similar to those of the analysis performed for the previous model at Lotung were used. The initial shear wave velocity at very small strain was selected as 250 m/s based on the resonant column test of the soil.

The resulting time histories of motion and corresponding Response spectra of 5 % damping of the roof top of the model are shown in comparative form in Fig. 8.

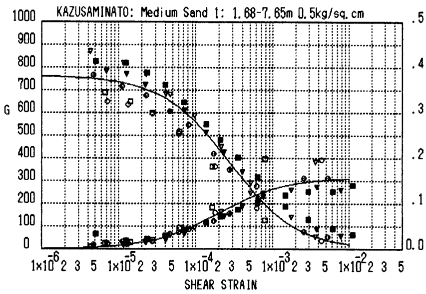
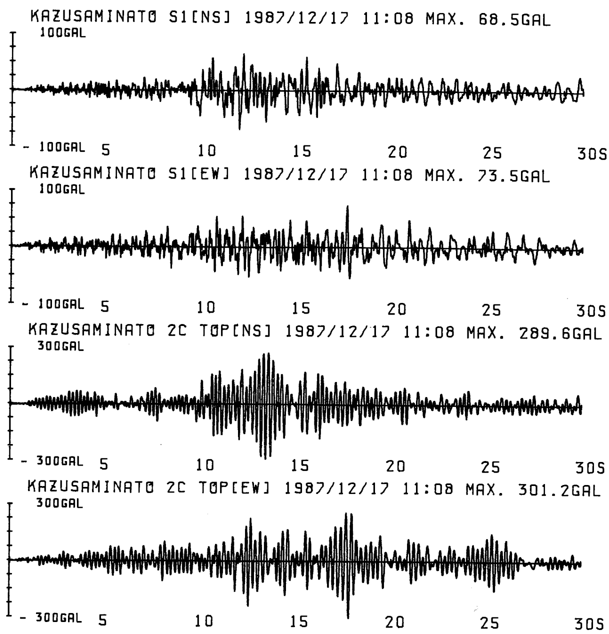


Fig. 6 Strain dependent dynamic shear modulus and damping ratio

The coincidence of the calculated and observed response of the model structure were fair as a whole, however, the predicted maximum accelerations became smaller than the observed; this may be attributed to the high-cut nature usually found in the complex modulus method.

The converged average shear wave velocities and damping ratios of the presumed nonlinear soil region I and those of adjacent soil are shown in Fig. 9. From the figure, the presumed non-linear soil region might be considered appropriately allotted in the analysis model, leading to the conclusion that the soil affected by non-linear soil-structure interaction can be limited to a fairly small region.



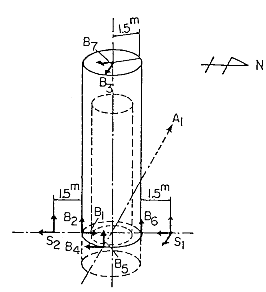


Fig. 7 Observed response of the model and ground motions at S1 and the location of seismographs.

CONCLUSIONS

The nonlinear dynamic soil-structure interaction analysis code by hybrid method - HASSI-7 - was found very capable to predict the seismic response of the structure with small embedment including soil-structure interaction effect through the correlation studies on the earthquake response records obtained at both models in Lotung and Kazusaminato with the predicted response of them.

It was found from the calculated results that the near-field soil region significantly affected by soil-structure interaction was restricted within a fairly small domain surrounding the base of the structure. It was clearly shown, in this context, the decay of the reflected waves at the soil-structure interface into far-field was very fast and so far the refined substructuring technique adopted in the code was found very reasonable and efficient.

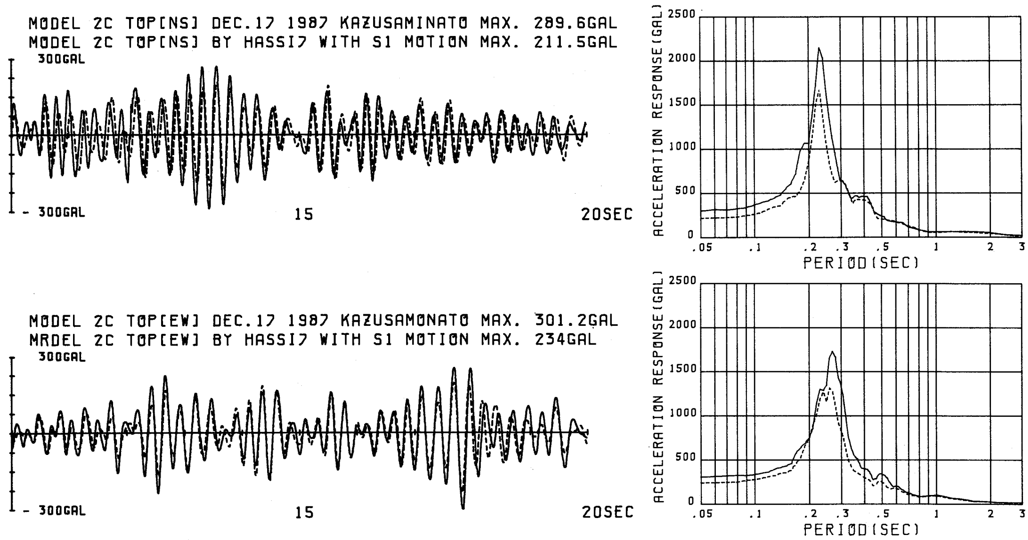


Fig. 8 Predicted response of the roof of the model and corresponding response spectra of 5 % damping

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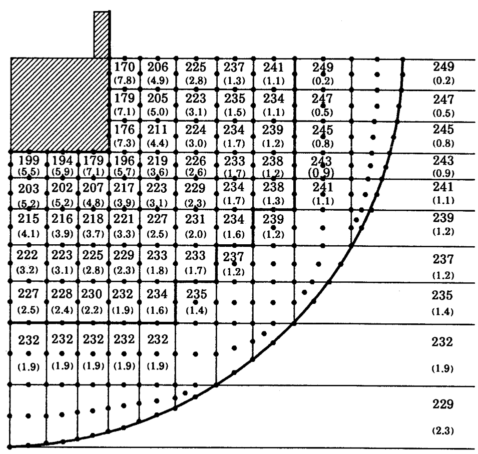

Fig. 9 Converged soil properties in nonlinear zone and adjacent soil.

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GINTER - A Computer Program for Efficient Soil-Structure Interaction Analysis Including Support Non-Linearities

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SUMMARY

The purpose of this paper is to present the details of a new computer program for soil-structure interaction, where the non-linear properties of the structural supports can be taken into account. Practical applications have shown the need for this type of analysis. The most likely applications appear to be in the dynamical analysis of structures with flat foundations, like nuclear reactor buildings, where uplifting occurs due to strong earthquake excitation. In this case the support non-linearity is described by the gap element. Another useful application is the case of the hysteretic and strain hardening properties of the structural supports. For example, the structures supported on neoprene pads as a type of aseismic foundation require this type of analysis.

The method used is based on the direct time integration of the structural equations of motion, where the non-linearity is restricted to the structural supports only. The advantage is that the time consuming correction of stiffness properties to account for non-linearity is restricted to only a small region of the structural stiffness matrix.

As a practical application two cases are analysed - the uplift problem for the nuclear reactor building and the aseismic foundation concept using neoprene pads.

The conclusion of the paper is that whenever a heavy mass structure is to be analysed for earthquake excitation more accurate results for proper physical behaviour can be achieved by the presented approach.

1. INTRODUCTION

There are many practical cases in the structural design of nuclear reactor buildings which lead to a non-linear dynamic analysis. In most cases and due to the design requirements the non-linearities occur essentially at only a few locations in the structure. This means that almost all degrees of freedom in the structure behave linearly and could therefore be excluded from the intensive numerical effort which is necessary for the non-linear case. Such structural systems will be called locally non-linear.

The following two typical cases are representative of locally non-linear systems:

a) Nuclear reactor building resting on soil being partly uplifted due to a strong earthquake excitation.
b) Nuclear reactor building based on an aseismic foundation concept using neoprene pads.

In the first case the local non-linearity occurs in the form of gap elements, which are placed between the base mat and the soil surface. During the earthquake excitation due to uplift some of the gap elements become unstressed, i.e. their stiffness becomes zero. This, in turn, causes a correction of the structural stiffness, which leads to the non-linear numerical procedure.

In the second case the local non-linearity is introduced into the structure by the designer, who defines the characteristics of the neoprene pads in order to achieve the aseismic effects, i.e. the protection of the building and components against earthquake excitation. Construction details of the aseismic bearing pads are given in many papers, e.g. (Gueraud et al.,1985). Usually the pads are constructed to have prescribed stiffness up to a precisely defined friction force, from which point upwards controlled sliding should take place. This leads to bi-linear force/displacement characteristics. The area enveloped by the one-cycle-curve gives information about the energy dissipation in the system. The aim of this measure is to keep the excitation energy away from the components in the reactor building. This is usually offset by an increase of the structural displacements.

Using the new computer program the relevant vibrational behaviour of the structure can be analysed, i.e. base overturning moment, soil pressure, percentage of the base mat uplift, floor response spectra in the building, time histories of structural displacements and stresses, etc.

In the next section the method of locally non-linear analysis is presented. In the third section the uplifting problem will be considered and comparison of the locally non-linear analysis to the linear one will be made. This is followed by a consideration of aseismic design of the nuclear reactor building using neoprene pads in the fourth section.

2. THE METHOD OF LOCALLY NON-LINEAR ANALYSIS

The equilibrium equation may be written in the form

$$M \ddot{r}_{n+1} + C \dot{r}_{n+1} + K(x_n) \Delta r = R_{n+1} - F(x_n) \qquad (1)$$

where Δr is the displacement increment vector and the right hand side is the difference between the applied force at time step n+1 and the stress divergence based on the displaced state and stresses at step n. The solution method implemented is the unconditionally stable implicit time integration based on the Newmark scheme. The Newmark constants β and γ are chosen to satisfy the relations

$$\gamma \geq 0.5 \quad \text{and} \quad \beta \geq 0.25\ (0.5 + \gamma)^2 \qquad (2)$$

As is well known, for $\gamma = 0.5$ and $\beta = 0.25$ the method becomes the trapezoidal rule and is energy preserving.

Let K^* denotes the updated stiffness matrix which is given by the expression

$$K^* = K(x_n) + (\beta h^2)^{-1}M + \gamma(\beta h)^{-1}C \qquad (3)$$

where h is the time step interval length. It can be written as a block matrix in the form

$$K^* = \begin{pmatrix} k_{11} & k_{12} \\ k_{21} & k_{22} \end{pmatrix} \qquad (4)$$

where k_{11} represents the dominant linear block. The main numerical effort which is needed to decompose this submatrix is done only once. At each non-linear iteration step the rest of the matrix is updated and decomposed. The partitioning of the stiffness matrix in the linear and non-linear submatrices is done automatically by the program.

The method is implemented in the computer code GINTER using the standard FORTRAN 77 compiler (RM/FORTRAN version 2.40) on an IBM compatible Siemens personal computer under MS-DOS 3.20.

3. BASE MAT UPLIFT PROBLEM

The nuclear reactor building is modelled using the two-dimensional beam elements as shown in Fig. 1. The total mass of the building is 141,120 tonnes. The non-linear gap elements are modelled as springs with no tension capability. The linear stiffness parts of these elements are equal to the corresponding soil stiffnesses of the circular sections which are covered by these elements. Using the complex impedance function approach for a layered soil the total vertical stiffness corresponding to the circular base mat of a radius 27.3 m is determined to be equal to 566,630 MN/m. The static settlement is therefore equal to 2.44 mm.

The earthquake excitation is given by two randomly independent acceleration time histories - 0.28 g for the vertical component and 0.42 g for the horizontal one - acting simultaneously on the structure. Both time histories as well as the corresponding response spectra are shown in Fig. 2. Excitation duration time is 10 seconds.

In order to discuss the results the uplift effect is considered first. It can be analysed by looking at the response time histories of displacement of two edges of the base mat denoted by nodes 46 and 66, Fig.4a and 4b. As it can be seen the base mat uplift occurs several times around 5 to 7 seconds. Whenever the node 46 undergoes maximum uplift (positive displacement in the diagram) the opposite side, which is given by the node 66 appears to have its maximum pressure, and vice versa. The extreme rocking happens to be at approximately

5.90 seconds, cf. Fig. 4b. The geometry of uplift at this stage is shown in Fig. 4c. The maximum uplift of 8.44 mm at node 66 corresponds to the negative displacement of 9.55 mm on the other side at node 46. The zero-point of the displacement is therefore 1.68 m away from the centre of the base mat. This gives 47 percent base mat uplift and 679.30 kN/m^2 maximum soil pressure at the base mat edges.

Floor response spectra of the reactor pressure vessel support in the reactor building, node 21 in Fig. 1b, exhibit an obvious reduction, Fig. 5a and 5b. For an extensive study of the uplift problem using the approach based on the eigenvalue solution of the linear part of the structure see the reference (Kennedy et al.,1976).

4. ASEISMIC DESIGN USING NEOPRENE PADS

The structure is modelled according to Fig. 1c. It is assumed that the seismic isolation is active for the horizontal direction only, which is true for the way in which the neoprene pads operate. The horizontal force/deflection characteristic is given in Fig. 3, representing the total effect of the number of neoprene pads in the bearing. It appears to be of a bi-linear type where the friction force is given with 222.8 MN corresponding to a friction coefficient 0.20. The isolation is defined by giving the basic frequency of the isolation, which is chosen here to be 1.0 Hz. For the total structural mass of 141,120 tonnes this leads to the bearing stiffness of 5,571 MN/m.

In order to discuss the results, the reduction of the floor response spectra due to base isolation is considered. The floor spectrum of the reactor pressure vessel, node 21, representing the non-isolated case, Fig. 5b, should therefore be compared to the base isolated case in Fig. 5c. An obvious reduction in response down to the rigid body value is exhibited in the frequency range above 1.5 Hz.

Therefore it may be concluded that the greater civil engineering effort for the base isolation is offset by savings on the component side due to the reduction of the floor response spectra in the reactor building.

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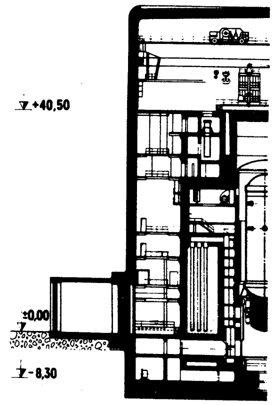
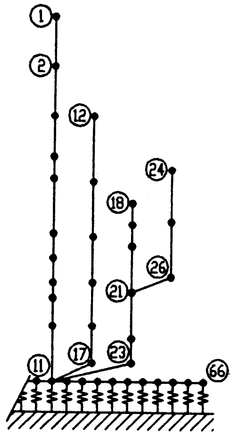
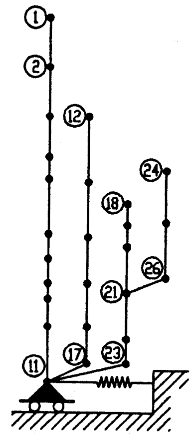


a) Reactor building



b) Finite element model for uplift analysis



c) Aseismic concept on neoprene pads

Fig. 1 Modelling the nuclear reactor building

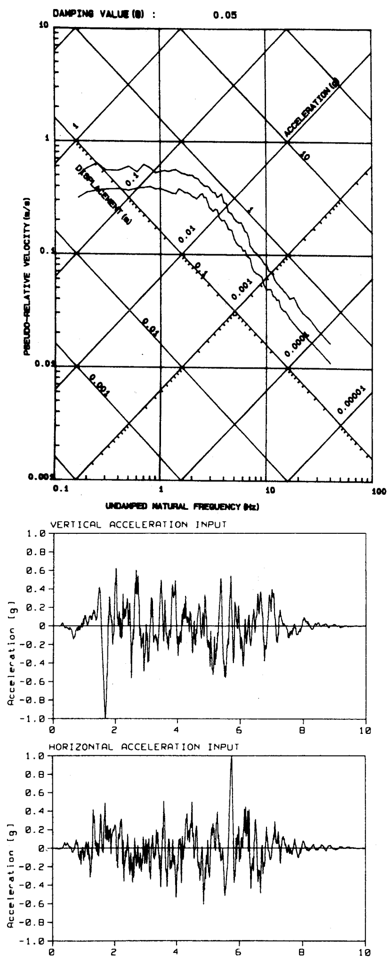
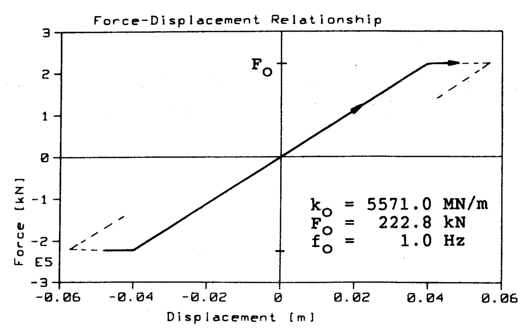
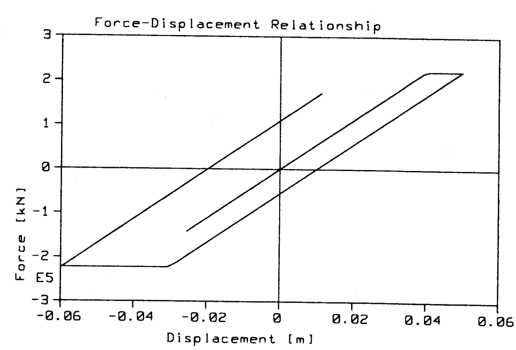


Fig. 2 Earthquake excitation

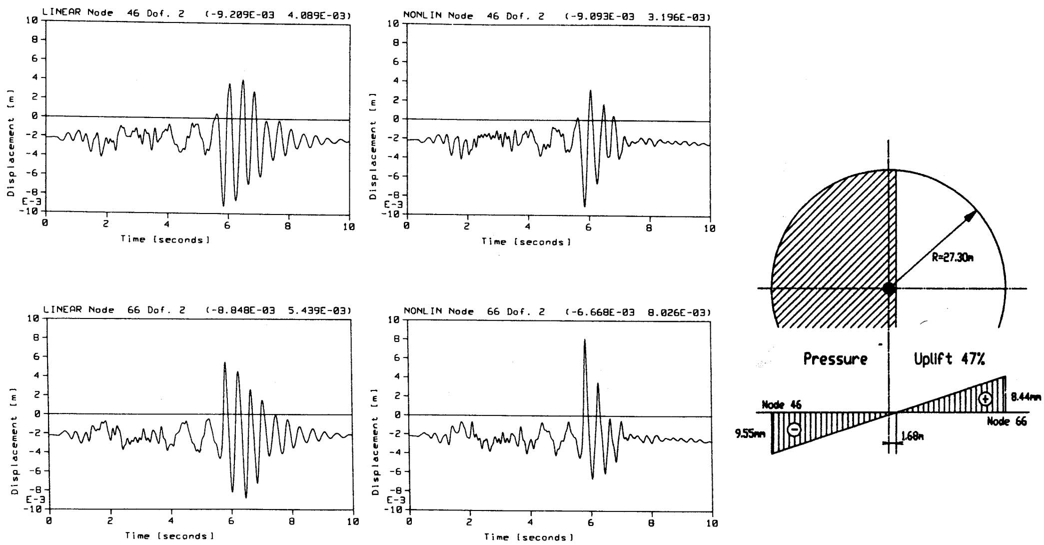


a) Defined characteristics



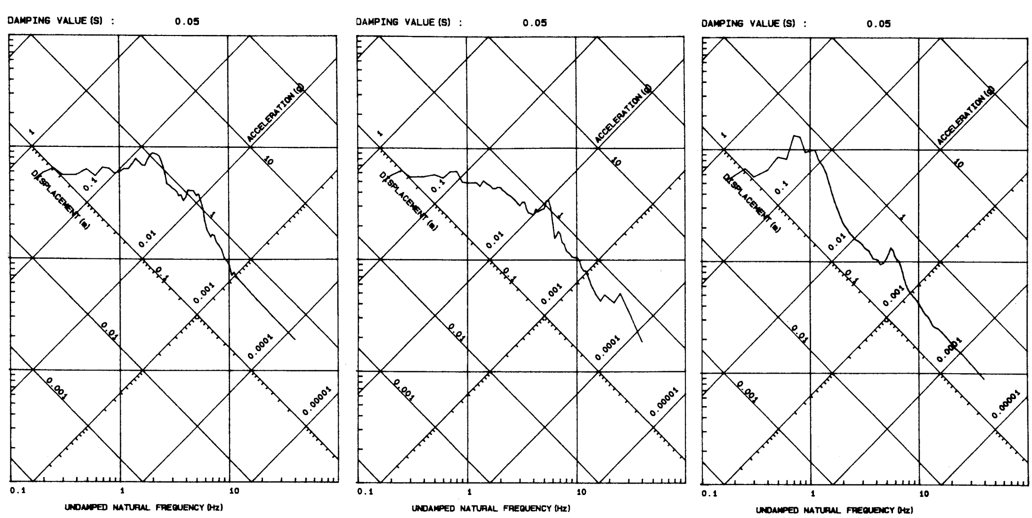
b) Curve written by the actual program run

Fig. 3 Horizontal characteristic of the aseismic bearing



a) Linear b) Non-linear c) Uplift geometry

Fig. 4 Displacement time histories of the structural response for nodes 46 and 66



a) Linear b) Non-linear (uplift) c) Non-linear (pads)

Fig. 5 Floor response spectra for the reactor vessel support