COMPARISON OF VIBRATION TEST RESULTS FOR ATUCHA II NPP AND LARGE SCALE CONCRETE BLOCK MODELS

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Abstract

In order to study the soil structure interaction of reactor building that could be constructed on a Quaternary soil, a comparison study of the soil structure interaction springs was performed between a full scale vibration test results of Atucha II NPP and vibration test results of large scale concrete block models constructed on Quaternary soil. This comparison study provide a case data of soil structure interaction springs on Quaternary soil with the different foundation size and stiffness.

1. VIBRATION TESTS OF ATUCHA II NPP

A full-scale vibration test results of Atucha II NPP was carried out in November of 1993 by the Commission National de Energia Atomica, Empresa Nuclear Argentina de Centrales Electricas S.A., Universidad National de Cordoba and Kajima Corporation. The main purpose of the tests was to provide experimental data on the dynamic characteristics of the main reactor building and adjacent structures of a full-scale nuclear power plant built on deep Quaternary soil deposits. Test results were intended to provide a benchmark case for control and calibration of state-of-the-art numerical techniques used for engineering design of new plants and assessment of existing facilities.

Atucha II NPP is located on the alluvial plains of Argentina on the Parana River, 100 km north of Buenos Aires. This is a low seismicity site, as results from scarce seismogenic features in the area and considerable distance to the seismically active western provinces of Argentina. Fig.1 shows general view of Atucha NPP site. The building has double spherical containment vessels, which are typical for this type of reactor, with steel inner wall (PCV) and reinforced concrete outer wall (R/B). The inner concrete structure (I/C) is encased by these vessels. The building is 60m high and the diameter of its base-mat is 60m. The supporting layer is mainly composed of Quaternary deposits of sandy clay soil, with a shear wave velocity of approximately 350m/sec and depth down to bed rock of approximately 500m. The depth of embedment is about 20m.

A total of 90 displacement components were recorded, twelve of them at foundation level of the neighboring turbine hall and at the soil surface at a distance of up to 200 m from the reactor building. Fig.2 shows measuring points on vibration test of Atucha II reactor building. Forced vibration tests were executed in November 1993 within a short period after construction was completed and before machinery installation had started. The test program included two types of dynamic excitation. The basic testing routine was a frequency sweep from 1 to 20 Hz by means of a

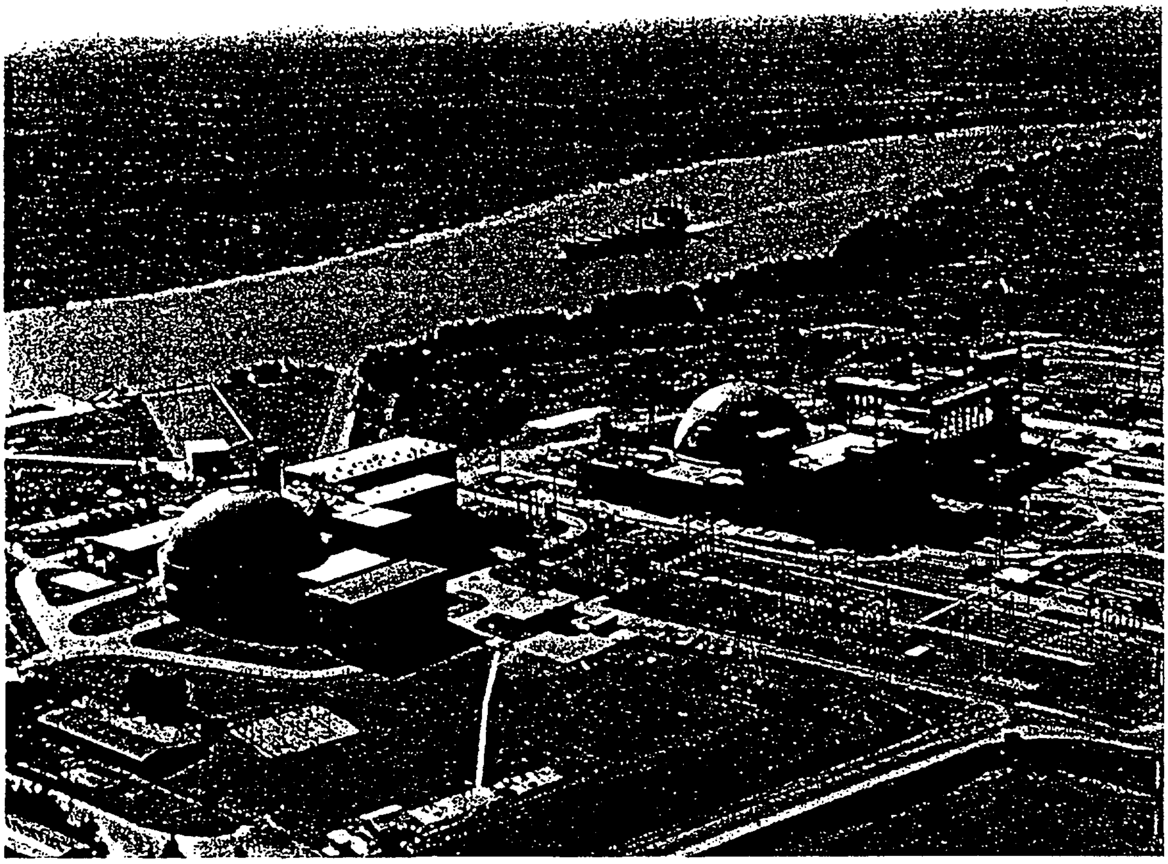
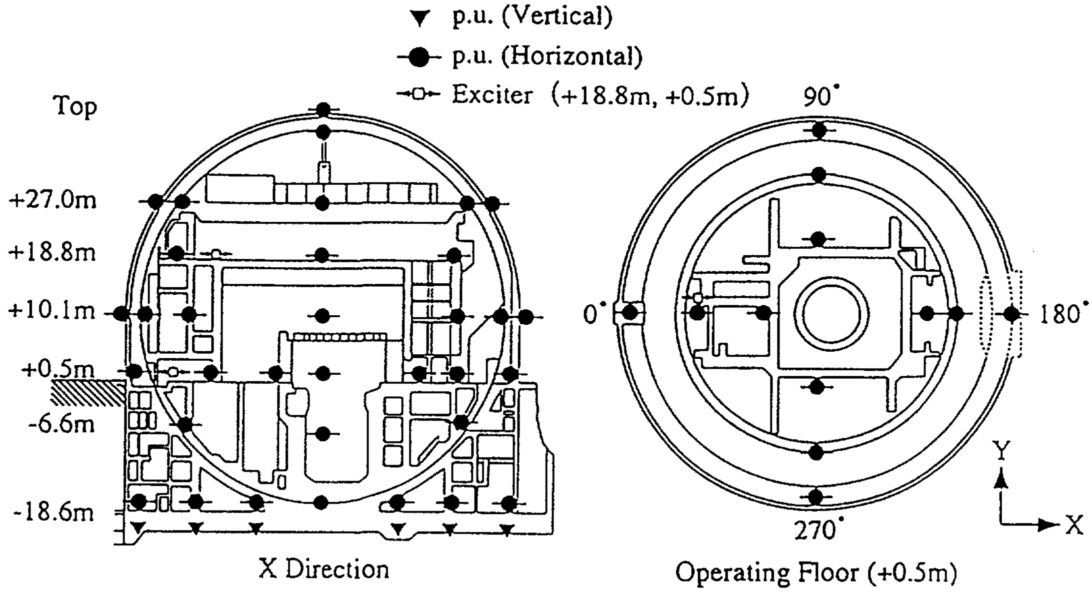


Fig.1 General view of Atucha NPP site



X Direction

Operating Floor (+0.5m)

Fig.2 Measuring points in vibration test of Atucha II reactor building mechanical exciter, with the exciter located successively in three different locations. These were provided to excite the building separately along the two main axes of the structure, and to add some degree of redundancy in the measurements. Taking the building's symmetrical shape into consideration, forces were applied along the X axis (0-180 degrees) and the Y axis (90-270 degrees), which cross at right angles on the same plane. In the X direction, an exciter was installed at two levels, GL+18.8m (at the top of the inner concrete structure) and GL+0.50m (on the operating floor), for the same measuring points. This was to observe the coupling characteristics between sway and rocking vibration. In the Y direction, the exciter was installed only at level GL+18.8m. Thus, three series of tests were executed.

Resonance and phase lag curves at the tops of the R/B, PCV and I/C for GL+18.8m excitation in the X direction are shown in Fig.3. Resonance amplitudes were normalized for an exciting force of 9.8kN. There is a small dominating peak in the range of 2.9Hz~4.5Hz, which is considered to indicate a fundamental resonance peak of the soil~structure interaction, as the phase lag curve crossing the 90 degree line. Such a wide-range low-level peak is considered to be caused by the soft soil compared to the rock and the deep embedment, which increased the radiation damping. This phenomenon is a feature of the Quaternary deposit siting. Although small peaks are observed, significant resonance peaks are observed only at 5.9Hz and 7.3Hz that are the resonance frequencies of the PCV and the R/B, respectively.

Fig.4 shows resonance and phase lag curves of Atucha II reactor building at the top of the R/B by excitation at the X+18.8m and at the Y+18.8m. Within the low frequency range, the building

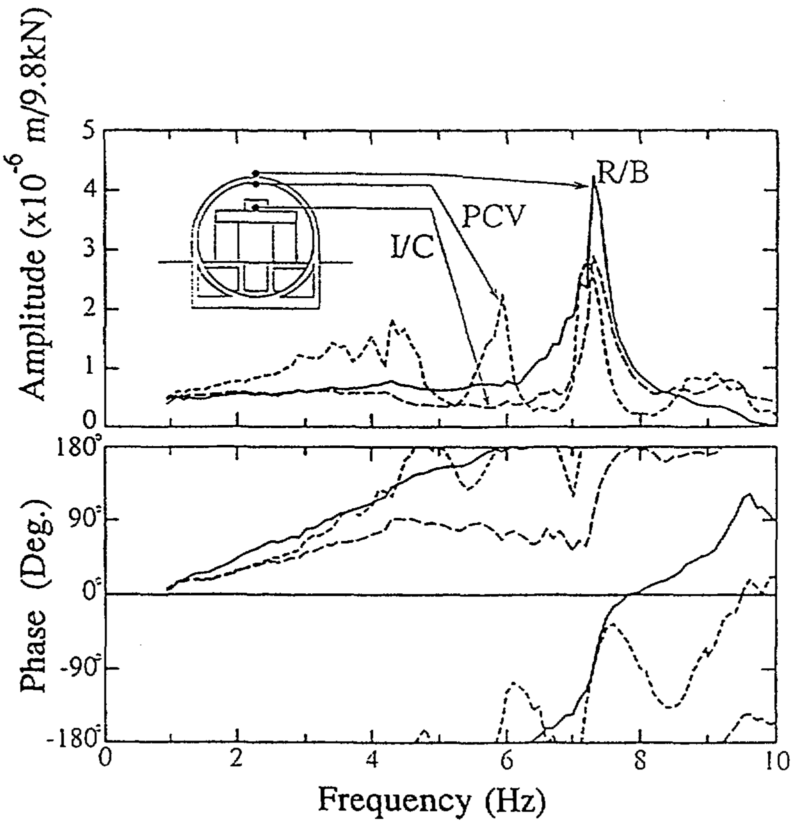


Fig.3 Resonance and phase lag curves of Atucha II reactor building by forcing at X+18.8m

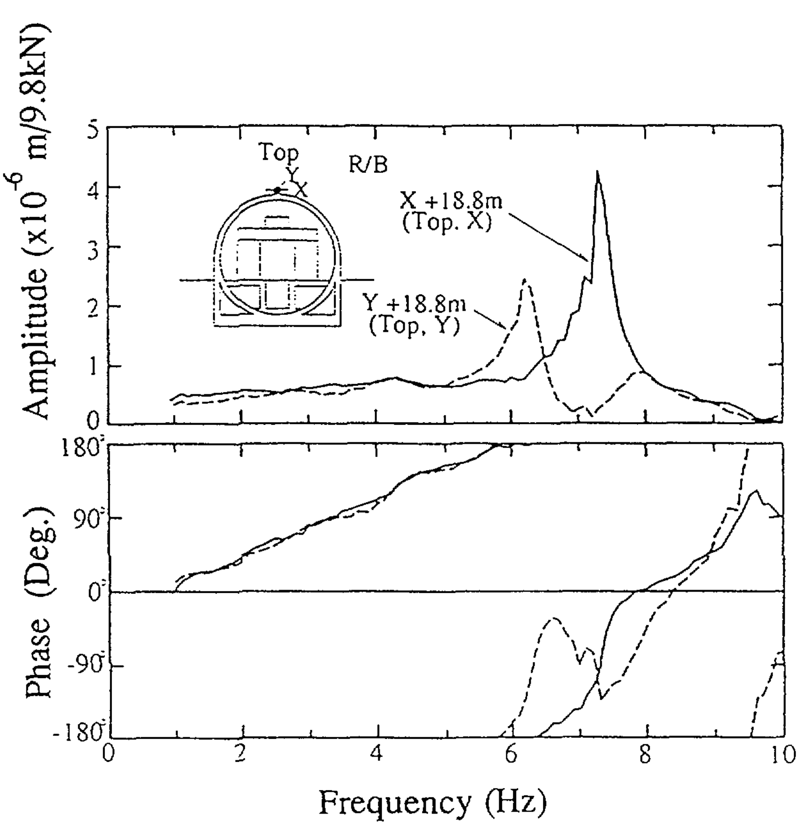
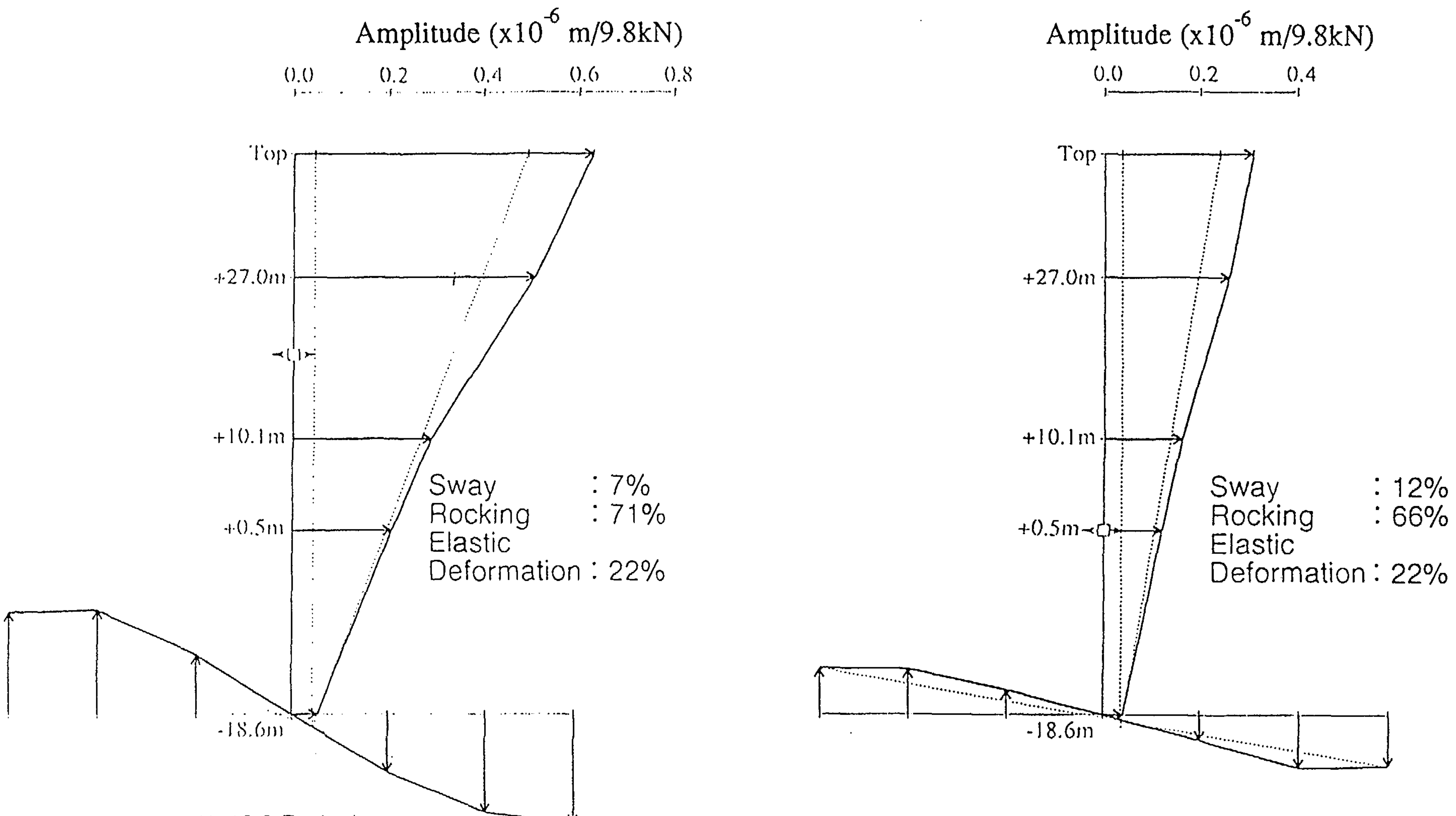


Fig.4 Resonance and phase lag curves of Atucha II reactor building by forcing at X+18.8m and at Y+18.8m

can be regarded as a rigid body, and the results show almost the same values for both amplitude and phase lag. The peak of 7.3Hz for the excitation in the X direction corresponds to the peak of 6.2Hz for the excitation in the Y direction. This difference in frequency is because the influence of the large opening at the 180 degree position. The influence of the opening is smaller for the X excitation, as it is out of the plane when the force is applied for the X excitation, while for the Y excitation, the large opening is in the plane of excitation, thus weakening the stiffness of the R/B. Fig.5 shows vibration mode shapes at 3.5Hz. Since the phase lags at measurement points of the structure at 3.5Hz are almost the same and equal to around 90 degrees, it is assumed to be the fundamental vibration mode shape of soil~structure interaction.

2. VIBRATION TESTS OF LARGE SCALE CONCRETE MODELS

The large scale field tests were performed on the grounds of Tadotsu Engineering Laboratory, Nuclear Power Engineering Center (NUPEC), Kagawa Prefecture, Japan in 1988, in order to verify the seismic stability of soil appertained to the siting technology on Quaternary deposits. For the field tests, two concrete blocks, block A and block B, were built on Quaternary gravelly soil deposits. The block A is weighing 30MN with earth contact pressure equivalent of actual reactor building, and the block B is weighing 50MN. The verification of soil-structure interaction were executed by dynamic loading tests. For the test site ground, a diluvium sand and gravel layer was



X+18.8 Excitation

X+0.5mExcitation

Fig.5 Vibration mode shapes of Atucha II reactor building (f=3.5Hz) chosen, which has high possibility of being the bearing soil when building a nuclear power plant on the Quaternary deposit. There was a surface layer of about 10m thick reclamation soil on top of the selected test gravel layer, therefore, the ground was excavated to 11m below the ground surface for constructing the concrete blocks. The ground water level was lowered by using wells and controlled to hold the level of 1.5m beneath the excavated ground surface.

Fig.6 shows general view of the field test models in Tadotsu site. Fig.7 shows the relations of the test ground and the concrete blocks A and B.

The block A of 10m height was designed to have the plan dimensions of 8m×8m at the lower part and 12m × 12m at the upper part, and to have the height 2m and 8m respectively, so that the

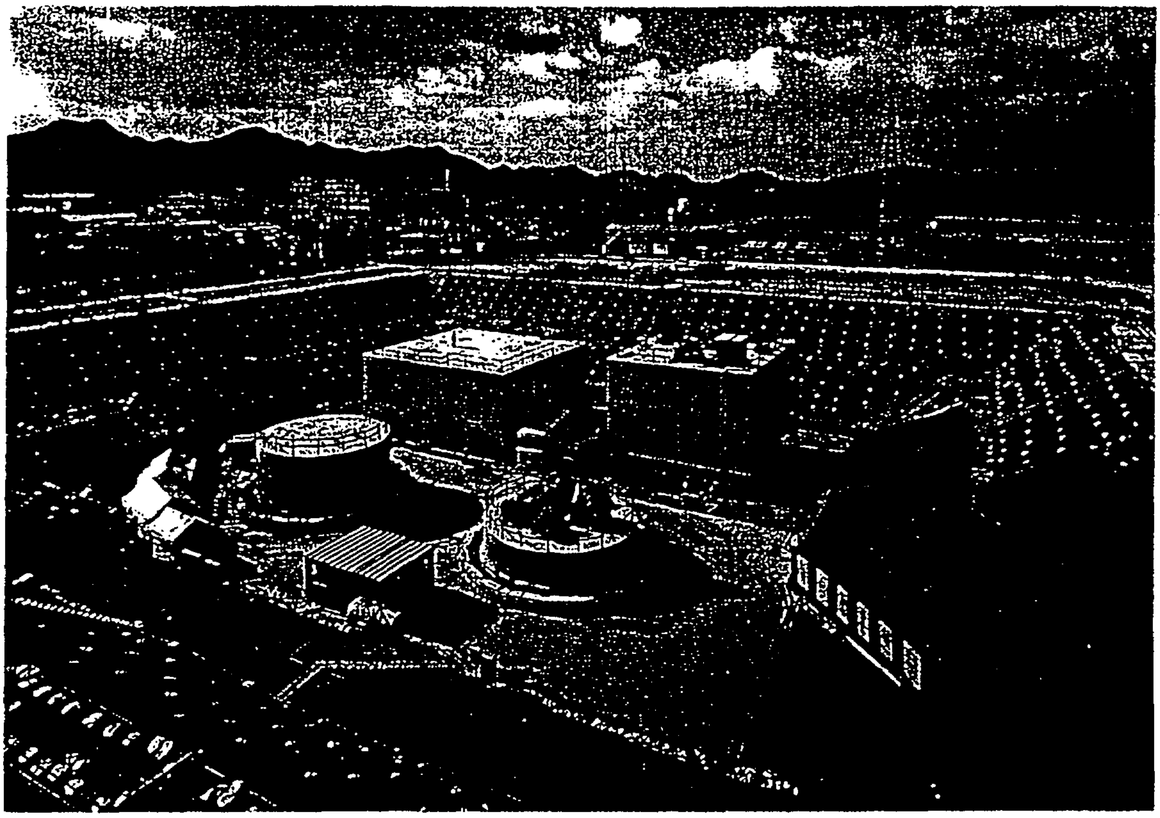


Fig.6 General view of field test models in Tadotu site

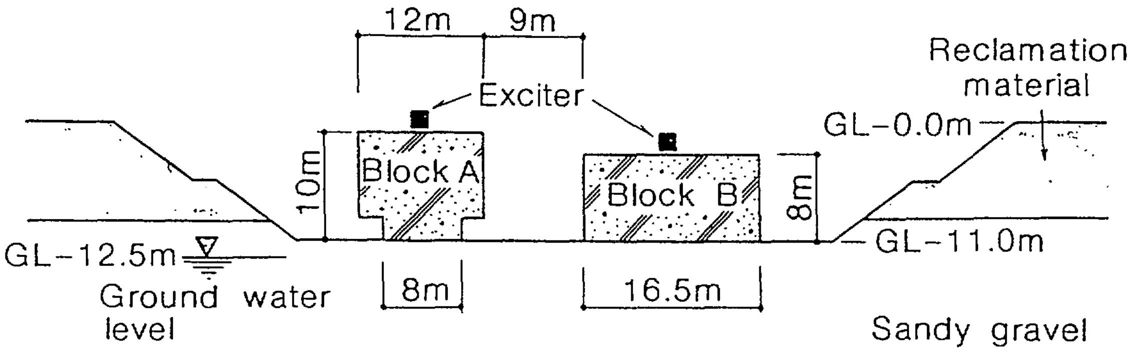


Fig.7 Concrete blocks A and B constructed on the test ground

contact pressure of approximately 470kPa could be attained. Regarding the soil-structure interaction, in order to assume the correlation with an actual building, because the non-dimensional frequency a_0 of the block A is small at 0.53, the block B was made to have the plan dimensions of 16.5m × 16.5m and 8m height, so that the non-dimensional frequency 2.09 would be the same as the actual building at approximately 2.0.

The dynamic loading test was performed by installing two sets of exciters on the top of the concrete blocks. The exciters were installed parallel to the excitations in the X direction and Y direction. Each of the selected exciter possessed the capacity; maximum eccentric moment of 6.2kN·m, maximum exciting force of 98kN, excitation frequency of 0.2Hz to 20Hz, and with plan dimensions of 2.2m × 3.7m. The excitation force was determined after confirming of its being sufficiently within the elastic limitation of the ground. Applied forces were P=19.6kN for the block A, and P=196kN for the block B. The excitations were conducted taking the procedure of increasing frequency, and were carried out by steady state tests.

Fig.8 and Fig.9 show the resonance and phase lag curves obtained at the top of the blocks A and B respectively. Each of the curve is an average of three points indicated in the figures. The

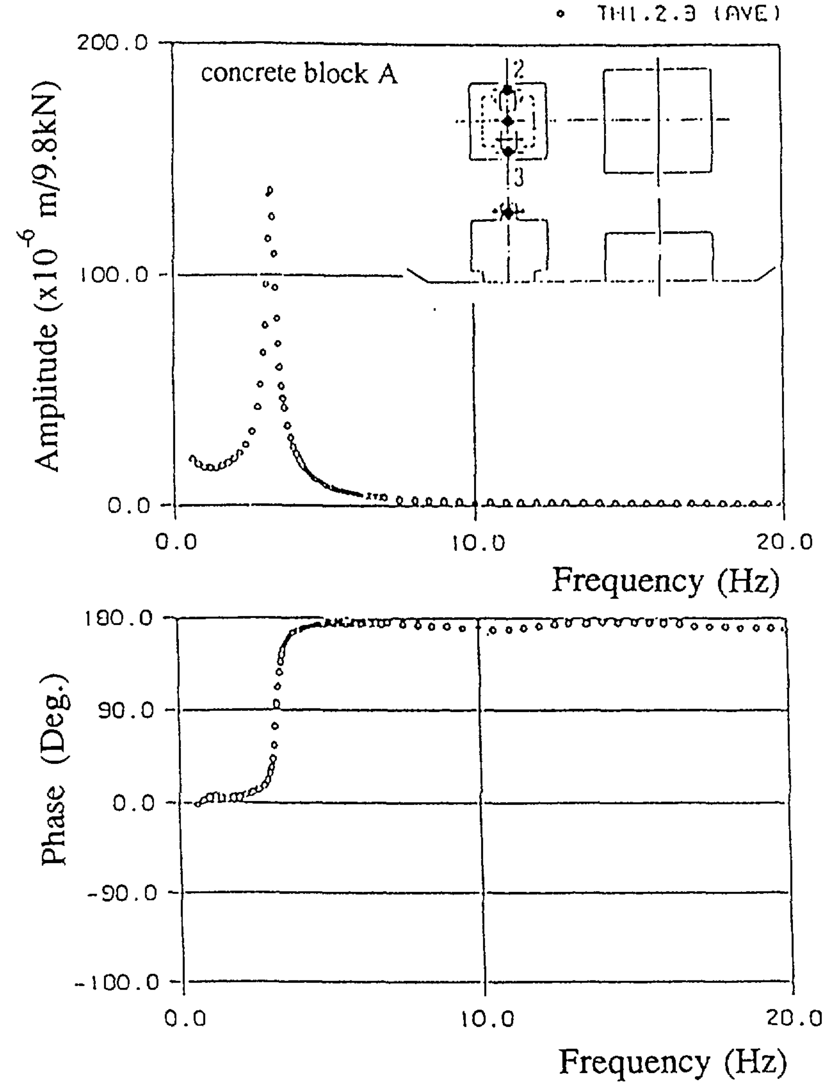


Fig.8 Resonance and phase lag curves of concrete block A
(X direction excitation)

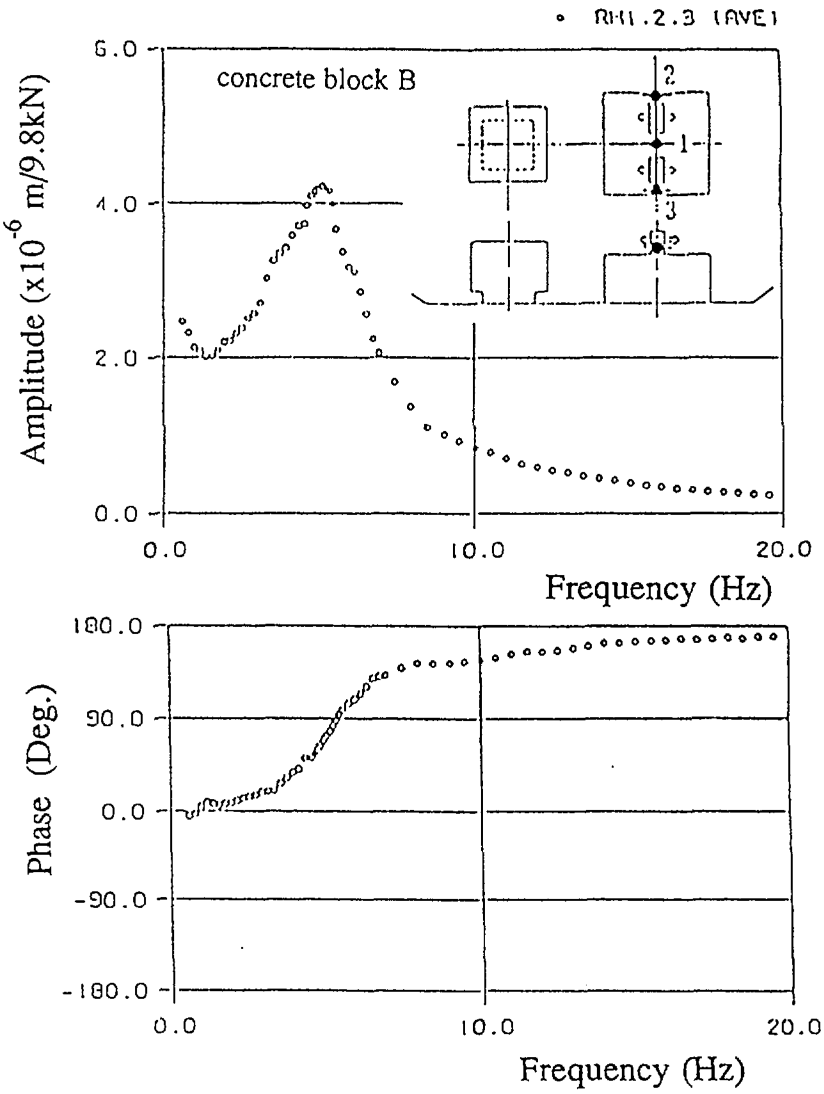


Fig.9 Resonance and phase lag curves of concrete block B
(X direction excitation)

amplitudes of the resonance curves are normalized to those corresponding to 9.8kN excitation, and the phase lag curves are indicated in term of phase lag from exciting force. Regarding the resonance curves for both blocks A and B, only the fundamental resonance frequency is shown to be predominant in the range of 1.0Hz~20Hz, and the difference between X and Y directions is quite small. The fundamental damping ratios obtained by power method are 5% for block A and 28% for block B in the both directions.

3. COMPARISON OF THE TESTS RESULTS

The comparison of the test results were performed in the following procedures. First, the calculation of soil springs by back fitting analyses of the test results were carried out for both of Atucha II reactor building and concrete block models. Fig.10 and Fig.11 shows soil springs concentrated at the basemat bottom of Atucha II reactor building derived by back fitting analysis in X direction and Y direction respectively. Fig.12 and Fig.13 shows soil springs of concrete block A and block B derived by back fitting analysis in Y direction forcing.

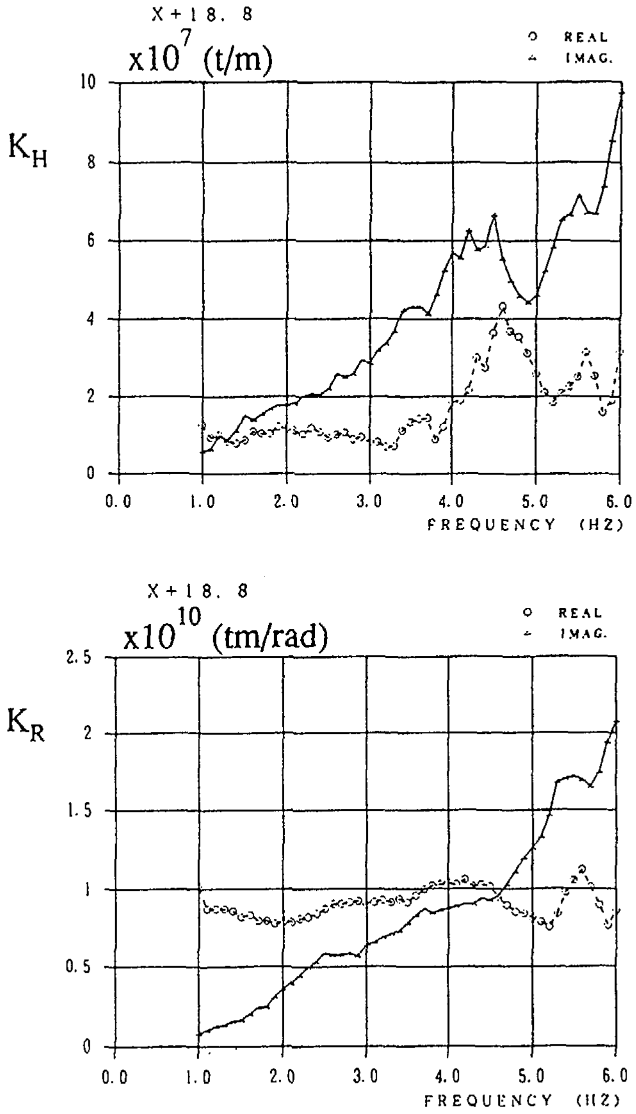


Fig.10 Soil springs concentrated at the basemat bottom of Atucha II reactor building derived by back fit analysis (X direction excitation)

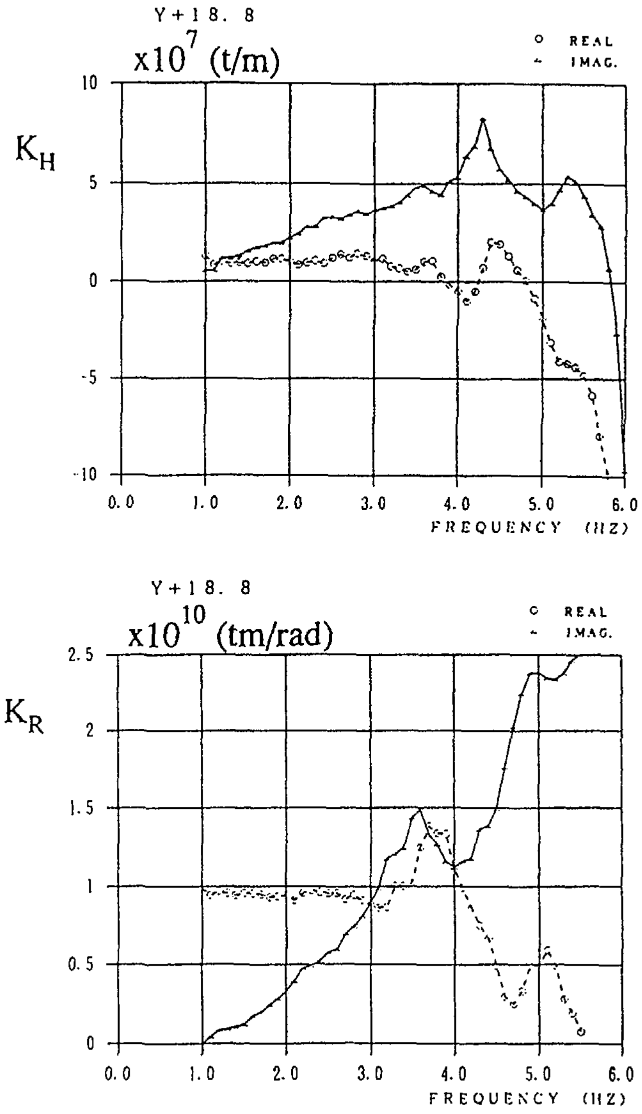
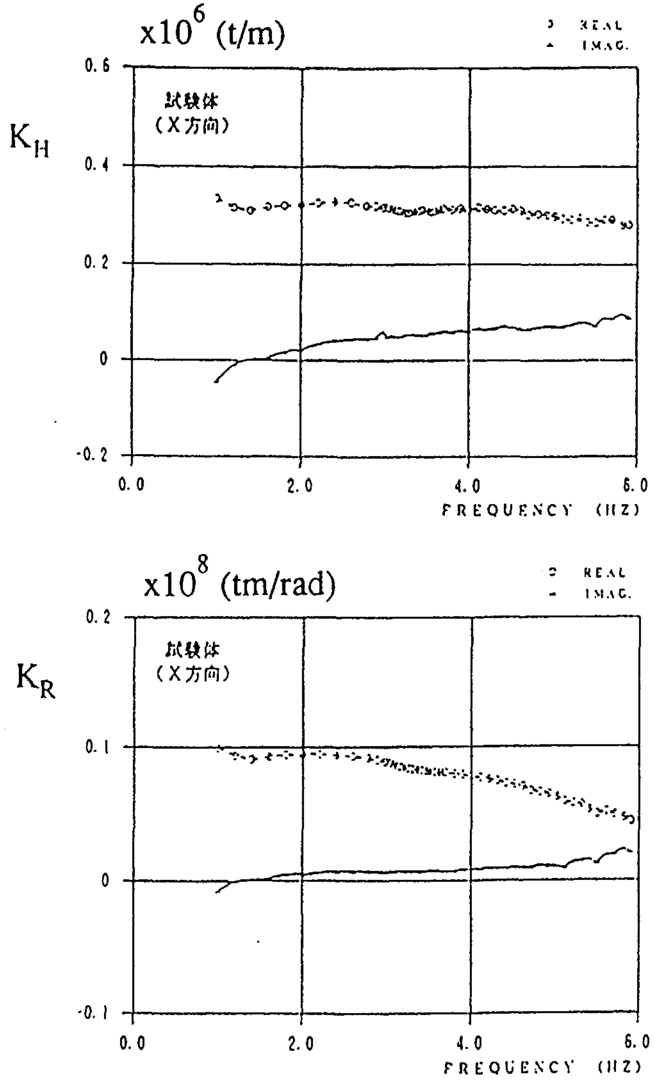
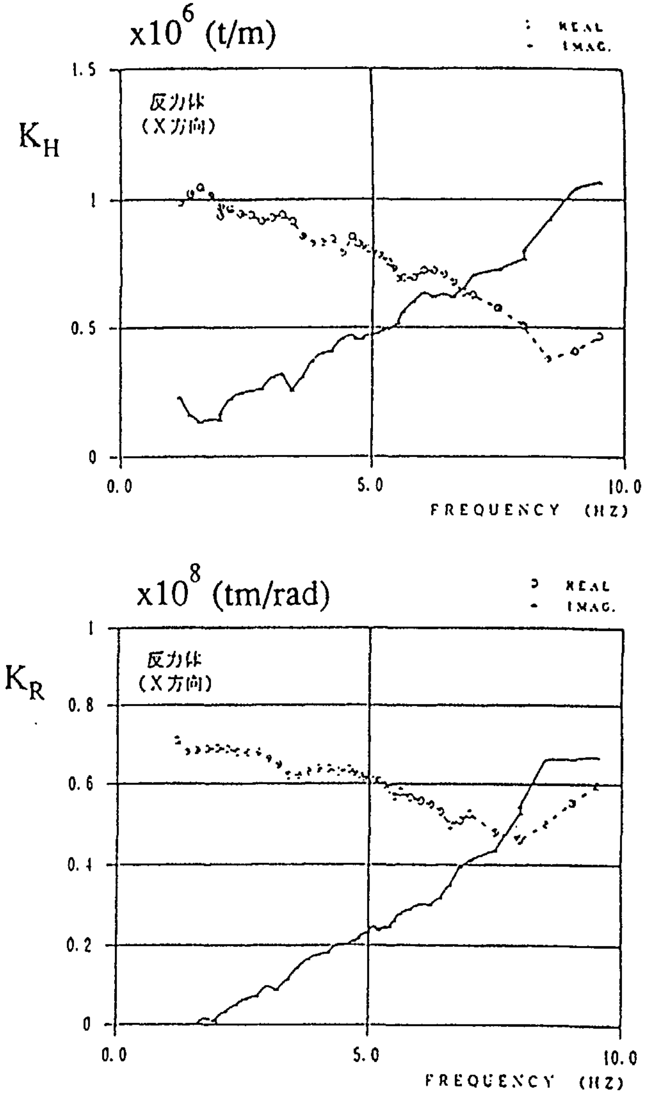


Fig.11 Soil springs concentrated at the basemat bottom of Atucha II reactor building derived by back fit analysis (Y direction excitation)



concrete block A

Fig.12 Soil springs of concrete block A derived by back fit analysis (X direction excitation)



concrete block B

Fig.13 Soil springs of concrete block B derived by back fit analysis (X direction excitation)

The soil springs of the Atucha II reactor building are included the embedment effects but the concrete blocks were not embedded. Hence, the soil springs with embedment of the Atucha II reactor building were translated to the soil springs without embedment using the coefficient ratio of soil springs with embedment and without embedment that were derived by the axisymmetric FEM analysis. Fig.14 shows the analysis result of the soil springs represented at basemat bottom of Atucha II reactor building in comparison of the with and without embedment. Fig.15 shows the coefficient ratio of the soil springs with and without embedment. The soil springs of Atucha II reactor building derived by back fitting analysis were converted to without embedment using the coefficient ratio shown in Fig.15.

Fig.16 and Fig.17 shows soil springs of Atucha II reactor building converted to without embedment, forcing at X +18.8m and forcing at Y +18.8m respectively.

Then, the damping constants were calculated using the complex soil springs obtained in order to make easy the comparison of the test results that were reflected the different structure and soil conditions. The damping constants were evaluated by the complex springs as considering that the real number portion represent the stiffness and the imaginary number portion represent the

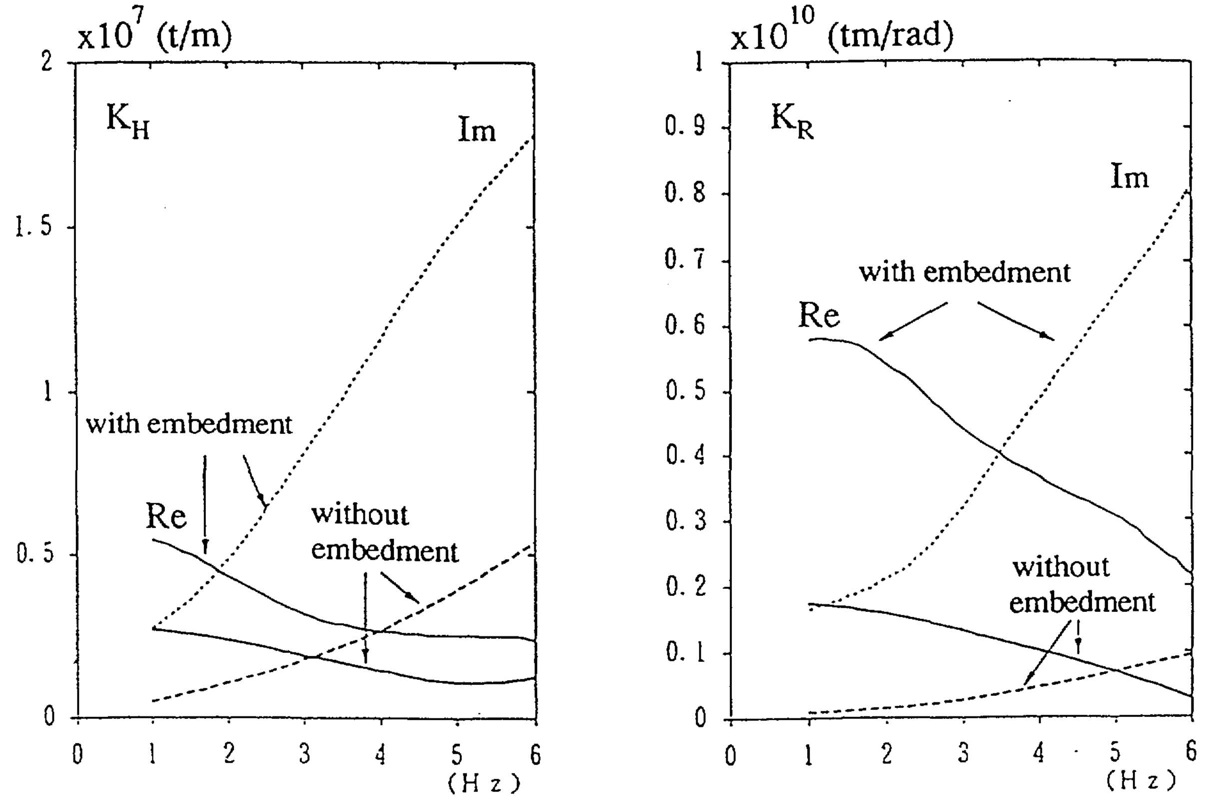


Fig.14 Comparison of the soil springs concentrated at basemat bottom as the with and without embedments derived by axisymmetric FEM analysis (Atucha II reactor building)

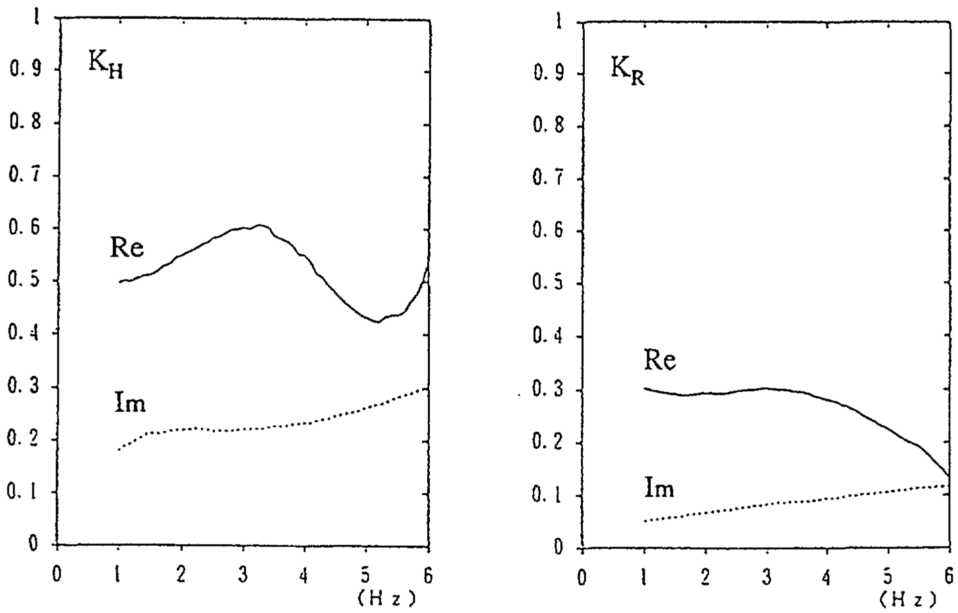


Fig.15 Ratio of soil springs of the with and without embedment (Atucha II reactor building)

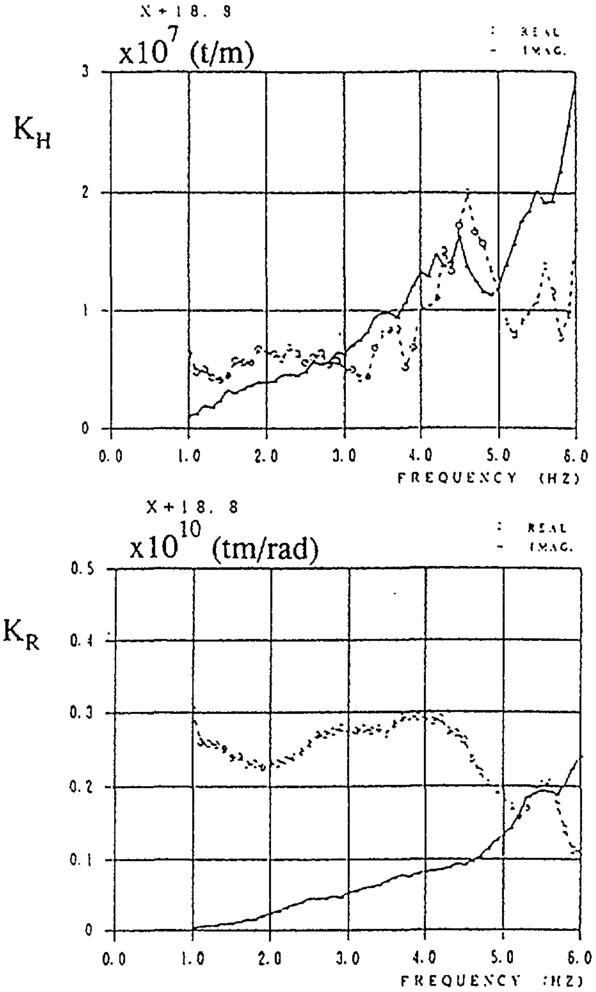


Fig.16 Soil springs of Atucha II reactor building derived by back fit analysis converted to without embedment (Forcing at X+18.8m)

damping. The damping constants were compared with the theoretical values for the three kinds of soil contact pressure distributions of Rigid plate, Uniform and Parabolic distributions derived by the vibration admittance theory. Fig.18 shows damping constants of concrete block A, block B and Atucha II reactor building, comparing test results and the theoretical value by vibration admittance. The damping constants of the horizontal components of the soil springs showed that the concrete block A (ao=0.53) and concrete block B (ao=2.09) showed the value like the Rigid plate distribution. The damping constants of the rotational components of the soil springs showed that the concrete block A (ao=0.53) and concrete block B (ao=1.74) showed larger value than the Rigid plate distribution and the Atucha II plant showed the value like Parabolic distribution.

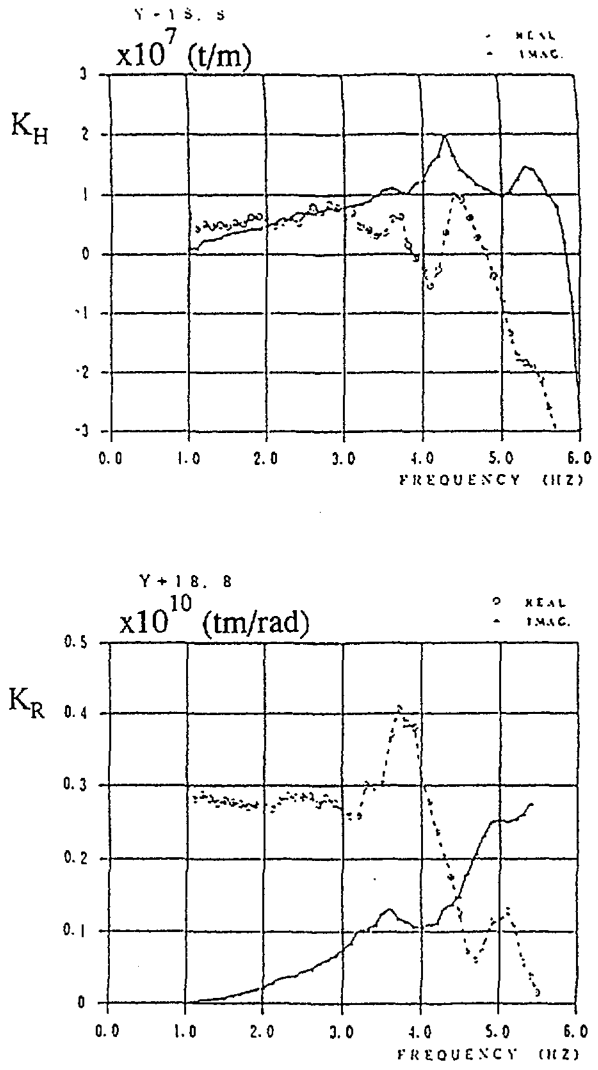
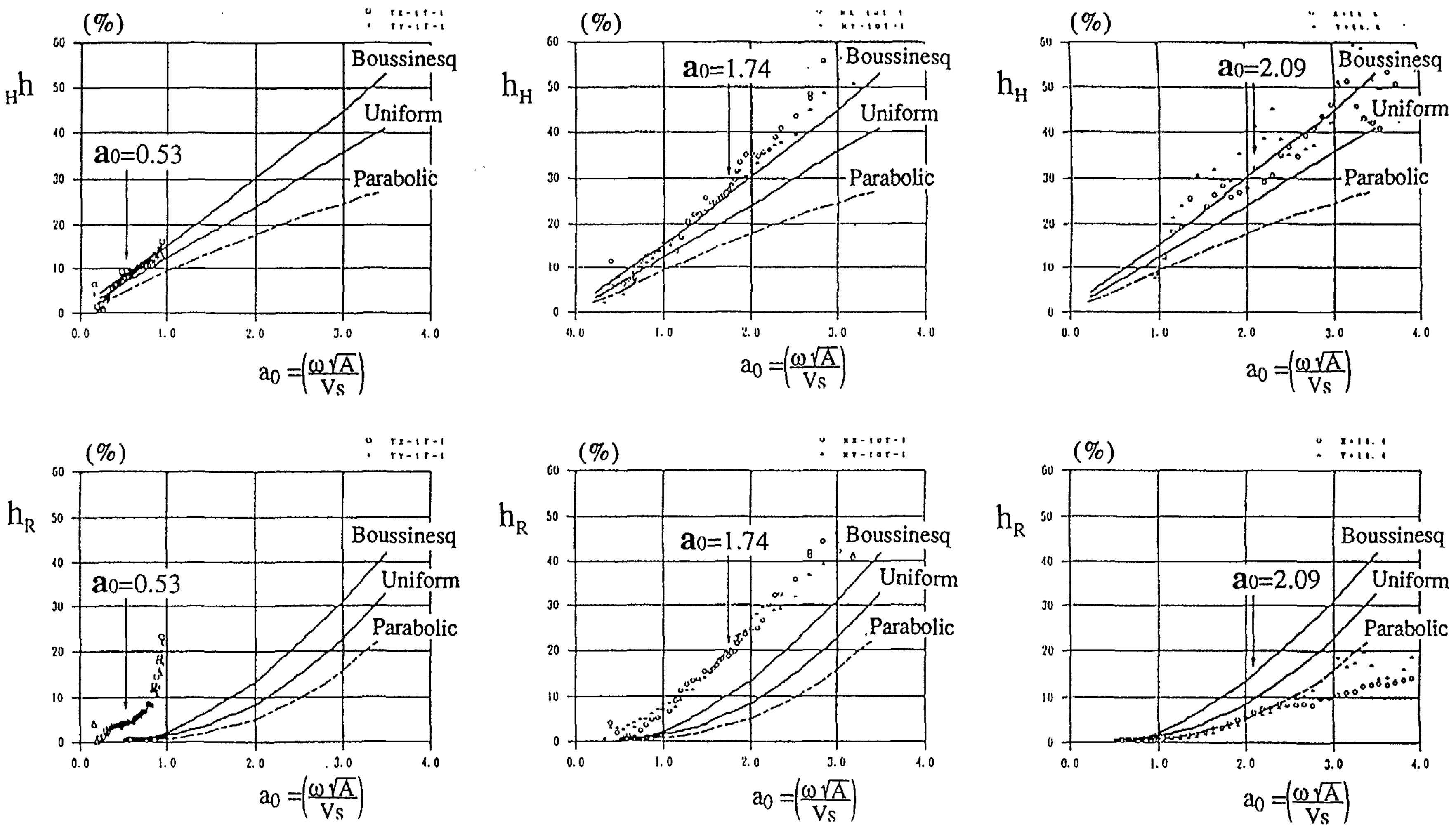


Fig.17 Soil springs of Atucha II reactor building derived by back fit analysis converted to without embedment (Forcing at Y+18.8 m)



(a) cocrete block A (b) concrete block B (c) Atucha II reactor building

Fig.18 Damping constants of concrete block A, block B and Atucha II reactor building comparing with the test results and the theoretical value by vibration admittance

4. Conclusion

From the comparison of the soil springs derived by the vibration test results for the large scale concrete blocks and the actual nuclear power plant, although, these test structures have different dimensionless frequencies as ao=0.53, 1.74 and 2.09 the soil springs characteristics observed were well correspond with the theoretical value and followings were identified.

1) To evaluate the horizontal springs of actual plants, the stiffness of the foundation can be considered as rigid plate and the distribution of the bearing soil pressure can be estimated by Boussinesq's formula.
2) To evaluate the rotational springs of actual plants, the stiffness of the foundation is mainly considered as elastic plate and the distribution of the bearing soil pressure would be varied between Uniform and Parabolic distributions.

ACKNOWLEDGEMENT

This paper described a part of the "Study on the siting technology of nuclear power plants on Quaternary deposits" by Nuclear Power Engineering Corporation (NUPEC), under the sponsorships of Ministry of International Trading and Industry (MITI) of Japan. The sponsorships and efforts made by all the member of this study are greatly appreciated.

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