EARTHQUAKE OBSERVATION AND COUPLED VIBRATIONAL ANALYSIS OF CYLINDRICAL WATER STORAGE TANK

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SUMMARY

In order to study the dynamic characteristics of a cylindrical water storage tank and to obtain future reference material on earthquake resistant design, earthquake observation has been carried out since 1979. The acceleration of the prestressed concrete tank and the surrounding ground, the strain of the tank wall, and dynamic water pressures have been measured.

This paper shows the dynamic characteristics of the prestressed concrete tank obtained from observed earthquake records and the propriety of the numerical model of soil-foundation-superstructure interaction analysis comparing observed and computed values.

KNOWLEDGE OBTAINED FROM EARTHQUAKE OBSERVATION

Outline of soil and tank

Photo.1 shows a general view of the prestressed concrete tank where earthquake observation has been carried out. In 1979, the tank, 28m in diameter and 8.32m in depth, was constructed in Hachinohe as shown in Fig.1, in northern Japan, to supply water to 20,000 citizens. This area was struck by severe earthquake (magnitude 7.9) in 1968. 49 people were killed and approximately 40,000 houses and buildings were destroyed.

Fig.2 shows the soil conditions and the shear wave velocity of the site.

The shear wave velocity of the loam layer over G.L.-11m varies from 120 m/s to 220 m/s and that of the sandy gravel layer under G.L.-11m is 400 m/sec.

The foundation piles are supported by this sandy gravel layer.

The locations of the measuring instruments are shown in Fig.3. Acceleration is measured on the top of the tank wall (A1), on the bottom plate (A2), on the ground

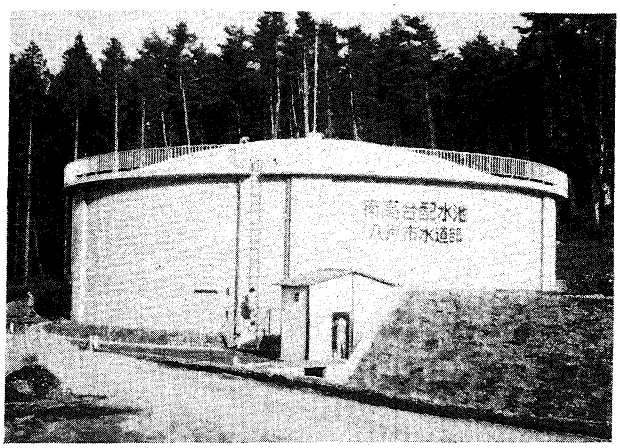

Photo.1 Outside view of PC tank

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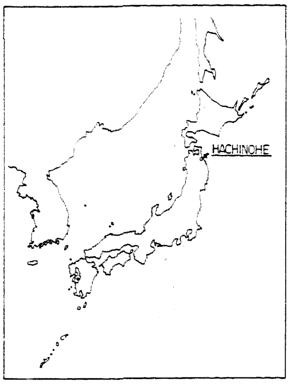


Fig.1 Location of earthquake observation

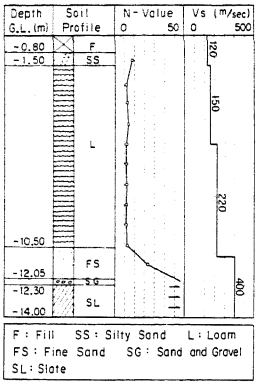


Fig.2 Soil conditions and shear wave velocity

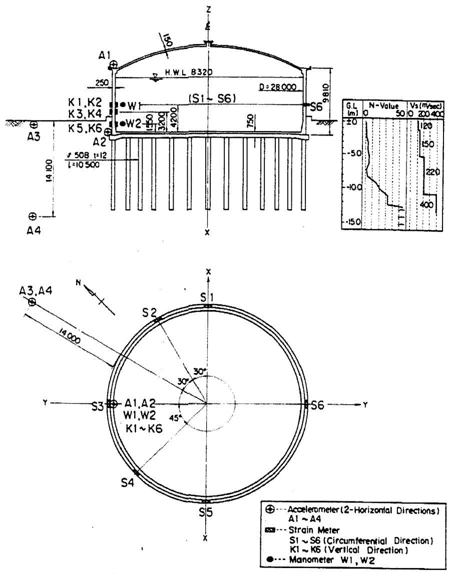


Fig.3 General view of PC tank and instrument locations

surface (A3), and on the sandy gravel layer (A4). In the circumferential direction, strains of the tank walls outside surface (S1∿S6), and in the vertical direction, strains of the inside and outside walls (K1∿K4), and dynamic water pressures (W1,W2), are measured.

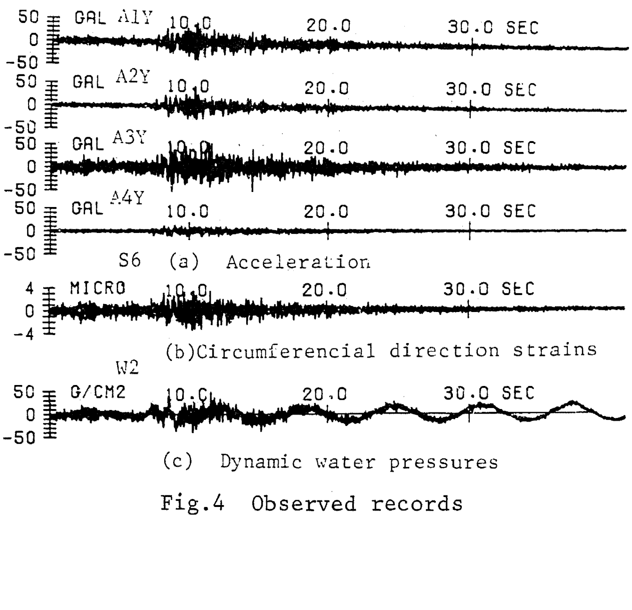
Vibrational characteristics

The earthquake reported in this paper occurred on December 2, 1981. This earthquake was of magnitude about 6.6, the epicentral distance was 78km, and the hypocenter was rather deep at around 40km. According to the acceleration records shown in Fig.4, the acceleration of the top of the tank wall, is at most, 1.1-1.2 times as large as that of the bottom plate.

On the other hand the acceleration of the surface ground is approximately 4 times as large as that of the base layer.

Fig.5 shows a transfer function of the acceleration of the top of the tank, bottom plate, and ground surface compared to the acceleration of the base layer.

The primary dominant period of the ground is 0.3 sec. The magnification of vibration with 0.25 sec., which is predominant on the top of the tank wall is the



(a) Acceleration

(b)Circumferencial direction strains

(c) Dynamic water pressures

Fig.4 Observed records

same as that of the bottom plate. Namely, this vibration is a sway motion of the tank including the pile foundation shown in Fig.6. It is a little shorter than the primary dominant period of the ground because piles are driven into the ground under the bottom plate. Vibration with 0.085 sec., which is next most prevalent on the top of the tank wall, does not exist in the bottom plate, and amplifies through the tank wall.

It can be considered that this vibration is a bulging motion (Ref.1) caused by the interaction of the tank wall with water. However, it is not a dominant mode of vibration in earthquake resistant design because the wall of a prestressed concrete tank like that in this study is so stiff that the period of bulging motion is shorter than the dominant period of earthquake used in earthquake resistant design.

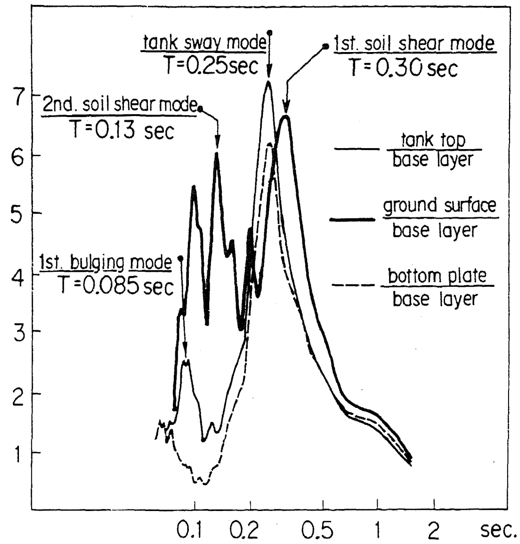


Fig.5 Transfer functions of tank and soil

On the ground surface a short-period component wave of under 0.2 sec. is greatly amplified due to multiple reflection into shallow soil layers of 3 or 4m deep. On the other hand, the bottom plate penetrates 3m underground and is much stiffer than the ground, so the response of a short-period component wave of under 0.2 sec. of the bottom plate is smaller than that of the ground surface.

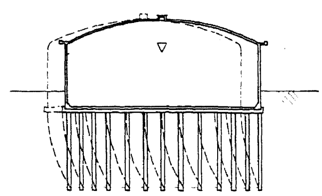


Fig.6 Sway motion of tank

NUMERICAL STUDY

Flow of simulation

Structures required to be highly stable in earthquakes, such as water storage tanks, are frequently designed not only by static analysis but also by dynamic response analysis. The authors set up a flow chart of simulation as shown in Fig.7.

As a whole vibration system model in Step 1, a 3-D axisymmetric finite element model, a 2-D finite element model or a lumped mass-spring model is used in response to the grade of earthquake resistant design.

In this paper an attempt has been made to clarify the accuracy and characteristics of these three models according to the analyses using observed earthquake records.

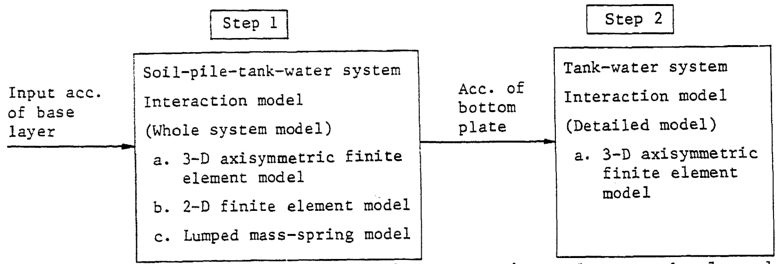


Fig.7 Flow chart of dynamic analysis and numerical models

Numerical models

a) 3-D axisymmetric finite element model

Fig.8 shows a 3-D axisymmetric finite element model. Soil and the bottom plate are expressed as solid elements. On the boundary of the soil model, wave dispersion in the surrounding ground is considered using the viscous boundary (Ref. 2).

The soil and piles under the bottom plate are assumed to be in the form of soil reinforced with piles, and these are treated as solid elements which cause equivalent displacement to that of the prototype upon horizontal force, vertical force, and the overturning moment as shown in Fig.9. The tank wall and water are treated as solid elements whose masses and natural period are equivalent to the bulging motion caused by the interaction of the tank wall with the water.

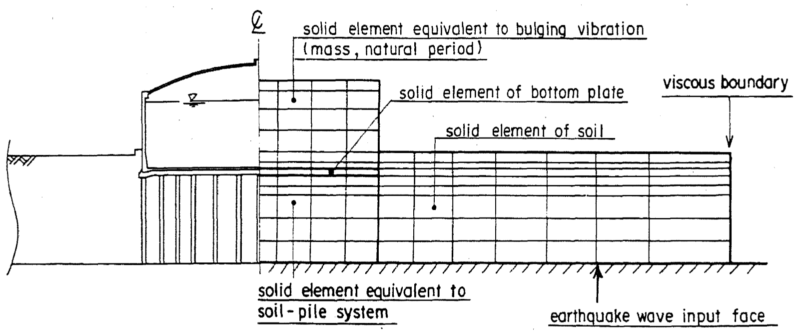


Fig.8 3-D axisymmetric finite element model

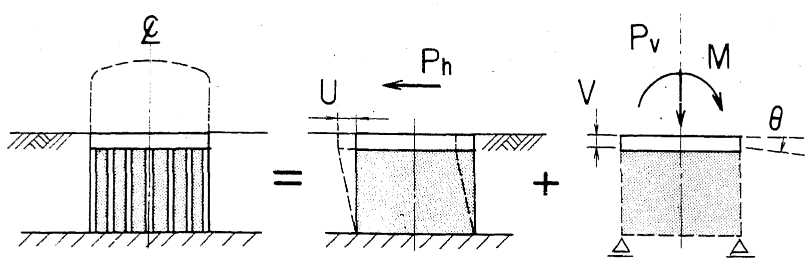


Fig.9 Model of soil and piles under bottom plate

b) 2-D finite element model

As shown in Fig.10, the region in which the superstructure influences ground vibration is assumed to be an effective width (W), so that the mass and stiffness of the superstructure and piles are considered to be distributed equally over the effective width. According to this assumption, these can be approximately treated as the 2-D finite element model shown in Fig.11. This model has the following two characteristics: the piles can be treated as beam elements and the non-axisymmetrical layers can be treated as solid elements.

On the boundary of the soil model, wave dispersion in the surrounding ground is considered using the energy transmitting boundary (Ref. 3).

In this analysis, as the effective width of ground becomes extensive, the relative mass of the ground to the superstructure becomes larger and the influence of ground vibration on structure vibration increases. With 2-D finite element model, the effective width from 1D to 4D (D: diameter) is analyzed and the most suitable effective width can be decided.

c) Lumped mass-spring model

Referring to the study by Housner (Ref. 4) and Sakai (Ref. 1), the storage water is treated as the impulsive pressure component and the convective pressure component supported concentrated springs equivalent to bulging and sloshing vibration as shown in Fig.12. Foundation piles are treated as a single beam column which has equivalent lateral stiffness and as a rotational spring which has equivalent rotational resistance. Ground vibration is treated as a far field model which is not influenced by the structure vibration, and the base acceleration and response displacement computed from the far field model are input into the lumped mass-spring model simultaneously.

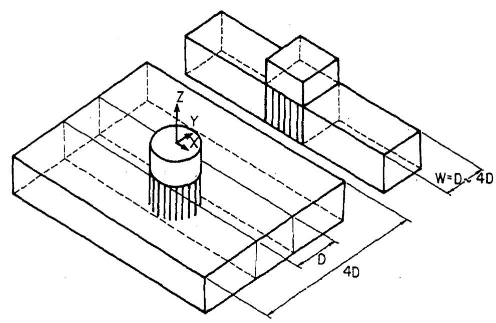


Fig.10 General view of 2-D finite element model

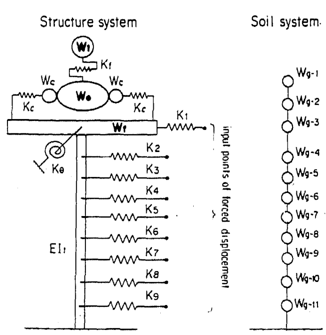


Fig.12 Lumped mass-spring model

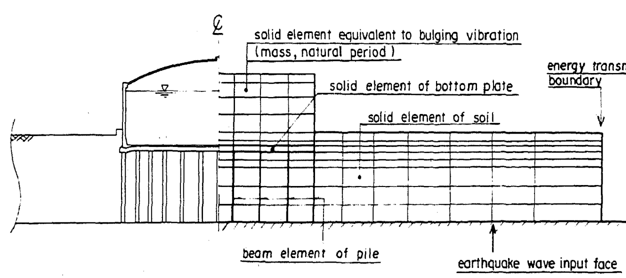


Fig.11 2-D finite element model

Input acceleration

The base acceleration wave (A4Y), shown in Fig.5, was input into the base layer of the numerical model. The maximum amplitude of acceleration of this wave is about 13 gal, and this is almost 1/10 of the maximum amplitude of acceleration used in earthquake resistant design in Japan.

Shear modulus and damping factors of soil

Table-1 shows the soil properties obtained from prospecting by drilling and seismic prospecting. In analyses, soil properties, depending on the amplitude of the shear strains, were adopted with consideration of the non-linearity of soil (Ref. 5).

Table-1 Soil properties

	Height (m)	Unit weight (t/m^3)	Shear wave velocity (m/sec)	Shear modulus (t/m^2)
Fill	1.16	1.4	120	2,060
Loam	3.95	1.4	150	3,210
Sand	5.60	1.7	220	8,400
Sand & Gravel	—	1.8	400	29,390

Results of analyses

Table-2 shows the maximum response acceleration values of the three numerical models. According to this table, the errors with these three models, compared to the observed values, are within the limits of about ±30%, and from the designing point of view it can be said that each model is almost satisfactory, accuracy-wise. The error with the result of the 3-D axisymmetric finite element model compared to the observed value is within the limit of about 10%. Therefore, the accuracy of this model is the greatest. Fig.13 shows the response acceleration waves and observed acceleration waves of the bottom plate by 3-D axisymmetric finite element analysis. The waves are almost consistent.

In 2-D finite element analysis, shown in Fig.14, the response acceleration of the bottom plate and ground surface respectively, are most consistent with observed values when the effective width is equal to 2D (D: diameter) so we adopted the effective with 2D in 2-D finite element analysis. The use of the 2-D finite element model is an approximative method of treating

Table-2 Comparison of max. values by interaction analysis

Acc.* / Model	Ground surface		Bottom plate		Center of impulsive pressure		shear force under bottom plate ***	
	gal	ratio	gal	ratio	gal	ratio	ton	ratio
3-D axisymmetric finite element model	51	0.96	45	1.05	52	1.13	179	1.10
2-D finite element model	58	1.09	39	0.91	40	0.87	144	0.88
Lumped mass -spring model	43	0.81	55	1.28	61	1.33	213	1.31
Observed values	53	*1.0*	43	*1.0*	46**	*1.0*	*163*	*1.0*

* response values to input acc. (max.13 gal).
** calculated values from acc. of tank top and bottom plate.
*** calculated values from acc. of the center of impulsive pressure and bottom plate.

a 3-D structure as a plane strain model. It is expected however, that the values obtained with the 2-D finite element model will be similar to the values of the 3-D axisymmetric finite element model and the lumped mass-spring model if a suitable effective width is set. Considering the simplicity of models and the economical analysis cost, 2-D finite element analysis is effective when preliminary design necessitates rough values.

The acceleration of the bottom plate of the lumped mass-spring model is a little larger than that of the other two models. The assumptions that stiffness of the bottom plate is not considered, and the wave dispersion energy from the structure to the ground is not considered, etc., are possible reasons for this.

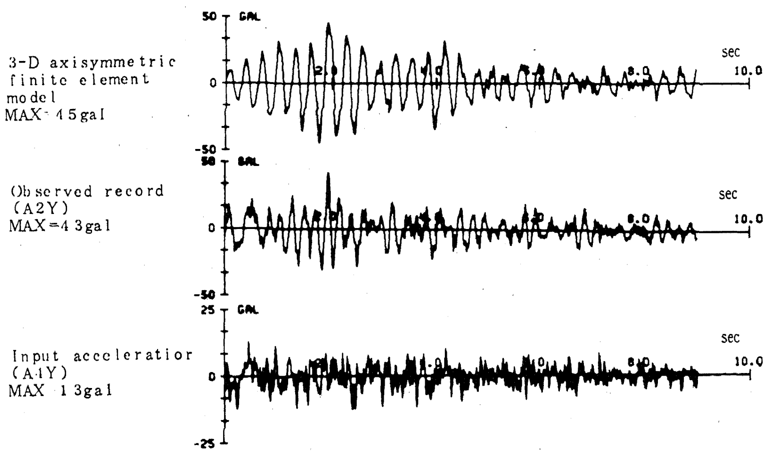


Fig.13 Response acceleration wave of bottom plate

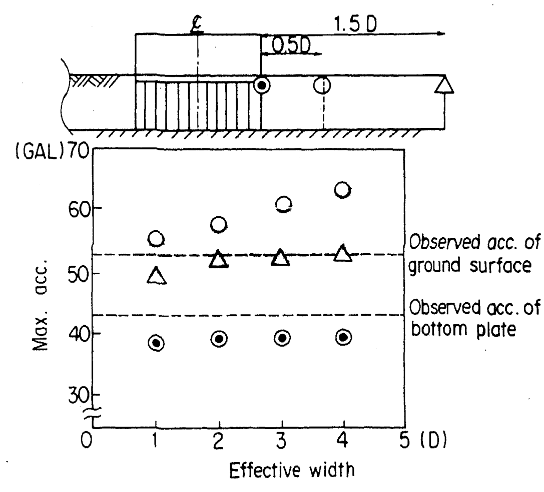


Fig.14 Relationship between effective width and max. acceleration

CONCLUSION

From the earthquake observations and some numerical analysis, the dynamic characteristics of soil-foundation-superstructure interaction were discussed. The major results obtained in this study can be summarized as follows:

(i) Rocking and bulging vibrations have little influence on the response of the tank, and the sway motion of the tank, including the pile foundation, is the most prevalent mode of tank vibration.

(ii) The maximum amplitude of response acceleration computed by 3-D finite element analysis was most accurate among the three analyses, although from the designing point of view, the results with all models were almost consistent with observed values.

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