AN EXPERIMENTAL STUDY ON THE EARTHQUAKE RESISTANT PROPERTY OF BREAKWATERS OF THE CYLINDRICAL SHELL TYPE

By

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SYNOPSIS

Recently, the breakwaters of the cylindrical shell type of prestressed concrete are constructed as large breakwaters of a new type. However, the stability of this kind of breakwaters during earthquake is not clarified because of its quite new type. In this study, the vibration tests have been done for two types of breakwater models and their stability has been discussed. By these tests, it has been found that one type of breakwater is pretty aseismic, but the structure of the other type must be changed. Also, in this paper, some data on the earthquake ressistant design of this type of breakwater are proposed.

INTRODUCTION

The stability of breakwaters has been discussed only from the view point of wave pressures. However, for the large breakwaters on deep water which are constructed recently, sometimes it is more influenced by earthquakes than by wave pressures.

This paper deals with the model experiment which has been done to investigate the earthquake resistance of the following two kinds of breakwaters of the prestressed concrete shell type; One is constructed at the site of about 11m depth in the water on the submarine clay of about 10m thickness, and the other is planned to be constructed at the deeper site in the water on the thicker clay. The latter model of the breakwater is preliminarily planned and its top is considered to be used as a highway, so that this may be called an embankment rather than a breakwater.

The prestressed concrete shell parts of both these breakwaters are constructed by the vacuum method. Namely by this method, the lower shell set on the sea bottom is covered by a lid, and the sea water in the shell is removed, and then by the external water pressure the shell is pushed into the submarine clay.

The structure of the breakwater of Type-A is shown in Fig. 1. In order to prevent the sand fill from flowing out, a packing of synthetic rubber is sandwitched between the upper and the lower shells. The top of the shell is coverd by a concrete lid and the parapet wall is constructed on it. In the opening between the shells, the steel cylindrical piles are driven in as shown in Fig. 1. On the other hand, the structure of the breakwater with the embankment of Type-B is shown in Fig. 2. The embankment part of the breakwater has a toe-shell and a protective work on the slope. The scales of the models are 1/15 for the Type-A and 1/30 for the Type-B in the size of the shaking tank. For the correspondence between the test results and the real behavior of the proto-

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type subjected to earthquake motions the law of similarity is needed in length, force, and time. At this point, the author follows the papers by Drs. R. W. Clough and H. B. Seed, etc.

As the sand layer, the sand fill of the shells, and the soil of embankment, the small grains of masa-do (the weathered granite sand) which are used for the sand fill of the prototype have been used, and as the clay layer of the models the remoulded clay which consists of the submarine clay of Kobe Harbor plus water has been used in the experiment. The similarity for the moduli of regidity of the soil materials of the models has been neglected because there is no big difference between the values required for the models and the actual values of the models (see Table 1). Since it is difficult to make the models with the stiffness similar to that of the prototype, only the weight of the models has been simulated. The models of Type-A have been made of concrete cylinders of the centrifugal force type of 1/15 in size and steel plates of 3.1mm in thickness. While, the models of Type-B have been made of steel plates of 2mm in thickness.

The next difficult problem is about the nature of the simulated earthquake. At present in Japan, there is no strong motion seismogram available for the aseismic design of habor structures. Therefore, in this study, by referring to 0.6sec of the period of the Kiisuido Earthquake in 1950 (max.acceleration of 20gal) at the peak of the acceleration spectrum and the natural period of the submarine clay, the periods of the simulated earthquakes applied to the models are assumed to be 0.11sec for the models of Type-A and 0.13sec for Type-B.

The constants for the models of Type-A and Type-B are shown in Table 1.

EQUIPMENTS AND PROCEDURES OF THE EXPERIMENT

A general view of the shaking tank is shown in Photo 1. The shaking tank is 4.8m long, 2.0m high, and 1.56m deep and made of steel. One side is covered by glass. The shaking tank is hung by four wire ropes and has a stopper in the lateral direction. The vibration of the tank is started by an impact of the 500kg weight of the pendulum. The impact has been given only once and the second shock has been prohibited by operating the rope attached to the weight of the pendulum. Photos 2 and 3 show the buffer spring and the anchor spring, respectively.

The numbers of the models tested are three of the steel shell models and four of the concrete shell models for Type-A, and for Type-B, three of the steel shell models, and one of the model without the shell and with an embankment on the wall of the tank.

The vibrating test of each model has been done as follows; one shock is given 8~11 times from 0.6 to 1.7g for Type-A and 7~9 times from o.7 to 1.2g for Type-B by the step of 0.1g of the accelereation of the tank. Then, for each shock the horizontal amplitude, during the vibration and the horizontal and vertical displacements after the shock of the shaking tank and the quaywall shell and the accelerations of the sand fill, the embankment and the submarine clay have been measured. After finishing the above measurements for one model, in order to measure the rigidity of the sand fill of the embankment, the propagating velocity of the elastic wave has been measured by the buried accelometer. At last, the dried density and water content of the sampled soil has been measured. Therefore, it must be notified that the measured values in the diagrams are those for the accumulated shocks.

In additin, for a few models, the loading tests of the shells are performed and the relationships between the load and the displacement are investigated for the purpose of the aseismic design.

TEST RESULTS

(1) Model of Type -A

Vibrational Behaviors of the shells:

Fig. 3 represents an example of the experimental records obtained for the case of 0.26g of the ground acceleration. As shown in the displacement record of the top of shell, the vibration of the shell after the maximum amplitude is reached is a comparatively regular damped sinusoidal vibration. From the average period of this part, the natural periods of the shells are observed to be 0.17~0.22sec for the steel models and 0.19~0.24sec for the concrete models. Fig. 4 shows an example of the distributions of the accelerations of the tank, the shell, and the sand fill. Through all the models, the acceleration of the top of the shell is about 2~4 times as large as that of the tank and the acceleration of the surface of clay layer is about 3~4 times as large as that of the tank. A clear relationship could not be observed between the acceleration ratio and the acceleration of the tank.

Displacements of the Shells:

Fig. 5 shows the relationship between the ground acceleration and the horizontal displacement of the top of the shell after the vibration excited by one shock. It is notified in Fig. 5 that every shell has displaced on the side of the anchor spring and that the displacements increase rapidly from 0.2~0.4g of the ground acceleration for the concrete models and from 0.6~0.7g for the steel models. It is also observed that the settlement of the shell is small comparing with the horizontal displacement of the shell and that the small amount of the elastic deformation and the fairly large amount of rotation has been caused in the shell.

Settlement of the Sand Fill:

Fig. 6 shows the relationship between the ground acceleration and the settlement of the sand fill measured at the center of the lid of the shell. For every model, the settlement of the sand fill increases almost linearly as the ground acceleration increases. For the steel model No. S-1 with the loose sand fill, the settlement changes suddenly in the range of 0.5~0.6g of the ground acceleration. Also, in this range of the acceleration, the outflow of the sea water from the circumference of the lid of the shell has been observed. Therefore, the liquefaction phonomenon is supposed to have occurred in the sand fill. By the way, this acceleration value corresponds to the suddenly increasing point of the displacement of the shell model, No. S-1, in Fig. 5.

Settlement of the Clay Layer:

It is observed that the settlement of the clay layer due to the vibration is small and it is about 1/2~1/4 of the settlement of the shell at the distance of 12cm from the shell model.

(2) Model of Type-B

Deformation of the Embankment before the Test:

Fairly large settlement of the embankment of model occurred even during the construction. Since the slope of the embankment was found to slide a little bit after the construction of the first model of the embankment, in constructing the next three models, the deformation of the embankment was observed by drawing a white line between each layers of 10cm thickness of the embankment.

The settlement of the embankment is large near its shoulder and it is about 6~7cm (12~14% of the height of the embankment) through all the models. It depends more on the sinking than or the tamping of the soil of the embankment (see Fig. 10). In addition, after the embankment was completed, the top of the shell inclined by 3~4cm on the sea side.

Vibrational Behaviors of the Models:

The models of Type-B are remarkablly different from those of Type-A at the point that a large permanent displacement remains during the vibration on the top of the shell. For all the models, the period of the principal wave of the ground acceleration is 0.11~0.14sec. The period of the displacement of the top of the shell is 0.26~0.29sec near the start of motion and the period of the principal wave is 0.26~0.30 sec. However, for the ground acceleration of more than 0.2g, the rocking vibration of the period 0.16~0.18sec appears pretty clearly after several waves. For the smaller ground acceleration the number of waves is some dozen while, for the larger ground acceleration of abount 0.7g, the number of waves is several and the duration of the vibration is very short. This is strongly influenced by the plasticity of soil. Since the shell and the embankment of the vibrating model do not always vibrate in phase, vibrational behavior of the models of Type-B is much more complicated than that of Type-A.

The distributions of the accelerations of the top of the shell and the top of the embankment when their accelerations are maxim are shown in Figs. 7 a) and b). The times of both maximum accelerations when the distributions a), b) are given do not coinside in general. In Figs.7 a) and b), the acceleration of the embankment is large comparing with the acceleration of the shell.

Especially, it is notified that the distribution of the acceleration of the embankment is in such a trianguler shape that it is zero at its bottom and maximum at its top. The average values of the ratio of the acceleration of the top of the shell to the ground acceleration for each model are 1.8 for No.1 and No.3 and 1.4 for No.4. The average acceleration ratios for the embankments are 1.4 for No.3 and 1.8 for No.4.

The reason why there is a difference of the acceleration between the models of No.3 and No.4 is supposed to be that the directions of both models are reverse each other and hence that the directions of the accelerations are also reverse. According to the above fact, the seismic coefficient for designs by the seismic coefficient method is recomended to be uniformly distributed and about twice as much as the ground acceleration in the direction of the sea for shells and to be triangularly distributed and about twice at the top as much as the ground acceleration in the direction of the harbor for embankments.

Displacements of the Shells and the Settlement of the Sand Fill:

The statical deformations of the shells after one shock are given in Fig. 8. In the case of the model No.4, the horizontal displacements of the shells depend mainly on the rotations as rigid bodies. Fig. 9 shows the relationships between the ground acceleration and the horizontal displacements of the tops of the shells. In this case, the shells subside as well as rotate, as a whole but the amount of the settlement is very small. The settlement of the shells is about 2~3cm

(4~8% of the height of the embankment) for the 0.7~1.2g of the ground acceleration and about a half of the settlement of the embankments.

Deformations of the Embankments and the Displacements of the Toe-shells:

The deformations of the embankments are caused mainly by the settlement near the shoulders and the raising of the slopes as shown in Fig. 10.

The former is about 5~6cm (10~12% of the height of the embankments) for 0.7~1.2g of the ground acceleration. Fig.11 shows the relationships between the ground acceleration and the maximum settlement of the embankments. The amount of the settlement increases remarkably from 0.1~0.3g of the ground acceleration.

Next, the horizontal displacements and the settlements of the toe-shells are small comparing with those of the quaywall-shells. Also, the settlement of the submarine clay is small.

Crucks and Other Damages

Regarding the crucks on the embankment by the vibration, some hair crucks have occurred in the direction of the axis of the embankment near the shoulder during the first shock of 0.04~0.08g. From the next shock of 0.1~0.13g fairly distinct crucks have appeared and, then, many crucks have occurred in the direction of the axis of the embankment as the ground acceleration increased. By such a process of the occurrence of crucks and the rising of the slope the sliding failure of the slope was expected, but according to the observation from the glass surface, neither sliding surface nor any change could be founded. In the case of 0.7~1.2g of the maximum ground acceleration of each model, the embankment was not failed.

SOME CONSIDERATIONS ON THE EARTHQUAKE RESISTANCE OF THE MODELS

Nature of the ground motion:

The ground motion applied to the models seem to be quite different from natural earthquakes by their external feature. However, in the problem of the earthquake effect against structures, the similarity of the acceleration spectrum of the ground motion comes into question. Although it is still in question to consider the acceleration spectra for the structures which mainly consist of soil, the acceleration spectra may give us pretty good estimates. Fig. 12 shows the acceleration spectra for the El Centro Earthquake (1940, N-S component, 0.32g), the Kiisuido Earthquake (1950, 0.02g) observed at the Kobe Harbor Construction Office, and the typical ground motion applied to the model. It is recognized from this diagram that for the period of larger than 0.17sec the acceleration spectrum of the ground motion of the model is a little larger than that of the Kiisuido earthquake and pretty larger than that of the El Centro earthquake, so that the ground motion against the model is satisfactory.

Loading Tests of the Shell Models:

To contribute to the stability calculation of the shell by the seismic coefficient method, the relationship between the horizontal force applied at the center of the upper shell and the deflection of the shell for the standard model of Type-A (1/15 on the scale) and the smaller model of the same type (1/40 on the scale) has been investigated and, then, the results of these static loading tests have been compared with the relationship between the maximum ground acceleration multiplied by the weight of the upper shell and the maximum amplitude in the dynamic test. For both models, the static displacement is about a half of the dynamic one. For the smaller model, the model was inclined as a whole, and the displacement of this model was of about the middle value between the above static displacement and the dynamic one. Therefore, for the displacement of the shell, the seismic coefficient for designs is needed to be about twice as big as the seismic coefficient of the ground.

For the shell models of Type-B, also, the loading tests have been done. As their results, the load-displacement diagrams appear to be a big hysteresis loop. In addition, the analysis of the vibration of the shell has been attempted for the model, but the static restoring force characteristics could not be made use of for the analysis of this system.

Stability Calculation of the Slope by the Sliding Circle Method:

For the fully reserved model, by calculating the stability of the slope by the sliding circle method referring to the appearance of the crucks of the embankment due to vibrations, the values of 0.15~0.18 have been obtained as the maximum seismic coefficient to keep it stable. In this case, the resisting moment by the slide friction of the side wall of tank was about 30% of that of the sliding circle. By comparing the safety factors for both the uniform distribution and the triangular distribution of the seismic coefficient under the same total seismic force, it is found that the latter is slightly small but there is no big difference between them.

CONCLUSIONS

In the experimental study on the stability of the structures mainly consisting of soil during earthquakes, as stated in the studies by Drs. H. B. Seed and R. W. Clough, etc., the following two difficult problems exist:

(1) The influence of the rate of loading on the material properties and

(2) The influence of internal pore pressures on the model behavior.

In discussing the results of this model experiment, also, careful discussions on the above items are necessary. The following items will be helpful in the aseismic design of the breakwater of the prestressed concrete cylindrical shell type.

Breakwater of Type-A:

(1) The natural period of the prototype is about 0.8sec. The elastic vibration is predominant for the ground acceleration of less than about 0.3g.

(2) The permanent deflection of the top of the shell due to the ground motion is less than 0.1% of the height of the shell when the ground acceleration is less than 0.4g. However, when the ground acceleration is larger than 0.5g, it increases rapidly and the joint between the upper and the lower shells is sometimes destroyed. This is supposed to be caused by the liquefaction of the sand fill in the shell.

(3) The settlement of the sand fill due to the ground motion is about 3% of the height of the shell for the ground acceleration of 0.4g. When the ground acceleration is larger than 0.5g, it increases rapidly.

(4) Therefore, the breakwater of this type can be made a good earthquake risistant structure by the following techniques; The use of the sand fill such as cobble stones of large rigidity near the joint between

the upper and the lower shells and to let the structure of the lid of the shell be that to reduce the pore pressure and the settlement of the sand fill.

Breakwater of Type-B

(1) Even during the construction of the embankment, the settlement of the clay ground is pretty large, so that the quaywall shell as well as the toe shell are required to be sunk into the sand layer pretty much.

(2) The settlements of the embankment and the sand fill due to earthquake motions are about 7~8% and 2~4%, respectively, of the height of the embankment for the ground acceleration of about 0.4g.

(3) The crucks appear along the shoulder of the embankment by the ground acceleration of about 0.1~0.3g and they grow as the ground acceleration increases. Also, the rising of the slope becomes pretty large.

By the way, the failure of the embankment could not be observed for the ground acceleration of 0.7~1.2g, no special change of the clay layer could be found, either.

However, in the case of the breakwater of Type-B, even for the small ground acceleration, the settlement of the embankment and the rising of the slope are pretty large, so that the slope part of the embankment is recommended to be changed into the rock fill or the quaywall shell. Furthermore, if possible, it is desirable not to make use of the breakwater for a highway, but to let it be a structure of Type-A.

The author expresses his sincere gratitude to the Third Harbor Construction Bureau at the Ministry of Transportation for executing this experiment.

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1) Construction works on the Fifth Breakwater at Kobe Port, Third Harbor Construction Bureau, Ministry of Transpotation, Japanese Goverment, (in Japanese)

2) R. W. Clough, D. Pritz: Earthquake Resistance of Rockfill Dams, Trans. ASCE, Vol. 82, Apr. 1956.

3) H. B. Seed, R. W. Clough: Earthquake Resistance of Sloping Core Dams, Proc. ASCE, Soil Mechanics and Foundation Division, Feb. 1963.

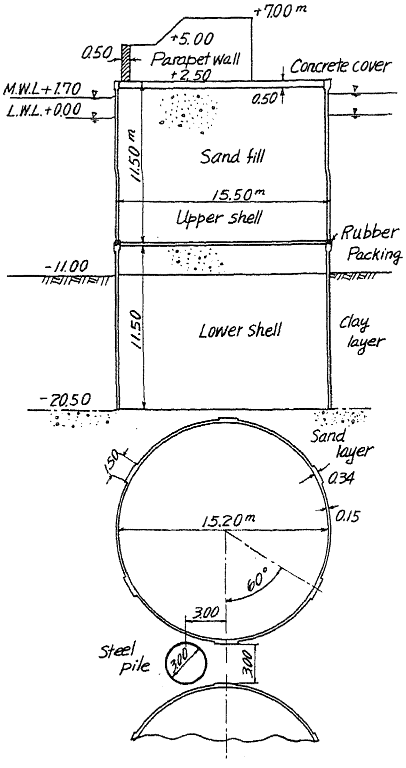


FIG. 1 TYPE-A BREAKWATER (PROTOTYPE).

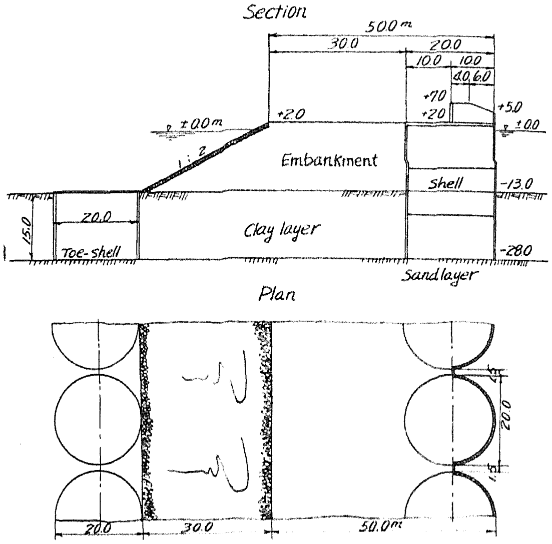


FIG. 2 TYPE-B BREAKWATER (PROTOTYPE).

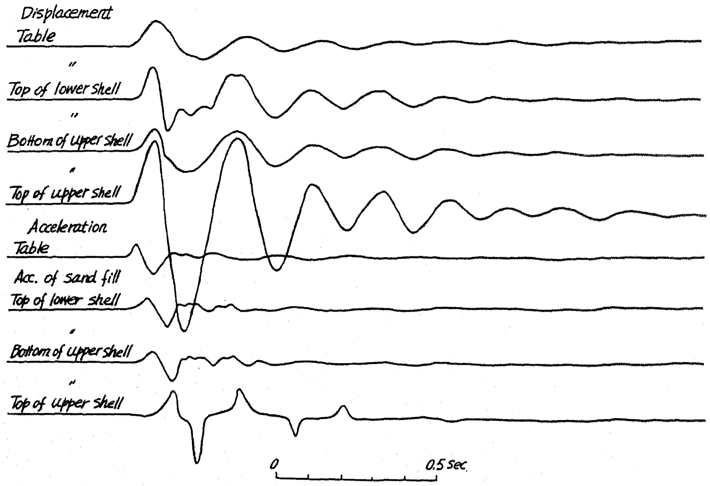
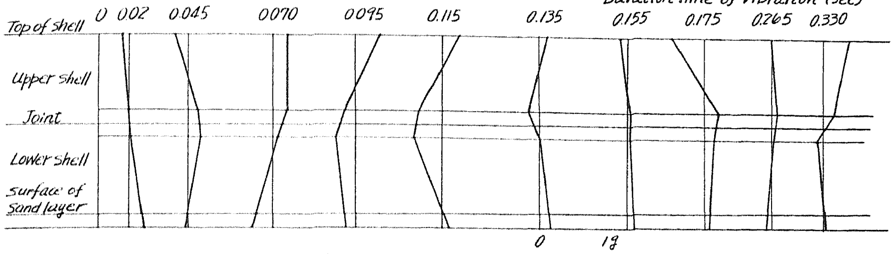


FIG. 3 EXAMPLES OF INSTRUMENTAL RECORDS OF TYPE-A MODEL, NO. C-1, OF THE GROUND ACCELERATION OF 0.26g.

a) Distribution of acceleration



b) Distribution of displacement

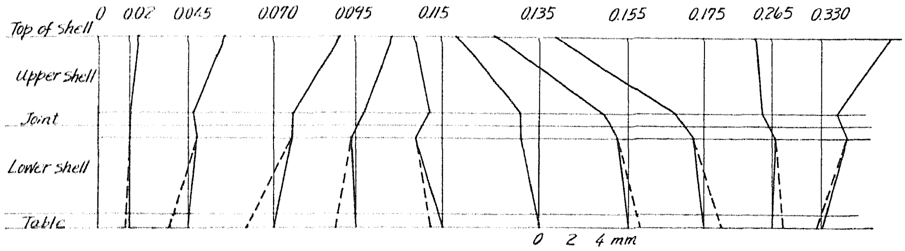


FIG. 4 EXAMPLE OF THE DISTRIBUTION OF ACCELERATION AND DEFORMATION OF THE SHELL (MODEL NO. C-1)

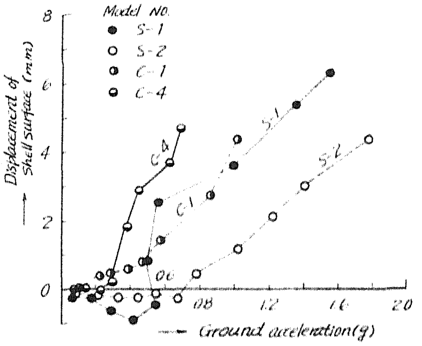


FIG. 5 RELATION BETWEEN THE TABLE ACCELERATION AND THE STATICAL DISPLACEMENT OF THE SHELL.

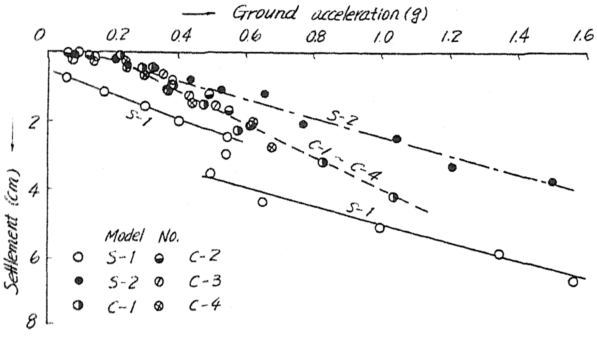
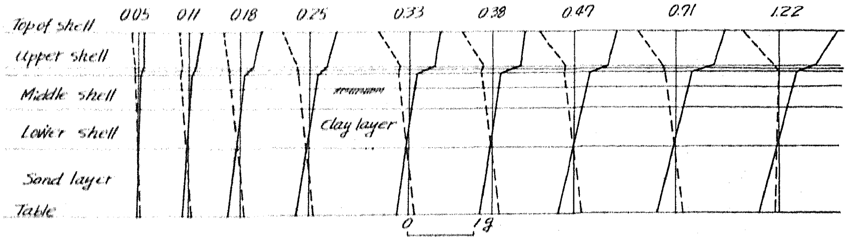


FIG. 6 RELATION BETWEEN THE TABLE ACCELERATION AND THE SETTLEMENT OF THE SAND FILL.

a) Shell



b) Embankment

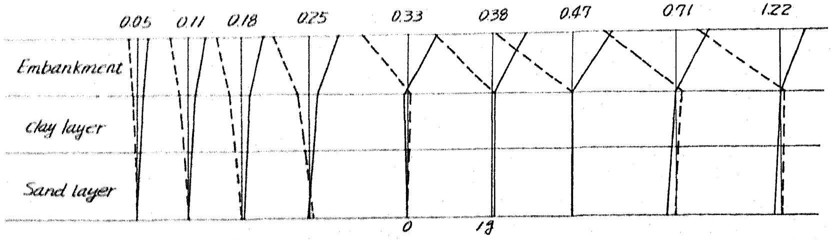


FIG. 7 EXAMPLE OF DISTRIBUTION OF ACCELERATION OF THE SHELL AND THE EMBANKMENT (MODEL NO. 4).

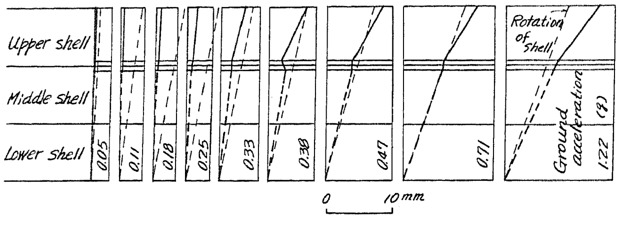


FIG. 8 STATICAL DEFORMATION OF THE SHELL.

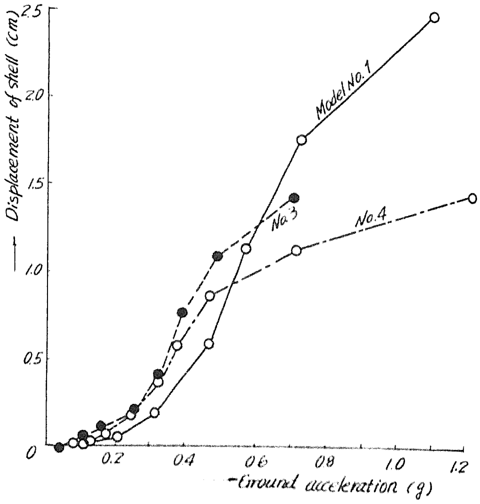


FIG. 9 RELATION BETWEEN THE TABLE ACCELERATION AND THE HORIZONTAL DISPLACEMENT OF SHELL.

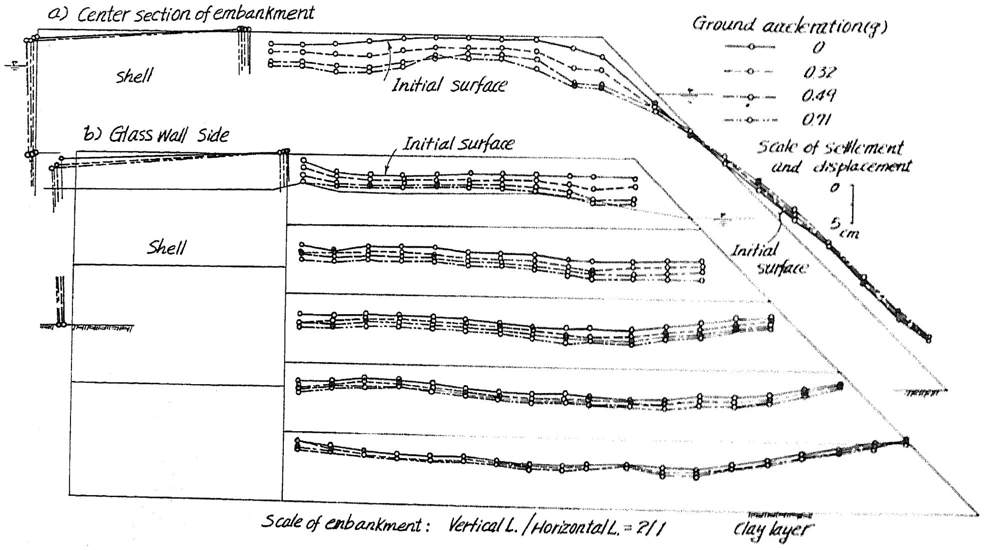


FIG. 10 DEFORMATION OF THE EMBANKMENT.

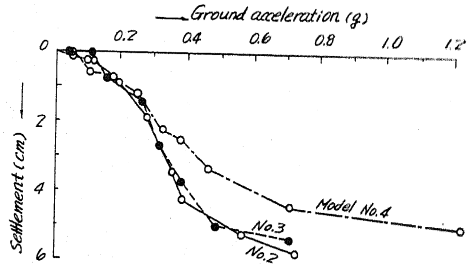


FIG. 11 RELATION BETWEEN THE SETTLEMENT OF THE EMBANKMENT AND THE TABLE ACCELERATION.

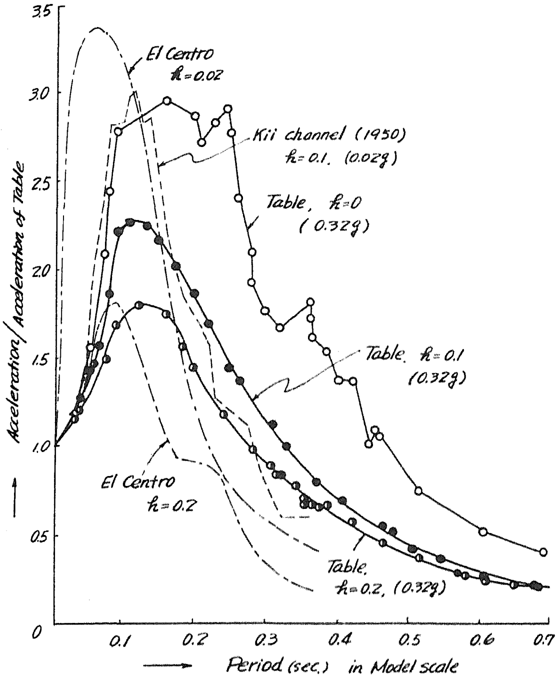


FIG. 12 COMPARISON AMONG THE ACCELERATION SPECTRA.

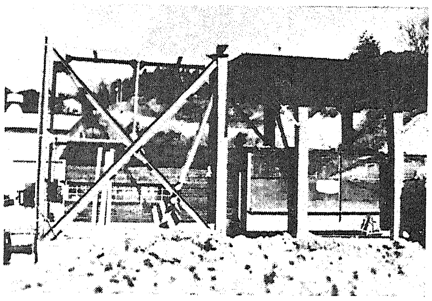


PHOTO-1 GENERAL VIEW OF THE SHAKING TANK.

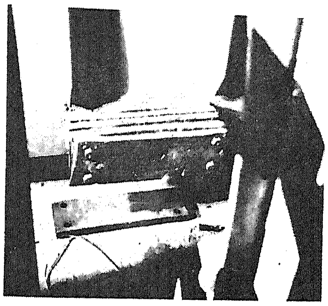


PHOTO-2 BUFFER SPRING OF THE SHAKING TANK.

Quantity	Required Ratio m. to p.	Required Value of Model		Actual Value of Model	
		Type-A	Type-B	Type-A	Type-B
Length	1 : S	1 : 15	1 : 30	1 : 15	1 : 30
Time	1 : √S	1 : 3.9	1 : 5.5		
Acceleration	1 : 1	1 : 1	1 : 1		
Weight of Soil (t/m³)	1 : 1	1.6~1.7	1.6~1.7	1.7	1.6~1.7
Weight of Clay (t/m³)	1 : 1	1.5	1.5	1.47	1.47
Angle of internal Friction of Soil (°)	1 : 1	35	35	35~36	35~36
Velocity of S-Wave (m/s)	1 : √S				
Soil	200	52	37	40~70	34~42
Clay	80	21	15	16~22	18~23
Cohesion of Clay (kg/cm²)	1 : S	0.02~ 0.07	0.008	0.012~ 0.016	0.007~ 0.009
Period of Ground Motion (sec)	1 : √S	0.11	0.13	0.10~0.14	0.11~0.14

TABLE-1 MECHANICAL REQUIREMENTS OF MODELS.

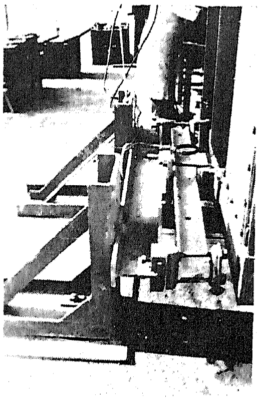


PHOTO-3 ANCHOR SPRING FOR THE SHAKING TANK

AN EXPERIMENTAL STUDY ON THE EARTHQUAKE RESISTANT PROPERTY OF BREAKWATERS OF THE CYLINDRICAL SHELL TYPE

BY M. HATANAKA

QUESTION BY: **A.J. WATT - UNITED KINGDOM**

What was the relative density of sand in the cylinder during tests on type A model? If this exceeded 85 per cent then liquefaction should not occur even under the high accelerations applied during the experiment.

Where a breakwater is constructed cannot the sand be compacted by deep vibrators (Vibroflotation) to prevent liquefaction?

AUTHOR'S REPLY: The relative density of the sand fill of the Type-A model was about 70% in my experiment and it was not dense enough.

I, also, think that the problem of liquefaction is one of the most important problems as you indicated. Since the compaction method of contraction was not adopted for the sand fill of the actual breakwater of this type, I proposed that the rubble stone should be used as the sand fill near the joint of the upper and the lower shells considering the experimental results as well as the contraction cost. However, I think that it may be better to compact the sand to a high density inside the shell by Vibroflotation as you proposed in addition to the use of the rubble stone.

QUESTION BY: **V.A. MURPHY - NEW ZEALAND**

How did you correlate the time scale with the linear scale?

AUTHOR'S REPLY: With the linear scale of $1:s$, the time scale of $1:\sqrt{s}$ should be taken naturally, as you indicated. So it is in my study.

The constants of the models in my study as well as the relation between the dimension of the prototype and that of the model are shown in Table 1 in my paper. I am, also, interested in the law of similarity which is very important. Would you kindly let me hear your comment on the following two problems?

1. What kind of scale do you take for the cohesive soil of $\phi \neq 0$ and $c \neq 0$?

I understand that the ϕ's scale $1:1$, for sand ($c=0$), and the c's scale, $1:s$, for clay ($\phi = 0$) can be taken.

2. What do you think of the law of similarity for the pore water pressure?

COMMENT BY: V.A. MURPHY - NEW ZEALAND

In my experiments, I have taken the same ratio of linear scale to time scale as you have, and my experience has been mainly with sand models. I, therefore, cannot comment on cohesive soils and have had no experience with pore water pressure where the law of similarity is applied.