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# Evaluation of Seismic Safety of a Large Caisson Structure

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# EVALUATION OF SEISMIC SAFETY OF A LARGE CAISSON STRUCTURE

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## ABSTRACT

Some centrifugal shaking tests were carried out to clarify the mechanism of seismic interaction between a large caisson foundation and soil layers. Based on the test data, two-dimensional seismic effective stress FE analysis was applied, followed by verifying its applicability. Also, to clarify the flexural and shear behavior of the caisson structure members to the ultimate state and to evaluate the flexural and shear resistance, some large-scale model failure tests of poorly reinforced concrete in caisson foundation were carried out. Based on these test results above-mentioned, seismic analyses of an existing large caisson foundation to ground motion in level 2 earthquakes were carried out, adopting the above-mentioned analysis. As the results, it was concluded that the seismic safety on the caisson foundation was confirmed.

## INTRODUCTION

Around the mouths of main rivers flowing into Osaka Bay are widespread lowland districts below the sea level, and many tide gates have been established to prevent high tide from flowing into these districts. Of these tide gates, the three mouths of rivers, such as Aji River, Kizu River and Shirinashi River, have arch-type tide gates, which are the largest among the same type in Japan. The three tide gates were constructed around the same time of thirty years ago with the same type of large caisson foundation. Especially, they have an important role for flood controls, and are required to have the seismic safety to level 2 earthquakes, such as the 1995 Kobe Earthquake. In this paper, the Aji River gate is taken up as an example for the deepest caisson foundation among the three gates, and the evaluation of seismic safety of the caisson structure to level 2 earthquakes is discussed.

## OUTLINE OF SOIL LAYERS AND STRUCTURES

Fig. 1 shows the soil profile at the site of the Aji River tide gate and its cross section. This site is located on the surface of reclaimed layer at the river mouth. The reclaimed layer (*B*), about 7m thick, overlies a Holocene sand layer (*As1*) and a

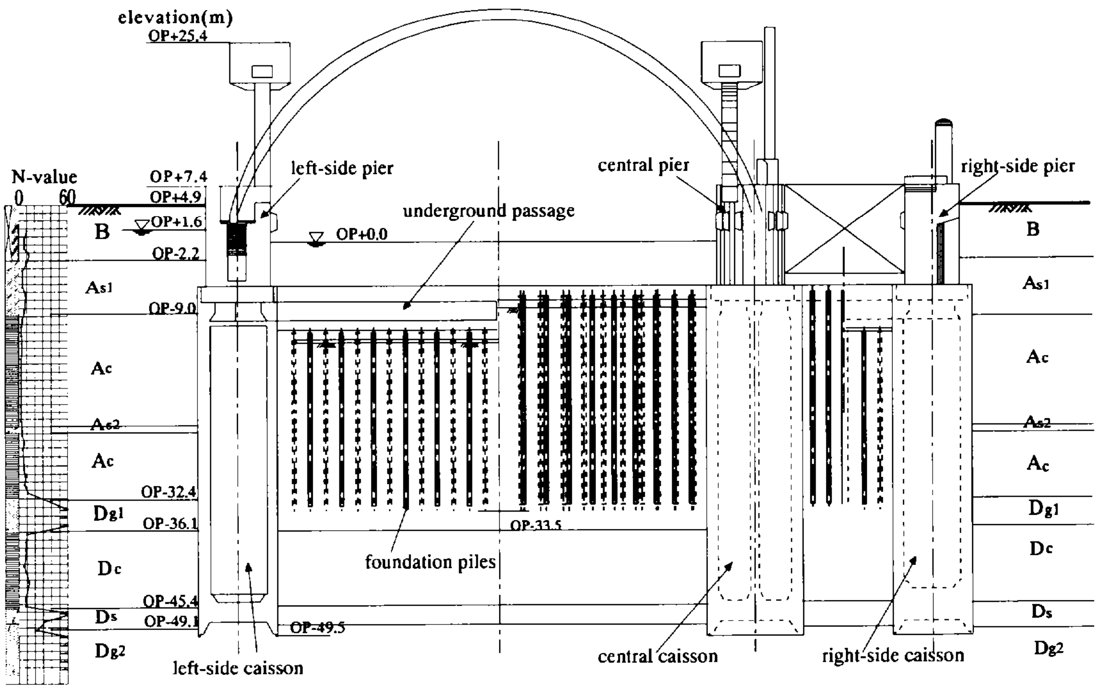


*Fig. 1. Soil profile at the site of the Aji River tide gate and its cross section*

Holocene clay layer (*Ac*), about 7m and 23m thick, respectively. *Ac* layer has a thin Holocene sand layer (*As2*). Below these layers lie Pleistocene gravel and sand layers (*Dg1*, *Ds* and *Dg2*) and a Pleistocene clay layer (*Dc*). *Dg2* layer lying below 54m in depth, which has over 60 in N-value is the bearing layer for caisson foundation.

Clay layers prominently occupy about 60% of layers above *Dg2* layer. Among the clay layers, *Ac* layer is normally consolidated with 3 to 6 in N-value. *Dc* layer is slightly over-consolidated with 8 to 12 in N-value. Ground water level in *B* layer is located at 3.3m in depth from the ground surface. Therefore, *B* layer under the water level and *As1* and *As2* layers should be considered as liquefiable layers.

The super-structure of the Aji River tide gate is an arch-type structure fixed with pin span of 66m long, 35m in radius, 13m high from the sea level when closing and 30m high when opening. The foundation is large caissons of 10m wide, 25m long and 43m deep executed by pneumatic type. Reinforced concrete of caisson foundation is very aged and has less amount of vertical reinforcements (0.3%).

From these characteristics of soil layers and foundation structure, it is required that the mechanism of seismic interaction between multi-clay layers and large caisson foundation. As well as the flexural and shear behavior of caisson structure members with poorly reinforced concrete should be elucidated in order to evaluate seismic safety of the Aji River tide gate to level 2 earthquakes.

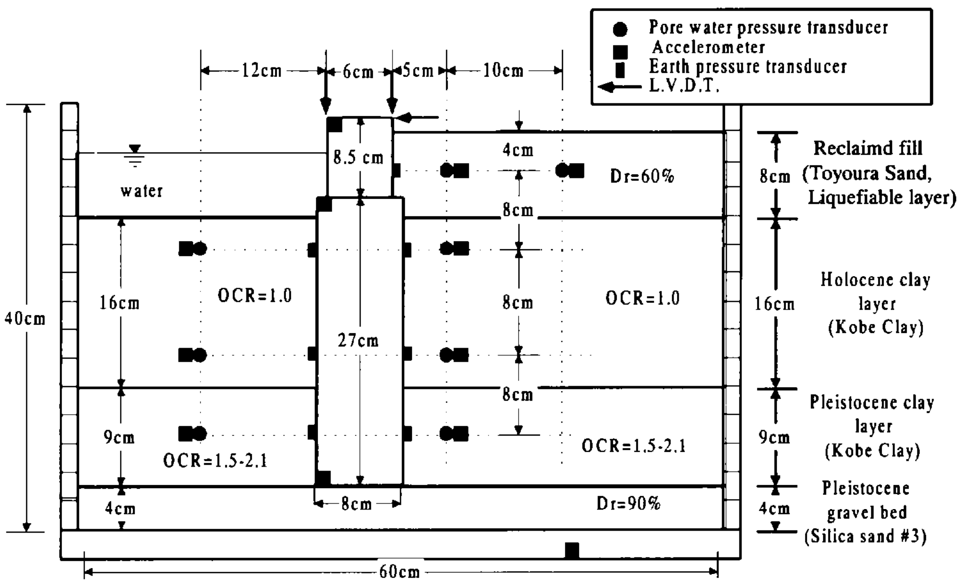


*Fig. 2. Model ground and caisson in centrifugal shaking tests and layout of monitoring sensors (Satoh et al. 2000)*

## SEISMIC INTERACTION BETWEEN SOIL LAYERS AND LARGE CAISSON FOUNDATION

### Method of Centrifugal Shaking Test

Fig. 2 shows the model ground and caisson in centrifugal shaking tests and the layout of monitoring sensors (Satoh et al. 2000). This model is approximated as a two-dimensional model on a scale of 1 to 75, assuming the half size of soil layers and caisson foundation shown in Fig.1 as the prototype. The test is carried out using a shear box of 60cm long, 30cm wide and 40cm high with 75g of centrifugal acceleration. Accelerometers and pore-water pressure, earth pressure and displacement transducers are set in the model to grasp the seismic interaction between soil layers and caisson foundation. Caisson model is made of aluminum, scaled by 1 to 75 of prototype rigidity EI, following the similarity rule. The test case is expressed as case1 (case1-1, case1-2, case1-3) and case2. Case1 is carried out three times to confirm reliability of the test. Case2 is adjusted to OCR=1.5~2.0 in Holocene clay behind the caisson, which strengthens the bearing capacity.

Fig. 3 shows the shaking wave used in the test. The irregular wave of maximum 200gal is set from input data of actual seismic wave in the bearing layer.

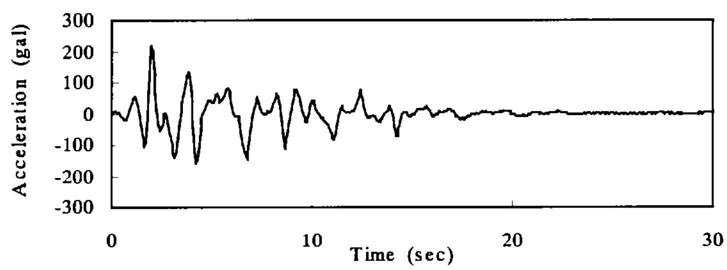


*Fig. 3. Shaking wave used in the test*

### Method of Analysis

Two-dimensional seismic effective stress FE analysis is applied to the results of the centrifugal shaking model tests. The FE meshes are made for the prototype, in which model ground and caissons are scaled by 75 times. In this analysis, Ramberg-Osgood model (Ramberg and Osgood, 1943) is applied for the non-linear relationship between stress and strain on each soil layer, and Bowl model (Fukutake et al. 1989) for the relationship between shearing strain and dilatancy. Caisson is treated as linear materials modeled by plane strain elements. Bottom boundary is set as fixed condition, and both side boundaries consist of semi-infinite free ground model through transfer elements. At the boundary between caisson foundation and the ground, joint elements are set to evaluate the sliding action. Table 1 shows ground parameters used in the analysis. These were estimated from the soil testing results of the material of model ground.

## Comparison of Test and Analytical Results

Fig. 4 shows measured and analytical acceleration responses of ground and caisson. At the ground in front of the caisson, both results are almost agreed showing less variation with depth. At the ground behind the caisson, both results in Holocene and Pleistocene clay layers show good correspondence. The shearing strain estimated from the acceleration response is approximated as 0.5-1.0%. In the reclaimed layer, however, the analytical results have a tendency of increase to the ground surface, which differs from the measured ones. It is also shown that in the reclaimed layer the measured and analytical excess pore-water pressure rates come to about 1.0 after 10 seconds shaking and reach the condition of liquefaction. Also, acceleration responses at the caisson have a good correspondence. Both measured and analytical displacement responses of caisson and earth pressures acting behind caisson are shown in Fig. 5. The measured earth pressure rises to the maximum value of 0.12N/mm$^2$ in the reclaimed layer, followed by confirming that liquefaction occurs. On the other hand, the measured displacement at the top of caisson has the maximum value of about 30cm, but the analytical value reaches a half of it, 15cm. This is because the sliding action on the bottom of caisson is observed in the test, while the joint element used in the analysis could not be sufficiently represented this phenomenon.
However, by centrifugal shaking tests, it was elucidated that seismic behavior of large-scale caisson foundation in the multi-layered clay ground with a liquefiable layer at the top. The analytical results applying the seismic effective stress FE analysis reasonably agreed with the measured ones. This supported to verify the applicability of the presented numerical simulation technique.

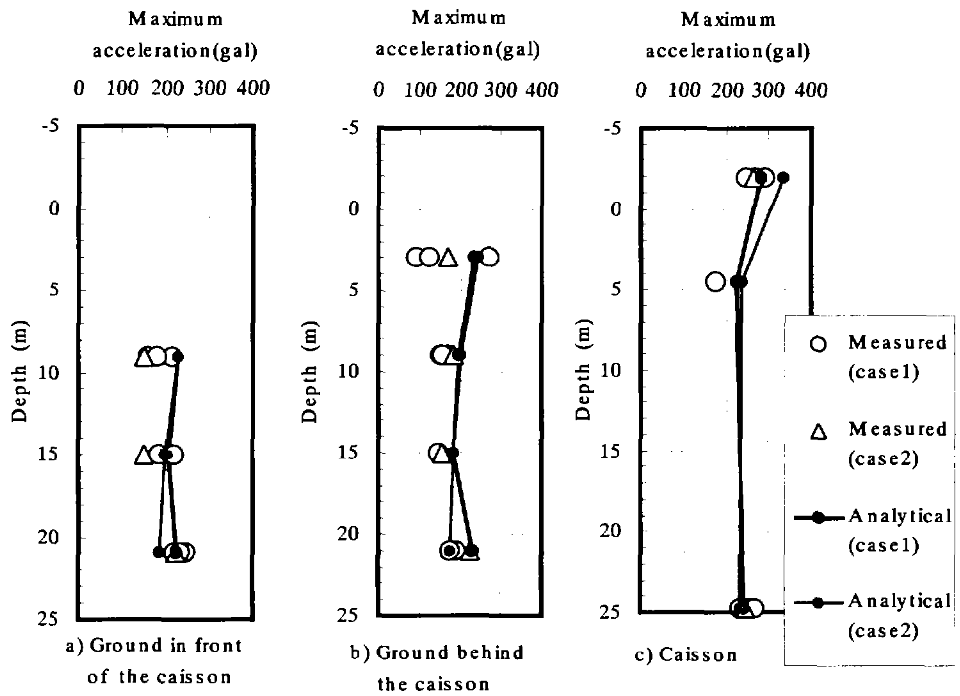


*Fig. 4. Measured and analytical acceleration responses of ground and caisson*

*Table 1. Ground parameters used in the analysis*

a) Soil paramters

| Layer | $\rho_t$ (kN/m$^3$) | Go (kN/m$^2$) | hmax (%) | $\gamma_{0.5}$ (%) |
|---|---|---|---|---|
| Liqufiable layer | 19.02 | 5.49E+05 | 23 | 0.028 |
| Holocene clay | 16.18 | 2.06E+05 | 20 | 0.045 |
| Pleistocene clay | 17.65 | 3.92E+05 | 20 | 0.038 |
| Pleistocene bed | 19.61 | 1.81E+06 | 20 | 0.016 |

b) Bowl model paramters

| Layer | A | B | C | D | Cs/(1+e$_0$) | $X_l$ |
|---|---|---|---|---|---|---|
| Liqufiable layer | -2 | 1.6 | 3 | 25 | 0.006 | 0.24 |
| Pleistocene bed | -5 | 1.6 | 9 | 45 | 0.005 | 0.29 |

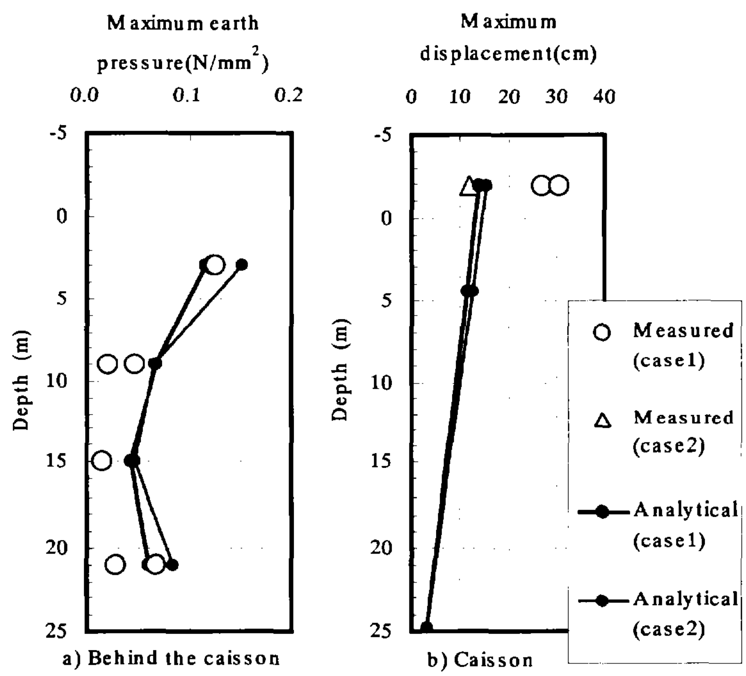


*Fig.5. Measured and analytical displacement responses of caisson and earth pressures acting behind caisson*

## EVALUATION OF RESISTANCE ON REINFORCED CONCRETE MEMBERS OF CAISSON

The reinforced concrete of the investigated caissons has less amount of vertical reinforcements and is composed of wall structures. On this type of caisson structure, the evaluation technique of the resistance of structure members has not been established yet. Therefore, to clarify their flexural and shear behavior to the ultimate state and to evaluate the flexural and shear resistance, some large-scale model failure tests of poorly reinforced concrete members in caisson foundation are carried out (Suzuki et al. 2000). The results of failure tests are shown as follows:

1) In the case of poorly reinforced concrete, in which the ultimate flexural moment is less than cracking moment, cracking distribution does not occur after the flexural cracking, so that the evaluation technique of the flexural resistance, which is different from conventional reinforced concrete, is required.

2) The flexural resistance rapidly decreases after cracking, however high deformation performance is shown until the ultimate point, which is defined by the breaking point of the vertical reinforcement.

3) It is possible to evaluate the flexural behavior in the tri-linear frame model by setting the local damage member to $12D$ ($D$ : diameter of reinforcing bar).

4) From the monitoring of reinforcing strain and cracking condition, the shear force is resisted by concrete until the cracking point, after that, by tie reinforcements following the progress of concrete cracking.

5) The shear resistance can be mostly evaluated by using evaluation formula of corbel members, considering the effect of shear span, which is proposed by Niwa et al. (1983).

## EVALUATION OF SEISMIC SAFETY ON THE AJI RIVER TIDE GATE

### Method of Analysis

Two-dimensional seismic effective stress FE analysis, of which the applicability was verified based on the results of the centrifugal shaking test previously mentioned, is applied to evaluate seismic safety of the Aji River tide gate. Both sides of two-dimensional FE meshes of ground and structures consist of semi-infinite free ground model through transfer element, and the bottom is set as viscous boundary. To evaluate the sliding action, joint elements are introduced between the ground and the caisson. The reclaimed layer ($B$) and Holocene sand layers ($As1$ and $As2$) are set as liquefiable layers in the analysis.

Table 2 shows soil parameters used in the analysis, which are results of ground investigations and dynamic tests. Fig. 6 shows the fitting result after applying Ramberg-Osgood model to dynamic test data, and Fig. 7 shows the fitting result after applying Bowl model to liquefaction test.

Caisson foundation is modeled as tri-linear beam elements, in which the ultimate curvature $\phi_u$ (=0.018) is defined by a point

*Table 2. Soil parameters used in the analysis*

a) Soil paramters

| Layer | $\rho_t$ (kN/m$^3$) | Go (kN/m$^2$) | $h_{max}$ (%) | $\gamma_{0.5}$ (%) |
|---|---|---|---|---|
| B | 19.61 | 6.47E+05 | 25 | 0.018 |
| Asl(upper) | 19.12 | 6.33E+05 | 25 | 0.029 |
| Asl(lower) | 17.65 | 5.84E+05 | 26 | 0.172 |
| Ac(upper) | 16.18 | 3.24E+05 | 26 | 0.282 |
| As2 | 18.63 | 6.87E+05 | 25 | 0.079 |
| Ac(lower) | 16.67 | 6.13E+05 | 25 | 0.285 |
| Dg1 | 19.61 | 1.65E+06 | 23 | 0.028 |
| Dc | 15.69 | 7.06E+05 | 24 | 0.493 |
| Ds | 18.63 | 1.60E+06 | 27 | 0.418 |
| Dg2 | 19.61 | 4.24E+06 | 23 | 0.028 |

b) Bowl model paramters

| Layer | A | B | C | D | Cs/(1+e$_0$) | Xl |
|---|---|---|---|---|---|---|
| B | -5 | 1.6 | 7 | 35 | 0.006 | 0.20 |
| Asl(upper) | -5 | 1.6 | 8 | 35 | 0.006 | 0.24 |
| Asl(lower) | -3 | 1.6 | 4 | 30 | 0.006 | 0.20 |
| As2 | -5 | 1.6 | 8 | 35 | 0.006 | 0.25 |

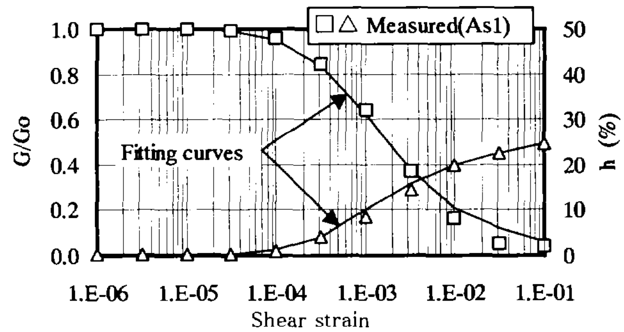


*Fig. 6. Fitting result after applying Ramberg-Osgood model to dynamic test data*

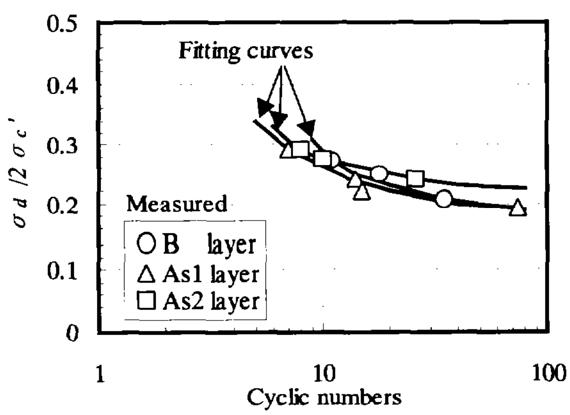


*Fig. 7 Fitting result after applying Bowl model to liquefaction test data*

in which reinforcement reaches breaking strain (=18%), by calculating bending moment-curvature relationship. Tri-linear beam elements are applied to the concentrated damaged parts of caisson, and linear beam elements are to the other parts. The concentrated damaged area is set as $12D$, based on the model failure tests of reinforced concrete members previously mentioned. Fig. 8 shows $M$- $\phi$ relationship of the left-side caisson applied the FE analysis. The pier of upper caisson is modeled by plane strain elements, and the tide gate arch by linear frame elements.
Fig. 9 shows input data of seismic wave profile, which is the incident wave to bedrock surface converted from the ground motion in level 2 earthquakes of the Osaka area.

## Analytical Results

Fig. 10 shows the distribution of excess pore-water pressure ratio. $B$ and $As1$ layers are almost liquefied with over 0.9 excess pore-water pressure ratio.
Fig. 11 shows the time history of pore-water pressure ratio at point $A$ in Fig. 10. It begins to rise in 2 seconds and becomes to almost 1.0 in 7 seconds reaching liquefaction. Fig. 11 also shows the time history of displacements at points $B$ and $C$ in Fig. 10. The maximum horizontal displacements at points $B$ and $C$ are 30 cm, the maximum relative displacement between the two points is 4.4 cm, which occurs at around the time to reach liquefaction. This behavior might cause slightly damage to the structure at the joint sections between underground passage and caisson.
Fig. 12 shows the hysteresis curve of $M$- $\phi$ relationship of non-linear members. The maximum response curvature is about 0.01, having sufficient allowance to the ultimate curvature $\phi_u$.
Fig. 13 shows the distribution of flexural moment of linear members. The maximum of the flexural moment is less than the cracking moment. Fig. 14 shows the distribution of shear force on caisson. Shear strength is less than shear forces, however, by adding the shear resistance of the tie reinforcements, sufficient shear strength is guaranteed.
From these results, it is concluded that the caisson foundation has enough seismic safety to the flexural and shearing resistance, so that large-scale reinforcement measures are not necessary.

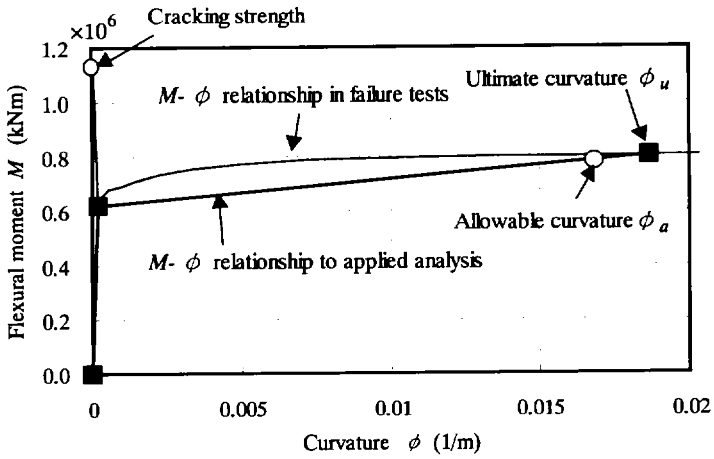


*Fig. 8. M- $\phi$ relationship applied to the analysis*

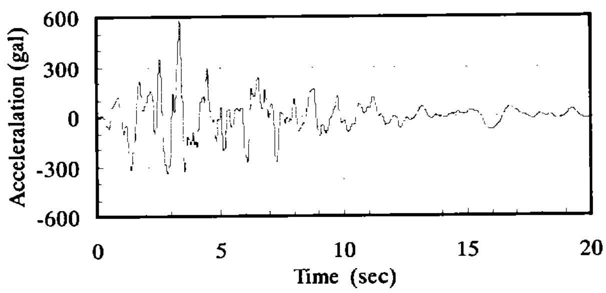


*Fig. 9. Input data of seismic wave profile*

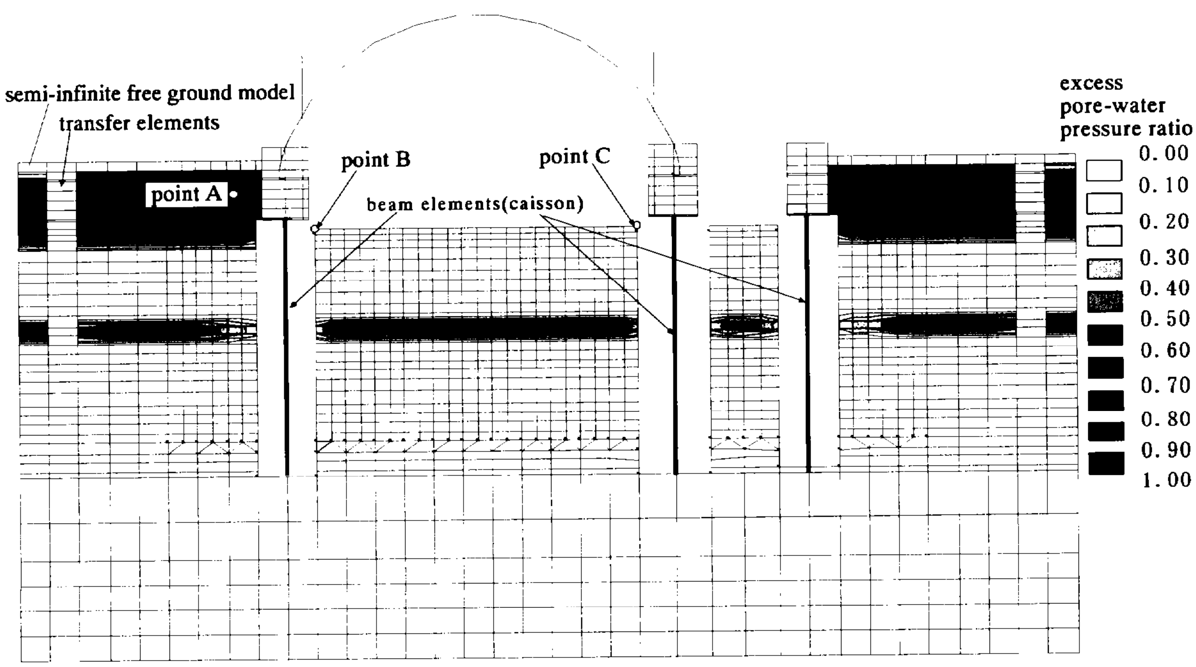


*Fig. 10. Distribution of excess pore-water pressure ratio*

## CONCLUSIONS

Main conclusion in this study are summarized as follows:

1) By centrifugal shaking tests, it was elucidated that seismic behavior of large-scale caisson foundation in the multi-layered clay ground with a liquefiable layer at the top. The analytical results applying the seismic effective stress FE analysis reasonably agreed with the measured ones. This supported to verify the applicability of the presented analysis.
2) The presented analysis was applied to evaluate seismic safety of the Aji River tide gate. Tri-linear beam elements were applied to the concentrated damaged parts of caisson, based on the model failure tests of reinforced concrete members. As the results, the maximum of the flexural moment was less than the cracking one. And the shear strength was sufficiently guaranteed by adding the shear resistance of tie reinforcements. But the relative displacement of two caissons might be caused slightly damage to the structure at the joint sections between under-ground passage and caisson.

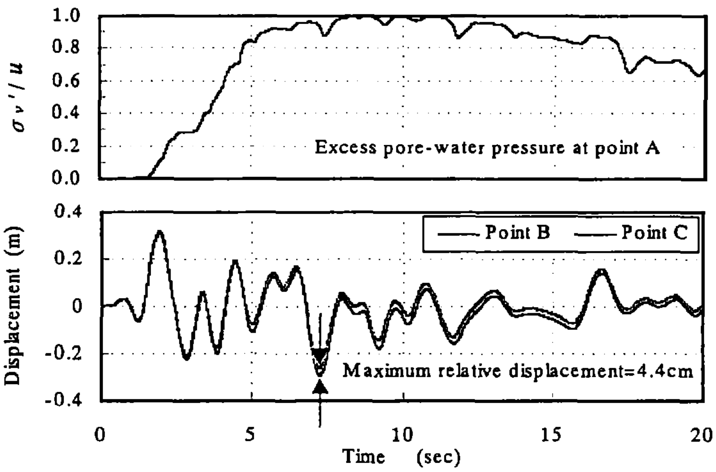


*Fig.11 Time history of pore-water pressure ratio at point A and displacements at points B and C*

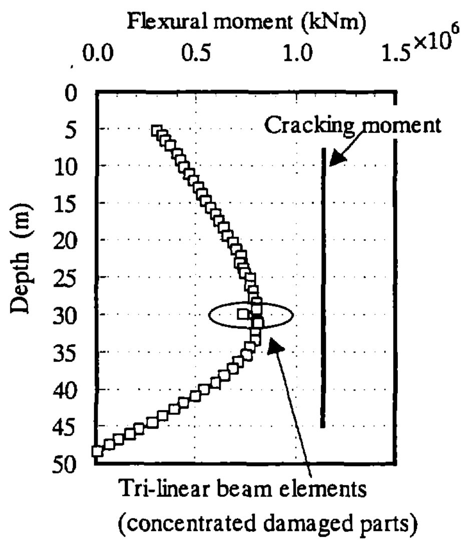


*Fig. 13. Distribution of flexural moment of members*

3) From these results, it was concluded that the caisson foundation had enough seismic safety to the flexural and shearing resistance, so that large-scale reinforcement measures were not necessary.

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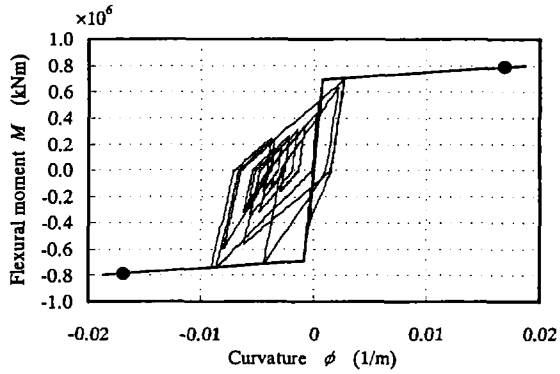


*Fig. 12 Hysteresis curve of M- φ relationship*

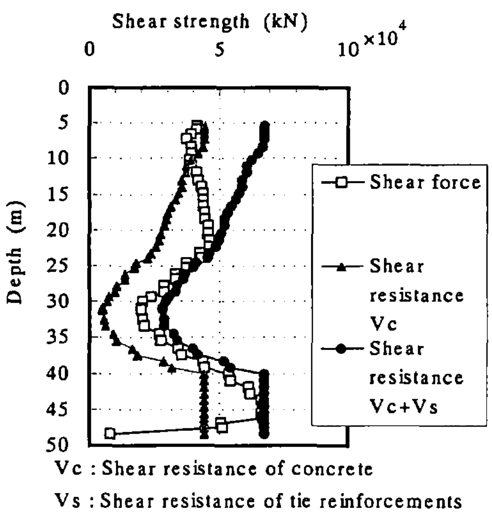


*Fig. 14. Distribution of shear force on caisson*