13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 805

SEISMIC PERFORMANCE OF A SEPARATED SELECTIVE-INTAKE TOWER AGAINST LEVEL 2 SEISMIC MOTIONS

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SUMMARY

A separated selective-intake tower is an asymmetrical slender reinforced concrete structure with multistage steel intake gates so that the behavior of the structure is not simple during an earthquake. Analytical studies using a three-dimensional finite element model were carried out to investigate the seismic performance of an existing intake tower against level 2 seismic motions defined in the Japanese seismic design standards. In order to make an adequate analysis model, a 3-D FEM model was calibrated by comparing observed seismic records of the existing intake tower. This study yielded useful information for design, especially in terms of a modeling method and definitions of limit states for separated selective-intake towers.

INTRODUCTION

A separated selective-intake tower is an asymmetrical slender structure constructed in a dam reservoir. Because of their slenderness, separated selective-intake towers show complex behavior during earthquakes, partly because of the influence of water. In Japan, there has been no structural damage caused by strong earthquake motions like "level 2" earthquake motions defined in the Japanese seismic design standards. This, however, is simply because stand-alone intake towers in Japan happened to escape the damaging effects of large seismic loads, and it is not that those structures are highly resistant to earthquakes. Until recently, the standard seismic design practice in designing separated selective-intake towers in Japan was to use a seismic calculation procedure based on the conventional seismic coefficient method for "level 1" earthquake motions (which are less strong than "level 2" earthquake motions), and dynamic analyses were conducted on an as-needed basis.

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In the wake of the Hyogo-ken Nanbu Earthquake of 1995, which seriously damaged many civil engineering and other structures, various design standards in Japan began to use a two-stage design approach, and it has now become the standard practice to consider both level 1 and level 2 earthquake motions in performing seismic design. As a result, it is now becoming necessary to follow this two-stage seismic design procedure in designing separated selective-intake towers, as in the case of other types of civil engineering structures. Because separated selective-intake towers perform an important function as water utilization facilities, malfunctioning of those intake towers caused by earthquakes would have serious social impact.

In view of the fact that "level 2" earthquakes caused by seismic faults are predicted in the near future in Japan and of unpredictability of the locations of such earthquakes, it is important to establish a seismic design method for separated selective-intake towers as soon as possible.

In this study, the authors conducted a series of studies based on numerical analysis of an existing representative intake tower in order to verify seismic performance of separated selective-intake towers for level 2 earthquake motions. This paper proposes calculation methods, modeling and a seismic design concept as practical measures to be taken at the design stage and presents an approach by which to satisfy the seismic performance requirements for the type of structures mentioned above. In considering calculation and modeling methods, effort was made to enhance accuracy by reflecting the modes of vibration and response characteristics obtained from earthquake observation records and micromotion measurement results.

DATA ON THE INTAKE TOWER

The intake tower considered in the present study is an intake tower of an existing dam (hereafter referred to as "A Dam") which passed for fifteen years since completed. Figure 1 shows the structure of the tower, and Figure 2 shows its typical cross section.

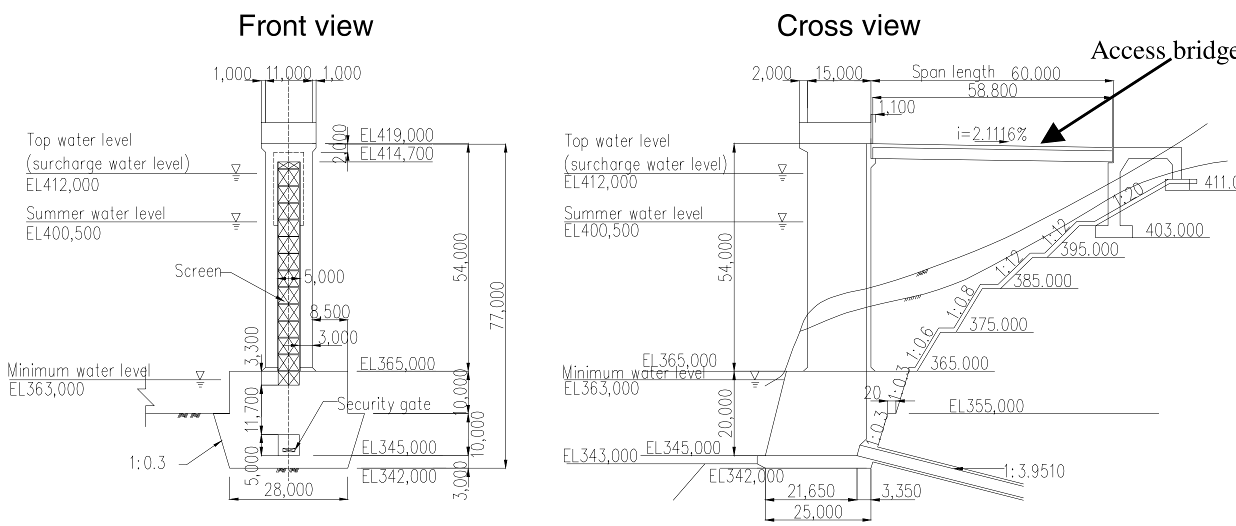


Figure 1 The A Dam intake tower

Built as a reinforced concrete structure, the intake tower is surrounded on three sides by walls, and the water inlets are located on the open side. Because of this configuration, the intake tower is likely to behave in a complex manner when subjected to strong lateral loads due to earthquake motions. The A

Dam intake tower is 82 m high (columnar section: 54 m, foundation section: 28 m). The long and short sides of the cross section are 15 m and 11 m long, respectively, and the wall thickness is 2.0 m to 3.0 m. The largest-diameter reinforcing bars used are D41 bars used as main reinforcement and D22 used as hoop reinforcement. Table 1 shows the material properties used for the design of the intake tower. At the time the intake tower was designed, the concept of level 2 earthquake motions did not exist. It was therefore standard practice to use a modified seismic coefficient method in which level 1 earthquake motions were used as design earthquake loads, and a seismic coefficient ranging from 0.13g to 0.3g (larger values for higher parts of a tower) was assigned according to the design seismic coefficient. The allowable stress method of structural calculation, therefore, was used as the criterion for seismic safety verification. The seismic performance of the intake tower was also checked through a dynamic analysis using three-dimensional shell models, and the rebar arrangement determined by the modified seismic coefficient method was strengthened accordingly. The input ground motions used in the dynamic analysis, however, had relatively low levels of acceleration ranging from about 200 cm/s^2 to 300 cm/s^2.

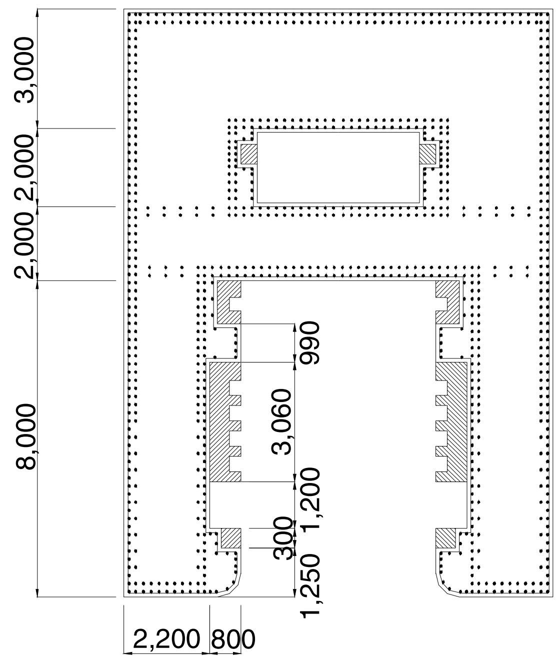


Figure 2 Reinforced concrete cross section of the intake tower

Table 1 Materials used for the intake tower (design values)

Unit weight of concrete	23.5kN/m^3
Young's modulus	26,500N/mm^2
Poisson's ratio	1/6
Reinforcing bars	SD295

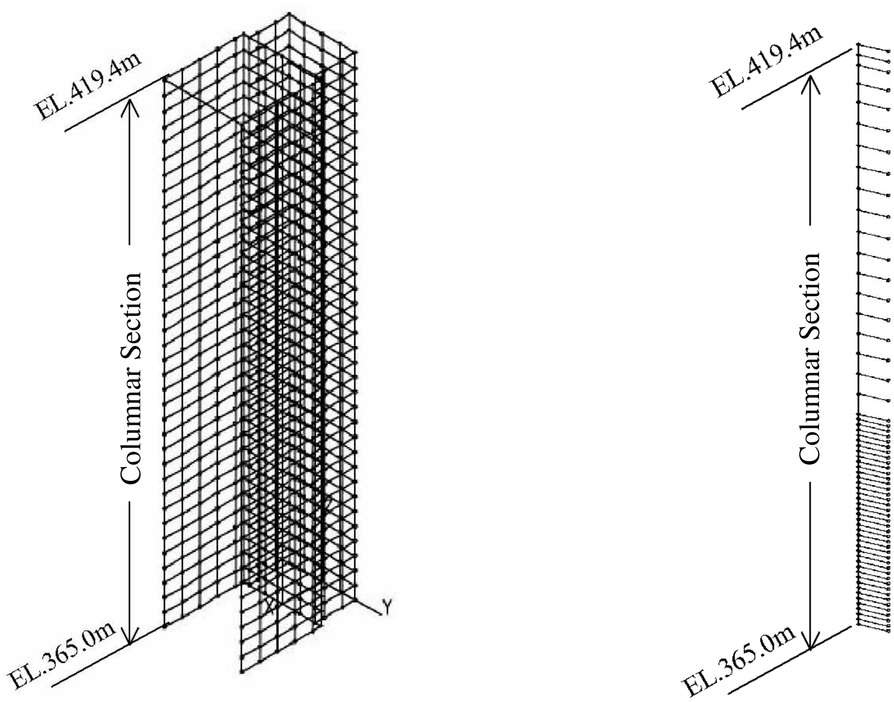
MODELING AND NATURAL VIBRATION CHARACTERISTICS

In modeling the intake tower structure, the following requirements were taken into consideration:

(1) To model the geometry of the intake tower as faithfully as possible
(2) To take into account only the weights of accessory equipment, superstructure facilities and the access bridge and add them as a concentrated load to the top of the tower
(3) To distribute the weights of the intake gates, intake gate stops and screens uniformly in the direction along the height of the intake tower because the influence of those components on the structural behavior during earthquakes is considered to be small
(4) To appropriately take into account the dynamic influence of water inside and outside the structure

To satisfy the first requirement listed above, a three-dimensional FEM model using shell elements was constructed to satisfy the second to fourth requirements listed above. Since the intake tower under consideration is a reinforced concrete structure, it was thought that its response to level 2 earthquake motions could enter the nonlinear range. In cases where the intake tower model responds nonlinearly after yielding, it is necessary to perform appropriate evaluation paying attention to analytical accuracy in seismic performance verification because shell elements are based on linear response calculation. For design convenience, three-dimensional frame analysis using beam element models, which is a widely used method of analysis, is a simple alternative that can be applied to both linear and nonlinear response. So, this method was compared with the method of using three-dimensional FEM shell models.

Figure 3 shows a three-dimensional FEM shell model and a three-dimensional beam model. The shell model has shell elements at the center of the cross section of each wall and the beam model has beam elements at the center of rigidity to add weight at the centroid. The elements in the bottom region of the beam model are finely separated for the purpose of nonlinear stiffness evaluation. Eigenvalue analyses were conducted using these models, and the natural vibration characteristics indicated by the two models were compared with measured values. The measured values used were natural vibration characteristics calculated from response values recorded during earthquakes by the seismographs installed on the intake tower. Table 2 compares the values obtained from the two models with measured natural frequencies. The table shows a number of characteristics of the different models:



(1) 3-D FEM shell model (2) 3-D beam model

Figure 3 Analysis model

- Both the FEM model and the beam model give effective mass ratios in the first mode of 50% to 60%, indicating that the first mode is the principal mode. The effective mass ratios, however, indicate that other modes, too, could affect the response of the tower.
- The FEM model underestimated vibration frequency both in the direction of water flow and in the perpendicular direction. Differences were particularly large in the water flow direction.
- Calculated natural frequencies were made closer to the measured values by adding stiffness of corners resulting from wall connection.

• Natural frequencies indicated by the beam model were higher than those indicated by the FEM model. The models taking hydrodynamic pressure into account gave values closer to the measured values.
• In both models, the addition of hydrodynamic pressure resulted in longer periods of vibration.

Table 2 Comparison of natural frequencies

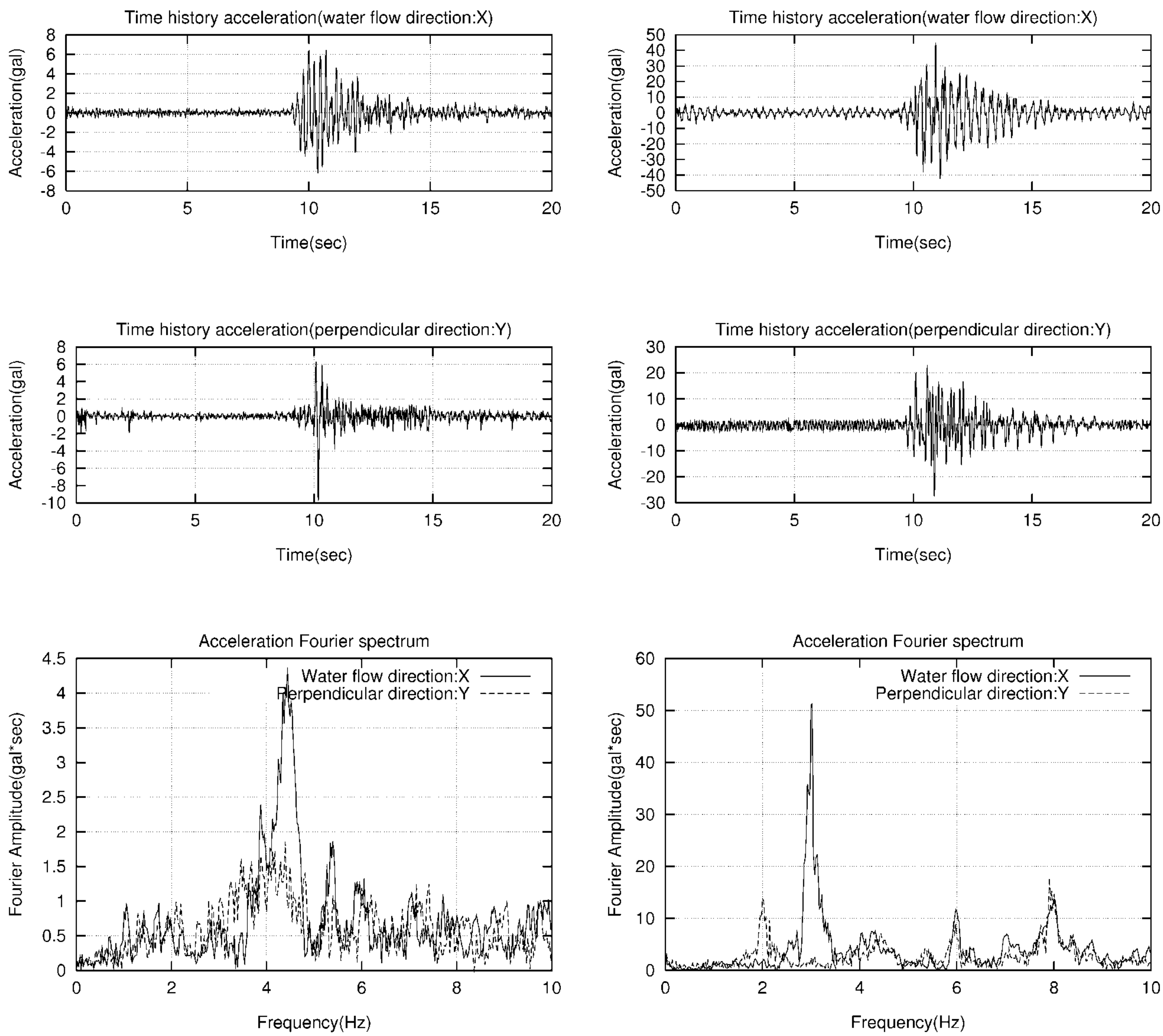
Case	Description of model	First-mode natural frequency (Hz)		Effective mass ratio (%)	
		Water flow direction	Perpendicular direction	Water flow direction	Perpendicular direction
Measured	Strong-motion seismograph record (July 30, 1992)	3.00	2.00		
FEM-1	Basic model (shell model)	2.36	1.95	62.8	63.2
FEM-2	FEM-1 allowing for corner stiffness	2.54	1.99	63.1	63.2
FEM-3	FEM-2 + hydrodynamic pressure (Goto–Toki equation)	2.47	1.80	57.7	57.6
Beam-1	Basic model (beam model)	3.32	2.09	61.9	62.1
Beam-2	Beam-1 allowing for hydrodynamic pressure (Goto–Toki equation)	3.19	1.95	53.9	51.7
Design	Shell model used at the time of design (hydrodynamic pressure in the Westergaard equation)	2.11	1.58		

For the hydrodynamic models, comparison was made through eigenvalue analysis using both the Goto−Toki equation [1] and the Westergaard equation [2] taking seismic loading into account. The Westergaard equation tended to overestimate added masses so that natural frequency was underestimated and deviations from the measured values increased. It was decided, therefore, that the Goto–Toki equation was more applicable to the problems associated with the particular structure under consideration. Comparison of the eigenvalue analysis results obtained from the FEM model and the beam model described above revealed a number of key considerations in modeling:

1) Common practice of seismic design is to model only main structural members and take into account only the weights of other members. If, however, real phenomena are to be simulated at the micromotion level, it is necessary to evaluate the stiffness of members other than the main structural members.
2) Possible causes of the poor accuracy of the FEM model include the stiffness of intake gate stops ignored in modeling, the effect of the access bridge and presumably low Young’s modulus used as the design value (real Young’ modulus may be higher than design used).
3) The beam model is based on Navier's hypothesis, so the model tended to indicate high stiffness and give higher frequencies than the FEM model. The reason why higher frequencies are indicated in the direction of water flow than the measured values is thought to be that the modeling was based mainly on bending behavior although the geometry of the tower makes it prone to shear behavior (high flexural stiffness).
4) For the same reason, the FEM model and the beam model show relatively good agreement with respect to behavior in the perpendicular direction in which the tower is more apt to undergo bending vibration.

COMPARISON WITH EARTHQUAKE OBSERVATION RECORDS

The validity of the FEM model and the beam model was evaluated by comparing the results of simulations performed using the two models with the seismic responses recorded by strong-motion seismographs installed on the intake tower. Earthquake motions were recorded simultaneously at three locations (the bottom, intermediate point and top of the tower), so acceleration waves recorded at the bottom of the tower were used, and three components (water flow direction, perpendicular direction, vertical direction) were input simultaneously. Figure 4 shows seismic acceleration waves in the water flow direction and the perpendicular direction recorded at the different locations on the tower, along with their Fourier spectra. The recorded earthquake motions, however, were relatively small as the response of a structure, and it was possible that the recorded motions included a certain amount of noise.



Bottom of intake tower　　　　Top of intake tower

Figure 4 Acceleration response (measured values) of different parts of the intake tower obtained from earthquake observation

In the direction of water flow at the bottom of the intake tower, a dominant frequency of about 4.5 Hz was particularly conspicuous, but in the perpendicular direction there was no such clear peak. At the intermediate point and the top of the tower, however, there were peaks at 3.0 Hz in the water flow direction and 2.0 Hz and 8.0 Hz in the perpendicular direction. This indicates that the columnar section and the bottom of the tower have different vibration characteristics and they are not closely related. Since the eartquake motions observed at the bottom of the tower could reflect the effects of ground and of the unusual shape of the foundation, and it cannot be denied that the seismic motions obtained are not clear enough for use as input motions in the analysis. The damping models used in the analysis for the FEM model and the beam model were Rayleigh-type models, and the material damping factor of concrete was assumed to be 0.05, which is usually used as a standard value in the linear response range. The dynamic analysis conducted under these conditions considered the case in which hydrodynamic pressure was ignored and the case in which it is taken into consideration, and the response values thus obtained were compared with the measured values. Table 3 compares these values. The results shown in the table indicate the following:

(1) On the whole, calculated response values tended to be smaller than observed values.
(2) In the cases in which hydrodynamic pressure was taken into consideration, both models indicated greater response values so that differences from the observed values became smaller.
(3) The two models (FEM and beam) showed close agreement with respect to vertical acceleration.
(4) On the whole, the FEM model gave response values closer to the observed values than the beam model did. (the case of FEM-3 shows the closest results to the observed values.)

Table 3 Comparison between earthquake response analysis results and measured values

Case	Description of model	Tower top acceleration (cm/s^2)			Mid-height acceleration (cm/s^2)		
		direction			direction		
		Water flow	Perpendicular	Vertical	Water flow	Perpendicular	Vertical
Strong-motion observation	Strong-motion seismograph record (July 30, 1992)	45.0	27.5	8.6	18.4	17.1	7.0
FEM-1	Basic model (shell model)	19.7	8.8	8.6	10.2	10.6	5.3
FEM-3	Hydrodynamic pressure (Goto–Toki equation) taken into account	26.1	23.0	7.9	13.0	13.0	5.6
Beam-1	Basic model (beam model)	17.9	13.1	7.8	4.3	7.4	6.5
Beam-2	Hydrodynamic pressure (Goto–Toki equation) taken into account	17.4	20.3	7.8	4.4	9.6	6.5

Concerning the first finding listed above, because the analysis model treated the intake tower foundation as being fixed, the assumptions including the input acceleration waves could differ from the actual conditions. In order to reduce differences between observed values and the analysis model as mentioned in the second finding listed above, it is necessary to take hydrodynamic pressure into consideration. Concerning the fourth finding, since the beam model is based on Navier's hypothesis, accuracy could become lower in cases where two-dimensionality and a torsional mode of vibration combine to create a complex mode of vibration as in the case of the intake tower under consideration. Figure 5 shows acceleration response waves and their Fourier spectra (only representative examples) obtained from the FEM-3 model that takes into consideration corner stiffness and the hydrodynamic pressure expressed by the Goto–Toki equation. The observed values and the analysis model show general agreement in terms of the tendency of acceleration amplification, indicating validity of qualitative modeling. Especially in the perpendicular direction, the observed dominant frequency of 2.0 Hz was simulated by the FEM model with sufficient accuracy. In the direction of water flow, however, the observed dominant frequency of 3.0 Hz was not reproduced by the FEM model, and a conspicuous peak was indicated at around 4.4 Hz. This is thought to be the cause of the poor accuracy.

The most likely reason for the poor accuracy mentioned above is that the dominant frequency (around 4.4 Hz) of the input ground motion coincided with the third mode of the FEM model, and resonance resulted. Since there are factors that make analytical modeling in the water flow direction difficult such as the effects of the access bridge, foundation configuration and actual stiffness, it is thought that these factors should be taken into account in order to achieve a higher level of accuracy.

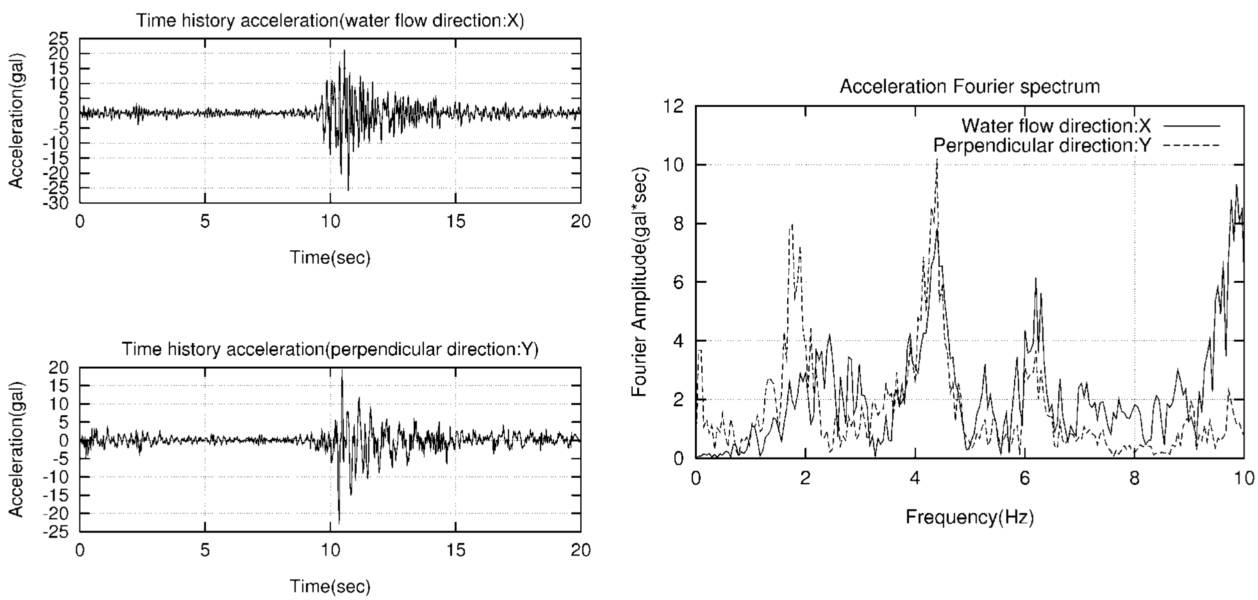


Figure 5 Acceleration response of the intake tower determined through FEM model analysis

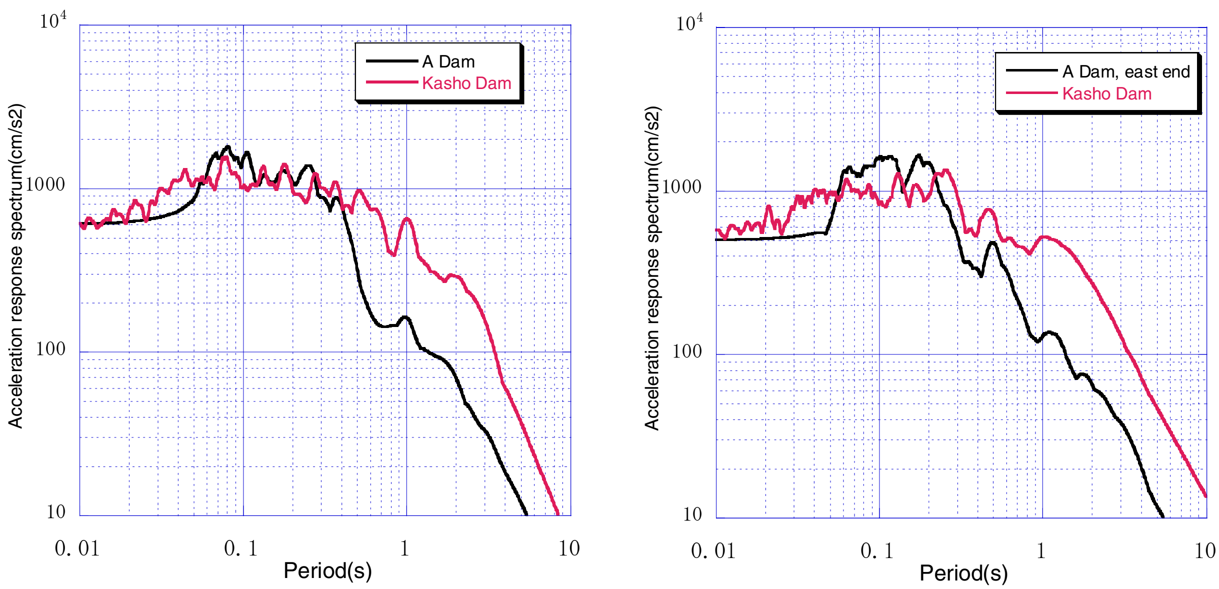
EVALUATION OF SEISMIC PERFORMANCE AGAINST LEVEL 2 EARTHQUAKE MOTIONS

The seismic performance of the A Dam intake tower against level 2 earthquake motions was evaluated using a three-dimensional FEM shell model that was considered to be valid from the engineering point of view. Level 2 earthquake motions to be used for the seismic design of dam-related facilities in Japan have not been indicated in concrete form. The strongest ground motion ever recorded for dam-related facilities and their surrounding is the ground motion (real earthquake motion) recorded at Kasho Dam during the Tottori-ken Seibu Earthquake of October 6, 2000. In recent years, it has become common practice to generate simulated earthquake motion from an assumed fault layer to the location of the facility under consideration. The A Dam simulated seismic wave (artificial seismic wave based on fault estimation), therefore, was developed as level 2 earthquake motion applicable only to the intake tower under consideration. Then, a dynamic analysis of the intake tower was conducted, using these two seismic waves and the three-dimensional FEM shell model, to evaluate the seismic performance of the intake tower.

Table 4 shows the analysis conditions such as the analysis model and method used. Figure 6 compares acceleration response spectra of the two input seismic waves in two horizontal directions. As

Table 4 Analysis conditions

Item	Description	Remarks
Analysis model	Three-dimensional FEM shell model	
Analysis method	Linear analysis by the direct integration method (Newmark- β method; time step: 0.01 s)	
Damping model	Rayleigh damping	
Damping factor	Concrete materials: 10% [3]	
Input earthquake waveform	Kasho Dam observed wave (max. 528 gal), A Dam simulated wave (max. 604 gal)	



Water flow direction **Perpendicular direction**

Figure 6 Comparison of acceleration response spectra of input earthquake waves

shown, the response spectrum of the Kasho Dam observed seismic wave for periods longer than about 0.3 sec shows larger values than the response spectrum of the A Dam simulated seismic wave, indicating that the Kasho Dam observed seismic wave is more influential on the response characteristics of the intake tower.

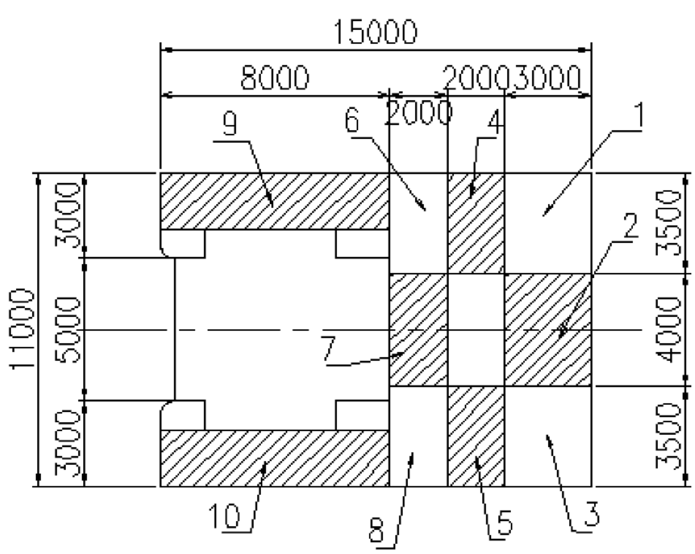
In evaluating the analytical results, attention was given to bending moment and shear force, selected from section forces. Bending moment is verified by checking seismic performance on the basis of the relationship between the bending moment occurring in the member (M_D) and the first yield moment in the reinforced concrete cross section (M_{Y0}) [3]. Shear force is verified by checking seismic performance on the basis of the relationship between the average shear stress (τ_{DA}) obtained by dividing the shear force occurring in the member by the cross-sectional area and the allowable shear stress (τ_a) because the conventional approach is simple and convenient to use. These relations are shown below:

$$M_D < M_{Y0}$$
$$\tau_{DA} < \tau_a$$

For shear stress verification, the allowable shear stress was determined taking into account the contribution of shear reinforcement (3.0 N/mm^2 for the design concrete strength of 24 N/mm^2; safety factor for earthquake loading=1.5) [4]. This section reports the results (see Table 5 and Figure 7) of verification of seismic performance against the Kasho Dam observed seismic wave, which would cause considerable structural response of the A Dam intake tower. Shear verification showed that all response values except the bulkhead response values would be within the allowable limits. Large section forces occur near the lower end of the structure because the intake tower is a columnar structure. Closer examination reveals that large bending moments occur in the walls (No.9 and No.10 shown at the bottom in Table 5) on both sides of the opening. The reason for this is thought to be that in cases where the structure is subjected to seismic force applied in the perpendicular direction, the part of the structure around the opening is more prone to vibration than other parts of the structure because the opening side of the structure is less stiff than the rear side. Examination of shear stresses reveals that stresses in the bulkhead (No.7) are larger than the stresses in other parts of the structure. In cases where the structure is subjected to seismic force in the perpendicular direction, the bulkhead and the rear section (No.2) act as shear walls to resist seismic force. The reason why stress concentration occurred is thought to be that the bulkhead is thinner than the rear section and is more prone to vibration because of relative proximity to the opening.

Table 5 Results of verification of the seismic performance of the A Dam intake tower against level 2 earthquake motions (Kasho Dam observed wave)

Location (numbers are shown in figure below)			9	10	4	5	2	7
Bending Moment		Indicated value/yield value	0.78	0.73	0.51	0.48	0.39	0.24
		Judgment（<1.0）	OK	OK	OK	OK	OK	OK
Shear	Out-of-plane	Indicated value/allowable value	0.23	0.21	0.31	0.30	0.27	0.17
		Judgment（<1.0）	OK	OK	OK	OK	OK	OK
	In-plane	Indicated value/allowable value	0.75	0.64	0.42	0.59	0.57	1.26
		Judgment（<1.0）	OK	OK	OK	OK	OK	NG



The above verification results indicate the possibility of bulkhead damage caused by the Kasho Dam observed seismic wave. It is thought, however, that since bulkhead damage will not hamper water flow even if the selective intake mechanism is damaged, the required seismic performance (overall structural stability and the intake function) can be retained. The A Dam intake tower was designed in the years when there was no concept of level 2 earthquake motions. The structure, however, was able to withstand level 2 earthquake motions without sustaining major damage because the modified seismic coefficient method was used as the design method in combination with verification through dynamic analysis.

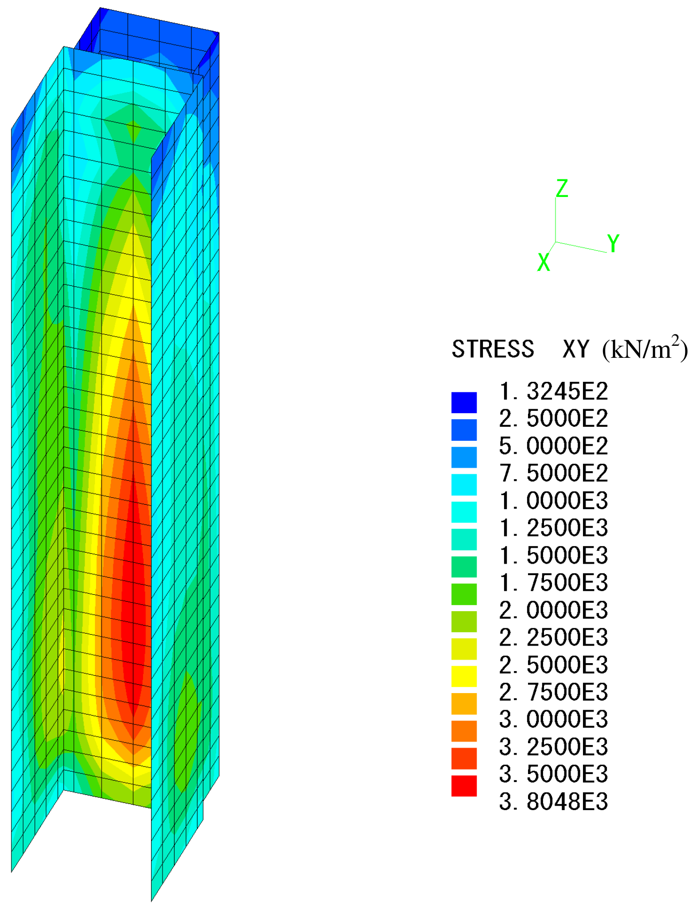


Figure 7 Maximum shear stress distribution obtained from 3-D FEM shell analysis

A CONCEPT OF SEISMIC DESIGN FOR SEPARATED SELECTIVE-INTAKE TOWERS

According to the above-mentioned analytical study results, a concept of seismic design for the separated selective-intake towers is proposed. In order to conduct seismic design, it is necessary to establish seismic performance levels of structures related to both structural importance and levels of earthquake motions considered.

Figure 6 shows target seismic performance levels proposed in the study. Seismic performance level 1 (Serviceability limit state) is defined that the main structure and its appurtenant and related structures are sound and fully functional (both structural safety and functionality are at a sound level). Seismic performance level 2 (Damage control limit state) is defined that the main structure remains safe and largely sound (earthquake-induced cracks without further damage are allowed), the appurtenant and related structures can be kept functional after the earthquake (to be verified through post-earthquake inspection). Moreover, Seismic performance level 3 (Ultimate limit state) is defined that the main structure can be allowed the degree of damage that does not destroy the stability of the structure (the degree of damage that permits permanent restoration at later dates) and the appurtenant and related structures permit permanent functional restoration through extensive member replacement or repairs. In the Japanese seismic design codes, two levels of earthquake motions such as level 1 and level 2 earthquake motions are set, and Seismic performance level 1 is required for level 1 earthquake motion and Seismic performance level 2 is required for level 2 earthquake motion, respectively. The two-stage seismic design procedure in Japan is based on knowledge accumulation of research work in terms of post-yielding behavior for structures such as bridge structures and so on. Therefore, the seismic design methods for level 2 earthquake motions are constructed under cost-performance advantages in design practice. It must be noted that seismic design of structures without knowledge accumulation for post-yielding behavior needs to pay attention for seismic performances required.

Table 6 Proposed seismic performance levels and limit states for intake tower facilities

Target level	Limit state and verification criteria	Earthquake motion level
Seismic Performance Level 1	• **Serviceability limit state** The main structure and its appurtenant and related structures are sound and fully functional. • **Verification level** Stresses are within the allowable limits.	Level 1 earthquake motion
Seismic Performance Level 2	• **Damage control limit state** The main structure remains safe and largely sound (earthquake-induced cracks are allowed). The appurtenant and related structures can be kept fully functional after the earthquake (to be verified through post-earthquake inspection). • **Verification level** Yield strength is not exceeded, and there is little residual displacement.	Level 2 earthquake motion
Seismic Performance Level 3 (for the present, this is not for use in practice)	• **Ultimate limit state** In the case of the main structure, the degree of damage that does not destroy the stability of the structure is allowed. In the case of appurtenant and related structures, permits permanent functional restoration through extensive member replacement or repairs. • **Verification level** The ultimate strength of the main structure is not exceeded, and residual displacement is kept within the allowable limits. The ultimate strength of the appurtenant and related facilities is not exceeded, but allowed partial damage.	Level 2 earthquake motion

A failure mechanism of stand-alone intake towers for large seismic loads is not clearly understood because of lack of research work in this field. Thus it seems that it is dangerous to expect a well post-yielding behavior for structures like intake towers. Intake tower structures can maintain intake and water flow functions even though partial structural damage may suffer. On the other hand, if the intake tower suffers large damage with residual deformation, it is completely hard to rehabilitate the structure and restore functions because of its low accessibility. Due to this reason, required seismic performance of the separated selective-intake tower need to separately make for both structural stability and water flow function. And also in order to meet this requirement, components of the intake tower divide into two parts as the main structure directly related to structural stability and its appurtenant structures related to water flow function.

Considering the above-mentioned issues, a proposed concept of seismic design for separated selective-intake towers is that the structures must meet requirements of Damage control limit state (Seismic performance level 2) for level 2 earthquake motions. In order to satisfy the requirement, calculated member forces due to level2 seismic loads need to stay within yield levels for both the main structure and its appurtenant structures like intake gates and gate stops so on. Seismic performance level 3 is defined in this study as listed in Table 6, however, it must be noted that for the present Seismic performance level 3 may not be straightforwardly applied for seismic design of intake towers because of lack of knowledge of post-yielding phenomena for the structures.

CONCLUSION

In view of the conditions of civil engineering structures in recent years, there is no avoiding the fact that intake towers, too, need to be properly designed against level 2 earthquake motions. Seismic design of structures against level 2 earthquake motions is important because it governs the seismic performance of the structures. In this study, a seismic design calculation method for level 2 earthquake motions has been proposed, and its applicability has been studied. Specifically, the micromotion-based calibration of structural models with natural periods and real earthquake records has been studied, and an appropriate modeling method has been defined. Comparison was also made between real earthquake records and analysis models to verify the engineering validity of the proposed three-dimensional FEM shell model. On the basis of these study results, the seismic performance of an existing intake tower against level 2 earthquake motions was evaluated. These results were obtained from a limited number of cases and cannot be generalized. The results do indicate, however, that intake towers similar to the one considered in this study can be rationally designed to stay largely within the elastic response range when subjected to level 2 earthquake motions.

The important thing in discussing the seismic performance of structures is to define seismic performance requirements and corresponding limit states and specific verification criteria. In this study, particular relationships among seismic performance, limit states and verification criteria have been proposed. Although further study is needed, knowledge that will prove useful in practical design has been gained from the study on the existing intake tower considered in the study. It is the authors' sincere hope that this paper will help develop a better seismic design method by which to protect intake towers from level 2 earthquake motions.

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