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DYNAMIC BEHAVIOR OF AQUEDUCT STRENGTHENED BY INVERTED TRIANGLE TRUSS DURING STRONG EARTHQUAKE

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ABSTRACT

It is very important to keep the performance of aqueducts safe after a strong earthquake. The seismic design code for aqueducts has been based on the updated Design Specification of Highway Bridge Part V Seismic Design in Japan. However, the characteristics of aqueducts are different from those of road and railroad bridges. There have been many reports dealing with the dynamic analysis of road and railroad bridges. Relatively little attention has been paid to the earthquake-proof characteristics of aqueducts. This paper describes the measured values of the damping factor and natural period of actual truss type aqueducts, and the nonlinear dynamic analysis of this aqueduct is discussed. This nonlinear dynamic analysis was conducted for simple beam type superstructure system of bridge subjected to some extreme ground motion. It is found from the analysis that the effect of the measured values of the damping factor is significant in the response of superstructure and bearing.

Introduction

The water supply system in Kobe city was completely disrupted and its principal functions were lost in the 1995 Hyogoken Nambu Earthquake. At the same time, aqueducts that are located throughout the disaster area in Kobe city were also damaged. The characteristics of the damage to aqueducts during the earthquake are summarized as follows; (1) damage to abutments that are moved due to liquefaction, (2) damage to bearing and mechanical joint that are severed from abutment and pier, (3) damage due to local buckling of superstructure. Although it is not realistic that these damages be perfectly prevented, strengthening their performance during strong earthquakes is very important. Some previous studies on aqueducts exist for experiment and analysis. In one experiment, Mizuta (1999, Japan) made a survey of the damping factor.

In the analysis, Mizuta (1999, Japan), Takeuchi (2000, 2001, Japan) and Otsuka (2002, Japan) have studied on dynamic behavior of aqueducts. Also we have been experimenting and studying. These are summarizes as follows; (1) the damping factor of aqueducts is smaller than that of Specification of Highway Bridge Part V Seismic Design in Japan, and (2) bearings that are made of iron steel are damaged in most linear dynamic analysis. However, in the previous analysis, nonlinear behavior of aqueducts was not sufficiently discussed. In this study, analysis was conducted for the truss type aqueduct in consideration of the nonlinear behavior of bearings.

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Analytical Model of Aqueduct

The outline of the aqueduct for analysis

The structure of the aqueduct in this study is a truss type, which is made of steel pipes on the soft ground. The superstructural length is 35.1 meters and skew is 60 degrees. Fig.1 shows the structure of the aqueduct. The clearance between the upper chords is 2.5 meters, the height between the upper and lower chords is 2.8 meters, and the chord size is 318.5mm. Water pipes have the same function as the bottom chord. Mechanical joints that are made of steel pipes are located at both ends of the superstructure. The passage for safety checking is located in this aqueduct. Fixing and movable bearings are used and made of steel. The substructure is supported by piles that are made of reinforced concrete.

Conditions and model

Analysis was carried out by the Newmark β method (β =0.25).The integral time step was 0.002 seconds. For the damping model, Rayleigh damping was used. Fig. 2 shows the analytical model. The aqueduct was modeled in three dimensions. The superstructure and substructure were modeled by beam elements and axial beam elements. In the bearing of the model, fixing bearings did not obstruct any rotation. Movable bearings obstructed neither displacement for change depending on temperature, nor retraining displacement due to movement limitation. The foundation of the structure was modeled by SR spring, and it was located in the X-direction and Z-direction separately as individual elements. In this analysis, water mass in the chords and ladder were modeled, but mechanical joints were not modeled since the characteristics of the restoring force were not sufficiently grasped. Nonlinear characteristics of bearings that were made of iron steel were modeled only for the Z-direction. It is because the attention was paid to the Z-direction of the superstructure, which stood high. Fig.3 shows the hysteretic curves. In the nonlinear model, when the restoring force of bearing went over F_2, which corresponds to force of linear limit, the load was shifted to F_1 on the hysteretic curve. Stiffness K_1 at this time was 100 times as large as substructure rigidity, and the factor for dynamic friction μ was set to 0.05. Moreover, two anchor bolts (SS400) of steel bearing with ϕ23mm (A_w = 415.5mm^2) were combined with the substructure per one bearing. In this analysis, the yield shear force of anchor bolts was defined as yield (121kN/piece) of steel bearing. Herein, τ_a of the anchor bolt was taken as 80Mpa. DYNA2E (CTC, Japan) was chosen, and it was a commercial program for dynamic analysis known to be powerful in handling structural nonlinearities.

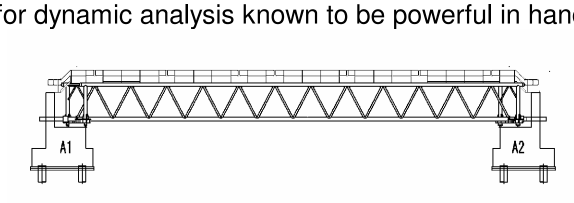


Figure 1. View of truss type aqueduct.

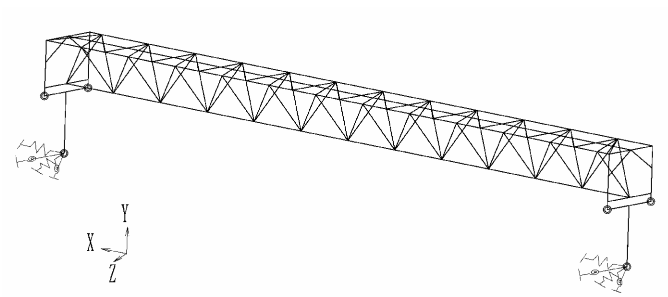


Figure 2. Analytical model.

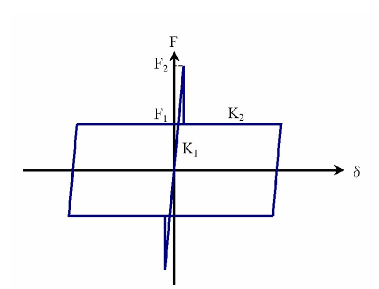


Figure 3. Restoring force model of non-linear behavior of fixed bearing model.

Table 1. Case of analysis.

Case No.	Damping method	Damping factor (superstructure)	Frequency and factor of mode factor
Case-1	Rayleigh	-	$f_1=3.27Hz$, $h_1=5.00$ % $f_2=50.0Hz$, $h_2=5.00$ %
Case-2	Rayleigh	h= 0.6%	$f_1=3.27Hz$, $h_1=1.78$ % $f_2=50.0Hz$, $h_2=5.00$ %

Table 2. Analytical results of natural period.

Mode No.	Frequency Hz	Effective mass ratio X	Effective mass ratio Y	Effective mass ratio Z
1	3.27	0.000	0.000	0.120
2	4.12	0.012	0.066	0.000
3	6.12	0.003	0.000	0.016
4	6.70	0.453	0.004	0.000
5	7.54	0.004	0.000	0.016

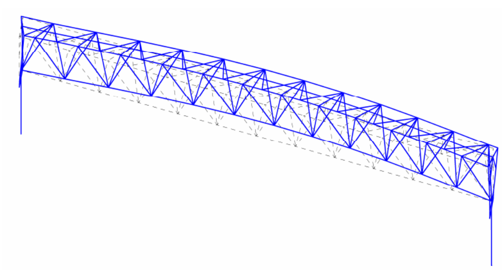


Figure 4. View of 1st natural mode.

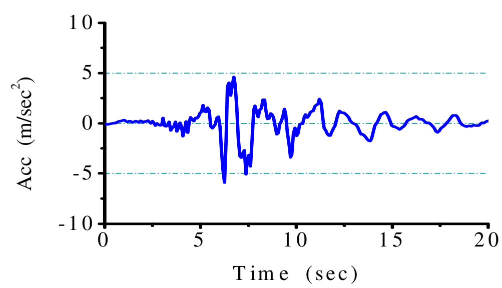


Figure 5. Input ground acceleration.

Analytical case

Table1 shows analytical cases. In the analysis, we used two cases. “Case-1” was based on normal damping factor of the Highway Bridge Code. In this case, the damping parameter of 1st-mode is used h_1=5%. “Case-2” was based on the experimental data. In this analysis, almost all the vibration modes were included, and the data in both cases were set to f_n=50Hz and h_n=5.00%. The value of h=0.6% (superstructure) as shown in Table 1 was obtained for experiments. In the experiments, the vibration characteristics were obtained by measuring the acceleration of micro tremors, forced vibrations induced by a walking live load with human weighing about 0.60kN and free vibrations after synchronizied jumping with one human weighing 0.60kN. Therefore damping factor h was deriverd from the damping ratio of the free vibration waveforms. Baseed on this value (h=0.6%), the damping factor h_1 for the first mode was calculated. The facter h_n for secondary mode of Rayleigh model was the value decided by the Highway Bridge Code in Japan.

Natural period analysis and analytical results

In natural period analysis, the sum of the total effective mass ratio became 100% using the Householder method. Table 2 shows the result of natural period, and Fig.4 shows the shape of 1st mode. The analytical result was in the primary symmetry mode with natural frequency of 3.27Hz, and the secondary mode was in the perpendicular direction with 4.12Hz.

Input acceleration waveform and input direction

Input ground motion was Higashi-Kobe Bridge (N12W) accelerogram recorded in Hyogoken-Nambu

Earthquake, and analytical time step was 20 second. Fig.5 shows time history of input ground motion. The maximum acceleration value was 591gal (t=6.29s). Z-direction was chosen for single input because the response for Z-direction distinguished higher than that for vertical direction through natural period analysis.

The Results and Consideration of Response Analysis

The results of case-1

(1) Sectional force of superstructure

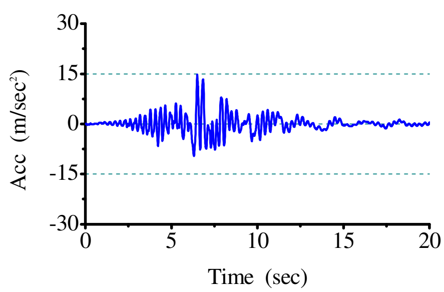
As for sectional force of superstructure, at axial force the main and lateral chords were noted. The maximum value of main chords was 726kN that corresponded to 37% of N_a, and the maximum value of oblique chords was 187kN that corresponded to 34% of N_a. These results were both in permissible range. The maximum value of lateral chords was 90kN that corresponded to 26% of N_a. Herein, "N_a" shows local buckling force.

(2) Time histories of acceleration and displacement

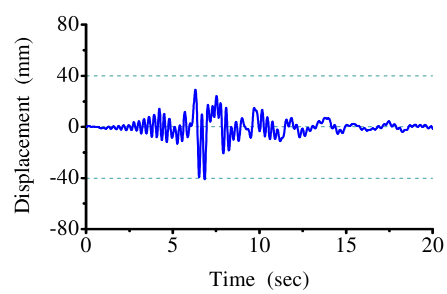
Fig. 6 shows computed time histories of acceleration and displacement at the center of the span. The horizontal maximum acceleration was 1480gal (t=6.508s) that was 2.5 times as large as the maximum of input acceleration, and the maximum displacement was 41mm (t= 6.840s).

(3) Force of bearing

Fig. 7 shows the typical time history response of bearing. This shows that the upper lift force has occurred at bearing. The maximum upper lift force was -44.9kN(t=6.25s) which was equivalent to 69% of the dead load (R_d). Moreover, since R_d of bearings were 65 kN/a piece, it turned out that the maximum upper lift exceeded -0.3R_d that was the minimum of the design upper lift in the updated Design Specification of Highway Bridge Part V Seismic Design in Japan.



(a) Time history of acceleration



(b) Time history of displacement

Figure 6. Time histories at superstructure (Case-1).

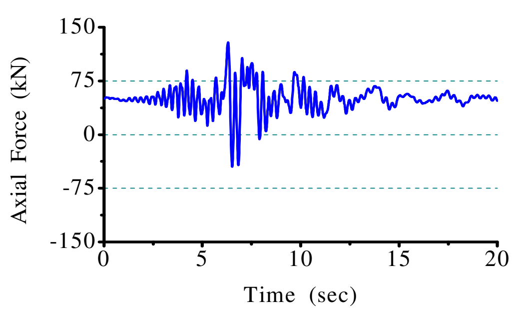
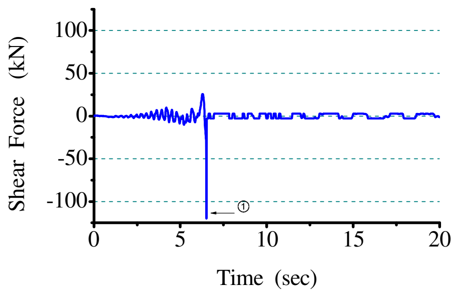
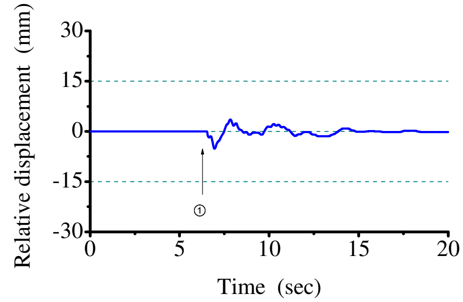


Figure 7. Time history of axial force at bearing (Case-1).



(a) Time history of shear force



(b) Time history of displacement

Figure 8. Time histories at bearing (Case-1).

(4) Shear force of bearing and displacement

As for shear force of bearing and displacement, the failure of steel bearing, the nonlinear behavior of shear force and relative displacement between the upper and lower bearing after the failure were noted. Fig.8 shows the time histories of shear force and relative displacement of typical bearing. After the bearing was broken at 6.530s as shown in Fig.8, relative displacement of 5.2mm (t= 6.946s) occurred at the maximum. From these time history figures, after the failure of bearing as shown in ① it was conformed that the behavior has a longer period. But they did not show free vibration, losing the support of a beam.

The results of Case-2

(1) Sectional force of superstructure

As for sectional force of superstructure, the upper, lower, oblique and lateral chords were noted at axial force. The maximum value of main chords was 644kN that corresponded to 33% of N_a, and the maximum value of the oblique chords was 191kN that corresponded to 35% of N_a. These results were both in permissible range. Maximum value of lateral chords was 87kN that corresponded to 25% of N_a.

(2) Time history of acceleration and displacement

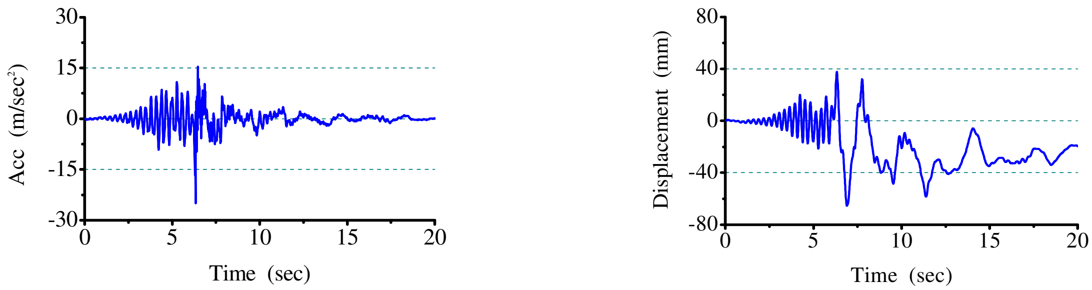
As for time histories of acceleration and displacement, the values at the center of the span of superstructure were noted. Fig. 9 shows the time history of response at this position. The horizontal maximum acceleration was 2641gal (t= 6.340s) that was about 4.5 times as large as the maximum of input acceleration, and the maximum displacement was 65mm (t= 6.892s).

(3) Force of bearing

As for bearing, reaction of uplift force of steel bearings was noted. Fig. 10 shows the time history of response figure for typical bearing. This shows that the upper lift force occurred at bearing. The maximum upper lift force was -48.2kN(t=6.316s) which was equivalent to 74% of the dead load (R_d), and it turned out that the maximum upper lift exceeded -0.3R_d.

(4) Shear force of bearing and displacement

As for shear force of bearing and displacement, nonlinear behavior of shear force and relative displacement after failure were noted. Fig.11 shows time history of shear force and relative displacement of typical bearing. As bearing was broken at 6.316s as shown in Fig.11, maximum relative displacement occurred that was 126.9mm (t= 7.024s). The response of superstructure behaved like free vibration which lost the support of bearing. Judging from the amount of maximum relative displacement, fatal damage like falling of a beam from substructure did not occur. But it turned out that the displacement exceeded 32mm which was allowable horizontal displacement of mechanical joints (CL-A, Japan).



(a) Time history of acceleration (b) Time history of displacement

Figure 9. Time histories at superstructure (Case-2).

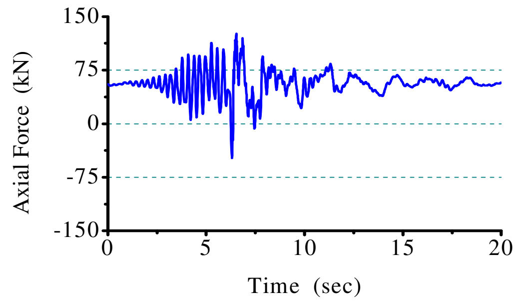
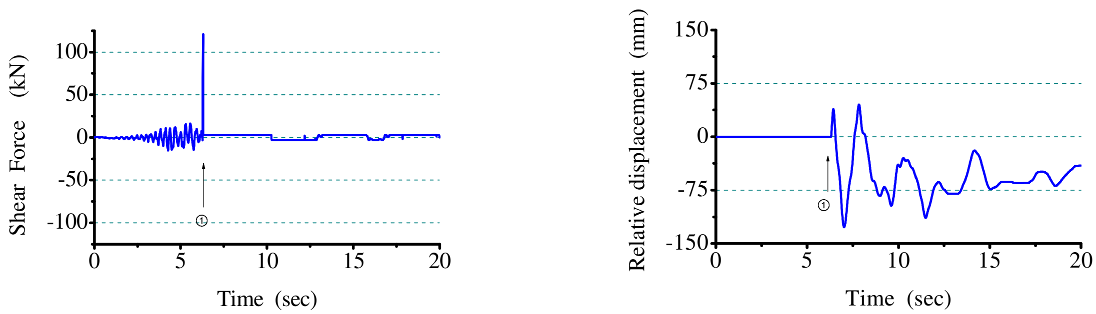


Figure 10. Time history of axial force at bearing (Case-2).



(a) Time history of acceleration (b) Time history of displacement

Figure 11. Time histories at bearing (Case-2).

Comparison and consideration of analytical results

(1) Sectional force of superstructure

As for sectional force of superstructure, it behaved linearly in both Case-1 and Case-2, and damage was not generated. At this analysis, the influence of the difference in damping factor was not checked. It is considered that this is because the earthquake input to superstructure was reduced when bearing was damaged and aqueducts had resistant power, since it was designed by horizontal wind load as past research(2000,Takeuchi) pointed out.

(2) Time history of acceleration and displacement

As for the time histories of acceleration and displacement, it turned out that Case-2 became a bigger response than Case-1. About these differences, the influence of the difference in damping factor was checked because the maximum acceleration of Case-2 was 1.78 times as large as that of Case-1 and the maximum displacement was 1.59 times. Especially from the displacement response of Case-2, it turned out that the response behaved like a free oscillation after around t= 8s. This is considered to be the influence of the damage to bearing.

(3) Force of bearing

As for force of bearing, the upper lift force occurred in both cases. The influence of the difference in damping factor was checked because the force of Case-2 was 1.07 times as large as that of Case-1. The upper lift forces generated were over $-0.3R_d$ and they were 69 to 74% of R_d. From the above, it is important to set up the damping factor of superstructure appropriately in setting up the seismic force which acts on bearing for the design of the bearing of aqueduct.

(4) Shear force of bearing and displacement

As for shear force and displacement of bearing, it turned out that bearing was damaged in both cases. Big shear force occurred at around t= 6s, and bearing was damaged as shown in Fig.8 and Fig.11(a) and since the restoring force moved to the hysteretic characteristics with slide force F_1 shown in Fig.4 after that, and shear force reached the ceiling. As mentioned above, after bearing damaged, the response showed a longer period in Case-1. But since the maximum relative displacement was also as slight as 5.2mm and residual displacement was also small, mechanical joints were not affected. However, in Case-2, large relative displacement was 127.9mm as 24.5 times of Case-1 after bearing damaged, and mechanical joints also broke. Considering the role of the bridge after an earthquake disaster, it is equivalent to the same damage as a falling beam from the substructure of a highway bridge. Therefore, in examining seismic design of aqueducts, it is very important to evaluate appropriately the damping factor of any aqueduct.

Conclusions

In this study, a nonlinear analysis of a truss type aqueduct with broken bearing was conducted. In the analysis, comparison analysis was tried in two cases that were based on a normal damping value from the Highway Bridge Code and based on the damping factor that was obtained by experiment. Although analysis was carried out based on assumptions and some subjects still remain, the results obtained in this analysis are given below:

(1) The section force of superstructure showed linear behavior, and the influence of damping factor was not checked.

(2) The maximum response acceleration was 1.78 times and the maximum response displacement was 1.59 times as large as the input earthquake motion. Namely, the influence of the difference in damping factor was checked.

(3) The upper lift force was generated in the reaction of the bearing. The upper forces were large and about 70% of the dead load.

(4) Relative displacement of a bearing was received large influence by the difference in damping factor. When the analysis was carried out, based on the damping factor that was obtained in the oscillating experiment, the mechanical joint was also damaged. Namely, it turned out that the performance of the aqueduct was not secure.

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Type: CL-A, ”CLOSER JOINT”, *Japan Victaulic., Co Ltd*, Japan.