SEISMIC DESIGN AND BEHAVIOR DURING THE HYOGO-KEN NANBU EARTHQUAKE OF THE AKASHI KAIKYO BRIDGE

Masahiko YASUDA[1], Makoto KITAGAWA[2], Toshimi MORITNI[3] And Susumu FUKUNAGA[4]

SUMMARY

The 1995 Hyogo-ken Nanbu(Kobe) Earthquake with magnitude of 7.2 broke out near Kobe City in Japan. The epicenter of the earthquake was very close to the construction site of the Akashi Kaikyo Bridge. When the earthquake occurred, all the cable strands were erected and cable squeezing work was in progress. Although this earthquake caused quite large damages to the surrounding area including Kobe City, the Akashi Kaikyo Bridge had hardly suffered from damages. But the center span length was widened by about 1m due to the movement of ground. This extension was handled by adjusting the length of some truss members and the places where the cable bands would attach.

This paper describes the seismic design of the Akashi Kaikyo Bridge, inspection after the earthquake, input seismic motion estimated at bedrock, analysis of the earthquake response of the Akashi Kaikyo Bridge. The estimated input ground motion was evaluated considering local ground condition based on records of very similar geology relatively near the bridge. And the seismic response analysis was performed by using the estimated ground motion to compare the analytic results with actual records of vibration of the towers.

INTRODUCTION

The following matters are the features of the Akashi Kaikyo Bridge distinguished it from other bridges on the viewpoint of the seismic design.

[1] Large substructure dimensions are necessary to support the superstructure.

[2] Because the granite layer at the bridge site is extremely deep, the supporting ground layer consists of Kobe formation of the Miocene soft rock and the Akashi formation of Pleistocene to Pliocene semi-cemented gravel and sand thickly distributed above the granite. (Figure 1)

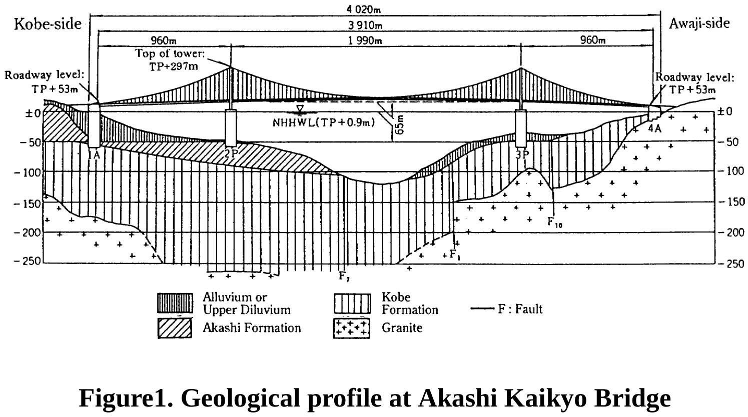


Figure1. Geological profile at Akashi Kaikyo Bridge

[1] *Earthquake Disaster Prevention Center, Ministry of Construction. Japan, Email:m-yasuda@pwri.go.jp*
[2] *Long span bridge Engineering Center, Honshu Shikoku Bridge Authority, Japan. Email:kitagaw@hsba.go.jp*
[3] *Long span bridge Engineering Center, Honshu Shikoku Bridge Authority, Japan. Email:moritani@hsba.go.jp*
[4] *Long span bridge Engineering Center, Honshu Shikoku Bridge Authority, Japan. Email:fukunaga@hsba.go.jp*

For this reason, the non-linearity of the ground and the dynamic interaction between the foundation and the ground during an earthquake could not be ignored. So the following two concepts were incorporated into the design.
[1] The constraining effect of a large foundation on the ground motion (effective earthquake motion)
[2] The underground radiation of vibration energy and the dynamic restoring force achieved by vibration of both the ground and the foundation.

For the design input ground motion, a magnitude 8.5 earthquake in anticipation of a interplate large-scale earthquake was considered and the probability of seismic activities with magnitude 6.0 or more within a radius of 300 km of the bridge site were also evaluated.

Considering the seismic design above mentioned, the locations of active faults were considered and the foundations were so placed as to avoid them.

INSPECTIONS AND INVESTIGATIONS FOLLOWING THE EARTHQUAKE

Visual Inspection

A visual inspection was carried out immediately after the earthquake to check the condition of the main tower foundations, the anchorage, main towers, cables and other bridge components along with the catwalks, squeezing machines, and other temporary structures and bridge construction machinery. This inspection revealed the following facts.
[1] There was slight movement between the first tier and bottom plate of the tower columns at both 2P and 3P, but later analysis confirmed that this shearing was not a problem because it applied little stress to the towers.
[2] The examination of the main bridge structure did not find any cracked concrete, buckled or cracked steel plate, damaged cable wires, or slippage above the saddle.
The visual inspection revealed that the earthquake motion had only slight effect on the Akashi Kaikyo Bridge.

Survey Study, Etc.

Shortly after the earthquake occurred, a substantial change in the Nojima Fault which is located in the southwest of the bridge was discovered in the northern part of Awaji Island. Because it was feared that this change might have effected the bridge, immediately after the earthquake, a transit was used to perform a simple survey, an electro-optical distance meter survey was performed, and finally, the locations of the foundations were surveyed by a GPS survey. These surveys confirmed lateral and vertical displacement of the foundations as shown in Figure 2 along with rotational displacement both horizontally and vertically. The amount of lateral displacement of the foundations exceeded the anticipated value of about 36cm.

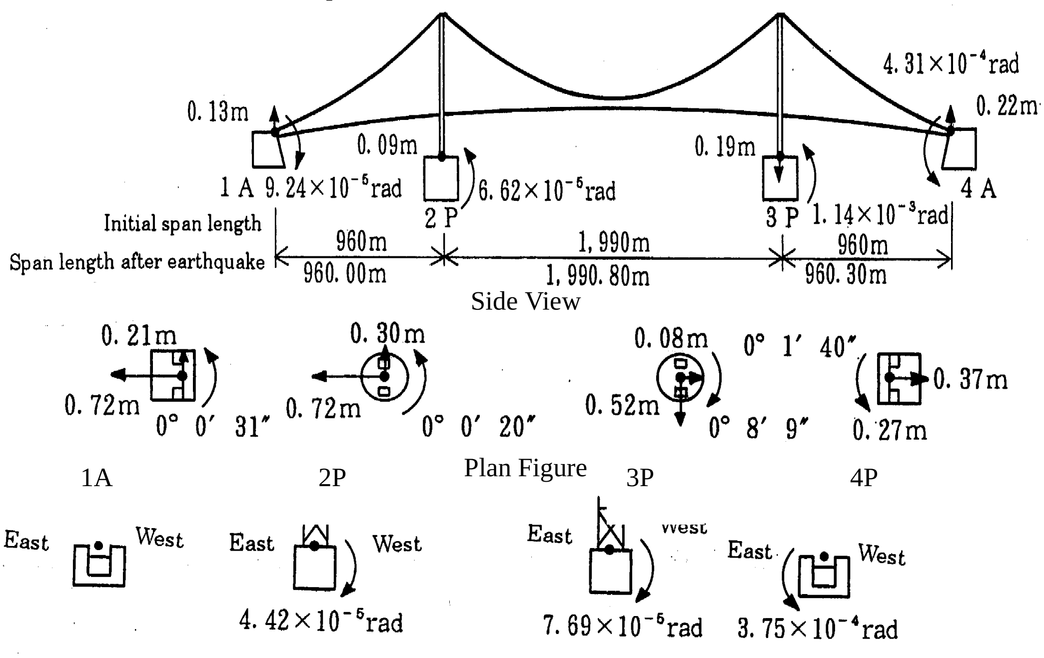


Figure 2. Displacement of the foundations

At the same time, surveying points installed in the surrounding ground in preparation for the bridge work were surveyed. The results have revealed that the ground had moved between 50 and 100 cm around the bridge. A separately conducted survey of the seabed found no trace of the movement of the foundation above the seabed surface and confirmed that the displacement of the foundation was a result of the movement of the ground itself.

Results of Measuring by Instruments

Foundation Settlement

According to the results of measurements by sliding micrometers that had been performed since the start of construction of the foundations in order to assess the amount of foundation settlement, as shown in Figure 3, the earthquake caused settlement of 20 mm at 2P. Because an examination of the distribution of settlement in the ground induced by the earthquake reveals that it was analogous to the distribution of settlement caused by past increased self-weight load, it has been concluded that the earthquake did not affect the bearing properties of the ground.

Main Tower Velocity Records

Velocity meters were installed in the top and in the middle part (about 2/3 of the way from the foundation to the top) of the two main towers. Figure 4 shows part of the observed records.

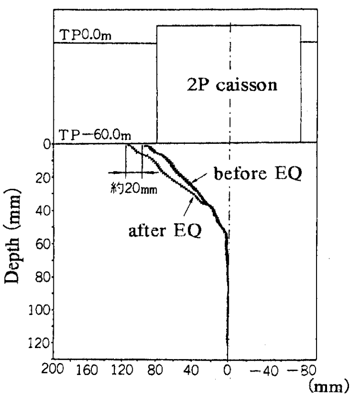


Figure 3. Foundation settlement (2P)

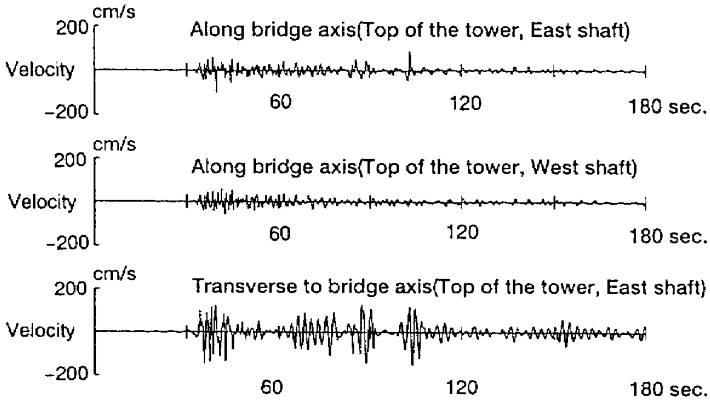


Figure 4. Velocity record at the top of Kobe-side (2P) tower

EFFECTS OF FOUNDATION MOVEMENT AND CORRECTIVE MEASURES

Study Method

It was assumed that the displacement of the foundations had altered the overall structural system of the bridge, creating additional stress in the towers and cables that had already been erected. Therefore, an analysis was performed by applying the actual measured lateral, vertical, and rotational displacements of the foundations to a three-dimensional model of the completed superstructure system as static displacements in order to verify the shape of the suspension bridge and the resultant stress of each of its structural components.

Effects on Structural Members

Figure 5 shows the new equilibrium state of the revised design of the completed Akashi Kaikyo Bridge system that was obtained analytically. The results revealed the following facts and the following corrective measures were taken.

[1] The amount of sag of the cables decreased by about 1.3 m in the center span and by about 0.3 m in the side spans. This created extra tension in the cable equal to about 0.6% of the maximum tension in the initial design, but this was not considered a problem because it was within the allowed tension.

[2] The stress added to the tower bases by the displacement was a maximum of 18.2 N/mm^2, which was not a problem because it was smaller than the stress of 24.5 N/mm^2 based on the foundation displacement accounted for by the initial design.

[3] Correction for the displacement of the suspension bridge shape was provided by lengthening the stiffening truss members that had not yet been manufactured by 800 mm in the center span and by 340 mm in the Awaji Island side span.

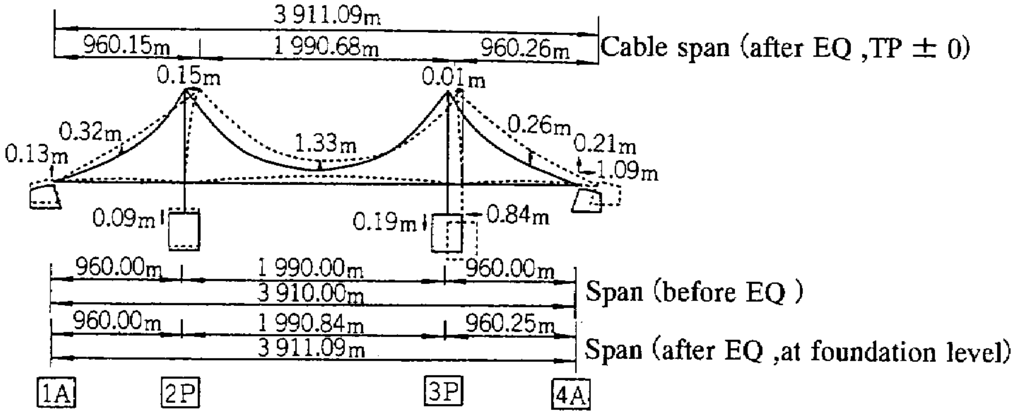


Figure 5. Relative deformation of bridge (East Side cable)

ESTIMATION OF THE IMPUT GROUND MOTION TO THE FOUNDATIONS

Because as already stated, the input ground motion around the Akashi Kaikyo Bridge was not measured, attempts were made to estimate the input ground motion using the following methods.

[1] Estimation of the ground/input seismic motion from the ground - foundation - tower transfer function based on the records of the vibration of the towers

[2] Estimation based on a fault model

[3] Estimation based on records of very similar geology relatively near the bridge

Estimation method [1] could not provide good results because of the non-linear properties of the ground. Estimation [2] was shown to be imprecise on the short period side. Therefore the estimation was made based on method [3] using records on a hard diluvial layer obtained at the Kobe Marine Meteorological Observatory.

The N 35° E wave and the N 55° W wave synthesized from the N-S and E-W components at the Kobe Marine Meteorological Observatory were set as the longitudinal and transverse wave respectively. As shown in Figure 6, based on the geological conditions in the Akashi Straits, equivalent linear seismic response Analysis of horizontally layered soil (SHAKE) was performed to find the input seismic wave form in the base ground at the foundation location (granite). The acceleration response spectrum of the input ground motion is shown in Figure 7.

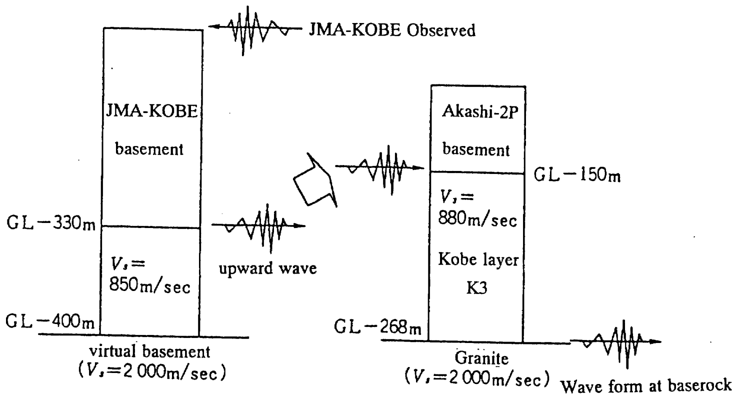


Figure 6. Concept of transformation method to wave form at base rock

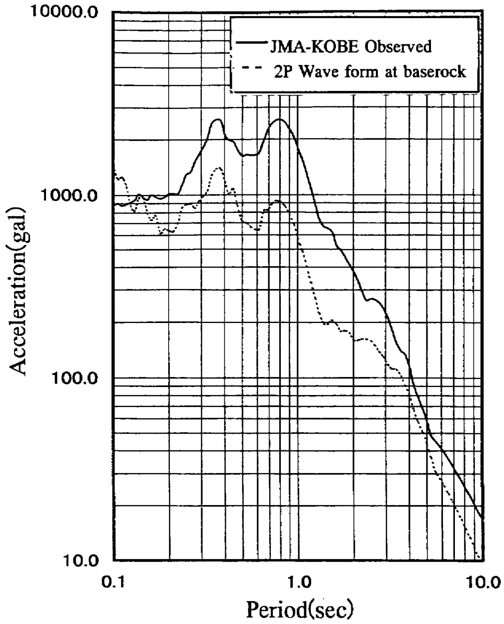


Figure 7. Acceleration response spectrum of 2P (transverse direction)

ANALYSIS OF THE EARTHQUAKE RESPONSE AT THE TIME OF THE EARTHQUAKE

As it would be necessary to observe the wind induced vibration in the main towers of the Akashi Kaikyo Bridge from during erection of the main towers to after completion of the bridge, velocity meters were placed inside the towers to measure their vibrations. The earthquake activated these velocity meters. From these data, the input ground motion was estimated to perform response analysis of a full model of the superstructure and the substructures during the earthquake in order to study behavior of the bridge.

Response Analysis of the Tower - Foundation System

To verify the appropriateness of the input ground motion and to study the behavior of the foundation during the earthquake, a response analysis of the tower - foundation system was performed for the transverse direction at the Kobe side. The analysis was performed using the time history FEM analysis method, which considered the non-linearity of the response, based on the strain dependence of the ground. Figure 8 shows the analytical model. The tower - foundation system model was used and weight of cables was added to the tower top so that the natural period of the main towers would be equal in order to account for the effects of the cables.

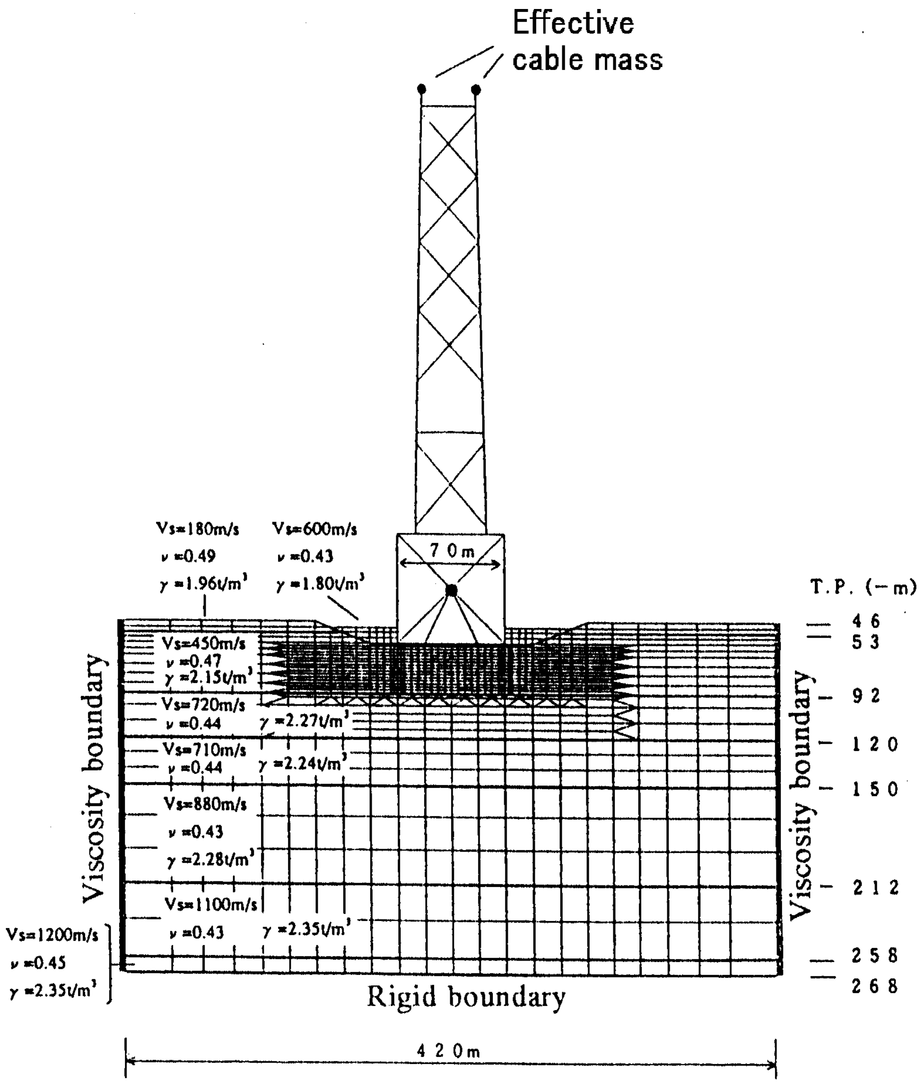


Figure 8. Analysis model of tower-foundation system

The non-linearity of the ground was taken into consideration by setting the stress-strain relationship in the modified Ramberg-Osgood model (hereinafter referred to as the "modified R-O model"). To incorporate separation and sliding of the foundation relative to the ground, the joint element was used.

Equation (1) shows the skeleton curve, which expresses the stress-strain relationship in the ground, and Figure 9 shows the conceptual diagram of the non-linearity of the ground. In the non-linearity of the ground, the parameters α and β were set in a way that satisfied the relationship between the shear modules G and the shear strain γ, and the damping constant h and the shear strain γ.

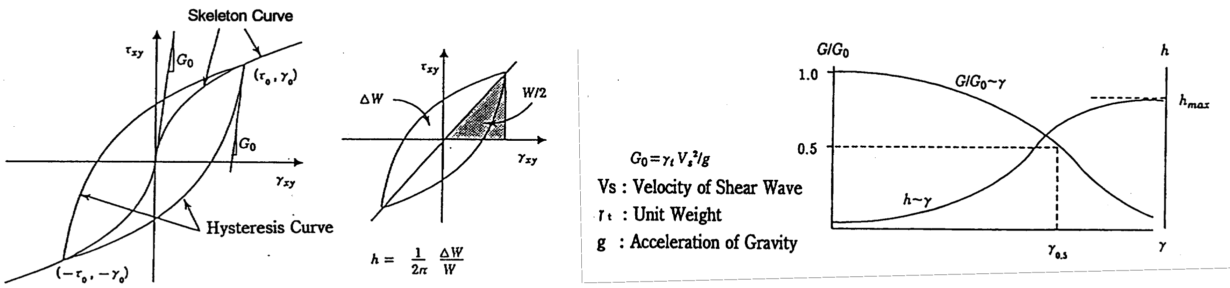
Skeleton curve: $$\gamma = \frac{\tau}{G_0}\left(1 + \alpha \cdot |\tau|^{\beta}\right) \qquad \cdots\cdots(1)$$

$$\alpha = \left(\frac{2}{\gamma_{0.5} \cdot G_0}\right)^{\beta}, \quad \beta = \frac{2 \cdot \pi \cdot h_{\max}}{2 - \pi \cdot h_{\max}}$$

where: γ :shear strain

τ :shear stress

G_0 : initial shear modules



α, β:parameter of the modified R-O model

Relation of Shear Stress(τ) and Shear Strain(γ), and damping Factor(h)

Analytical Model of Nonlinear Characteristics

Figure 9. Modified Ramberg-Osgood Model

As the modified R-O model is based on the Masing rule, stress-strain curve has a closed form. In this analysis, for the Akashi layer, to incorporate residual strain, non-linear parameter was introduced considering the results of the triaxial compression test on undrained condition.

Figure 10 shows comparison of the analytical values and the measurement records of the main towers' response. Although there are differences in the absolute response values, the analytical wave form properties generally reproduce the measurement records.

Figure 11 shows comparison of the analyzed and the measured residual settlement of the foundation. Although the distribution of settlement in the direction of depth is different, the total amount of the residual analyzed settlement is nearly same as the actual settlement.

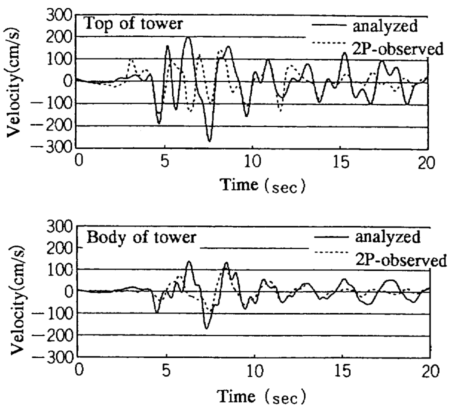


Figure 10. Comparison of analyzed data and observed data at tower

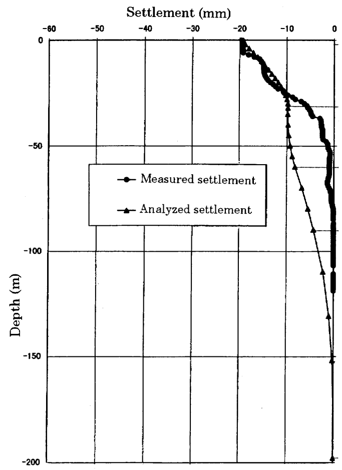
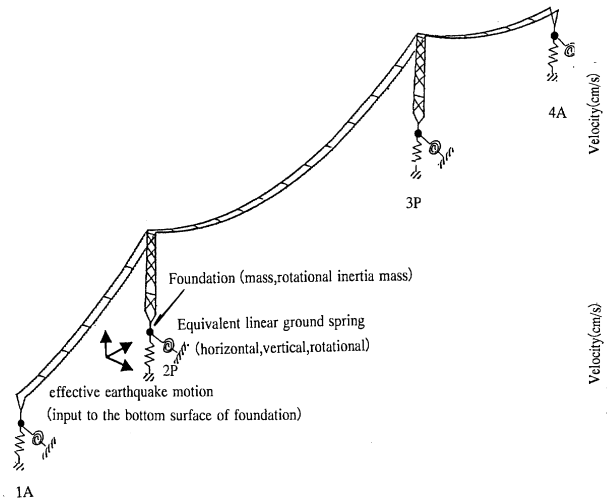


Figure 11. Comparison of analyzed and measured residual settlement at 2P

Response Analysis of the Structural System during the Earthquake

A response analysis that reproduced the behavior of the bridge during the earthquake was carried out. Figure 12 shows the analysis model of the structural system at the time of the earthquake. The analysis model, a three-dimensional frame model including the foundations, was used for a time history response analysis by simultaneously inputting ground motion to each foundation from three directions (transverse, longitudinal, and vertical).

The input ground motion to the bottom surface of the foundation (effective earthquake motion) was calculated by two-dimensional FEM analysis of the foundation - ground system. The ground spring used was an equivalent linear spring that equalizes the maximum response acceleration at the top of the foundation obtained from the tower - foundation system to accounting for non-linear properties. Assuming that the superstructure remains within the elastic range, its non-linear properties are not considered.



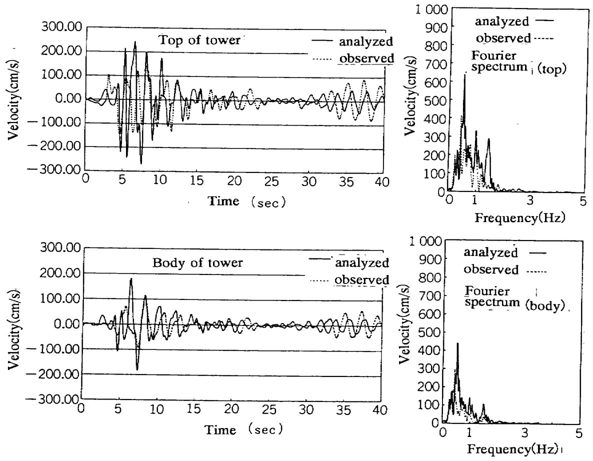


Figure 12. Analysis model of the structural system in the earthquake

Figure 13. Comparison of observed data at 2P (transverse direction)

Figure 13 shows the response of the main tower in the transverse direction along with the measurement records. It shows that although there are differences with the absolute value of the response, the analytical wave form properties generally reproduce the measurement records. They also reproduce the resurgence that characteristically occurs after nearly 30 seconds in measurement records. The analyzed Fourier spectrum of 2P shows a good conformity with the measurement records .It would be considered that the input seismic wave form at bedrock of 2P is nearly equal to the actual wave form.

When the stress in various parts of the main towers was calculated from the analysis results, the stress in the tower base was found to be lower than the stress allowed during an earthquake.

CONCLUSION

As the first example of a large scale earthquake occurring where a large suspension bridge was under construction, this study of the behavior of the Akashi Kaikyo Bridge during the 1995 Kobe Earthquake will make an important contribution to the design of future large bridges. From the results of the study ,it could be concluded that,

[1] Prior inspection for active faults is very important. As the locations of the foundations of the Akashi Kaikyo Bridge were decided to avoid the faults, the bridge had hardly suffered damages.

[2] As movements of faults have large effect on bridges, the movement of faults has to be considered in initial design.

[3] It is very useful to use modified R-O model for non linear FEM analysis in estimating residual displacement of foundations.

[4] It is very useful to evaluate input ground motion considering local ground condition based on records of very similar geology relatively near the bridge.

[5] It is necessary to conduct non-linear FEM analysis in estimating spring coefficients and damping ratio of foundations accurately during large earthquake.

REFERENCES

1. Yasuda,M., Kawaguchi,K., Ohe,S., (1996) “Revised Design of the Akashi Kaikyo Bridge due to the Hyogo- ken Nanbu Earthquake”, Technical Report of Honshu-Shikoku Bridge Authority,No77.

2 Tada,K., Jin,H., Kitagawa,M., Nitta,A., Toriumi,R. (1995) “Effect of Southern Hyogo Earthquake on the Akashi-Kaikyo Bridge” Structural Engineering International, March, PP179-181