RESPONSE ANALYSIS OF MIYAGAWA BRIDGE BASED ON A MEASURED ACCELERATION RECORD

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This paper presents an analysis of a measured acceleration record at Miyagawa Bridge, the first seismically isolated bridge in Japan. Amplification of the peak accelerations is examined between the deck, pier and ground motions. Seismic isolation effect can be clearly observed. Dynamic response analysis using the analytical model adopted for seismic design is made assuming the acceleration recorded at underground as the input ground motion. Response acceleration computed at the deck is quite similar with the measured acceleration although peak acceleration is larger in the analysis.

Key Words: Miyagawa Bridge, strong motion obsarbation, seismically isolated bridge, menshin design

1. INTRODUCTION

Miyagawa Bridge is the first seismically isolated bridge in Japan, and is located on National Highway No. 362 in Shizuoka ken [1) 2)] . It completed in March 1991. Strong motion observation has been made since the completion. Because the first record was obtained by an earthquake with magnitude 4.9, which took place close to the site, an analysis was made for verifying the effectiveness of Menshin Design [3)] . This paper presents the seismic behavior of Miyagawa Bridge based on the analysis of the recorded accelerations.

2. STRONG MOTION OBSERVATION AT MIYAGAWA BRIDGE

As was reported in the previous paper [1) 2)] , the Menshin Design (seismic isolation design) of

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Miyagawa Bridge was made in accordance with the Guidelines of Menshin Design of Highway Bridges [2]. In the Menshin Design, the Menshin devices are designed by both the Seismic Coefficient Method and the Bearing Capacity Method. In both methods, the lateral force is statically applied to the bridge, and the seismic safety is examined based on the allowable stress design approach in the Seismic Coefficient Method and the bearing capacity considering ductility in the Bearing Capacity Method.

The lateral force coefficient was not, however, reduced at the Miyagawa Bridge from the design value specified in the Specifications of Design of Highway Bridges [4], because it was the pilot project for verifying the effectiveness of the Menshin Design. The lateral force coefficient is therefore 0.2 in both longitudinal and transverse directions.

Lead rubber bearings [5] were adopted so that damping ratio of 0.12 can be obtained for the first mode of the bridge. Natural period of the bridge in longitudinal direction was assumed as 0.76 second in the Seismic Coefficient Method and 1.0 second in the Bearing Capacity Method. Because of the deformation dependence of the stiffness of the lead rubber bearings, the natural period is not the same for the Seismic Coefficient and the Bearing Capacity Method.

Soil condition at the site is I−group (stiff) based on the classification of ground condition specified in the Seismic Design Specifications of Highway Bridges [4].

Three 3−components feed−back servo type strong motion accelerometers were placed on the deck and pier crest at pier 1 (P1) and 10 m below the ground surface 10 m apart from the abutment 1 (A1) as shown in **Fig. 1**. The data are recorded on the analog magnetic type recording system when the ground acceleration larger than the triggering level is developed.

The first record was obtained by an earthquake with magnitude 4.9 which took place about 30 km north east from the site. Seismic Intensity of Japanese Methodological Agency was II.

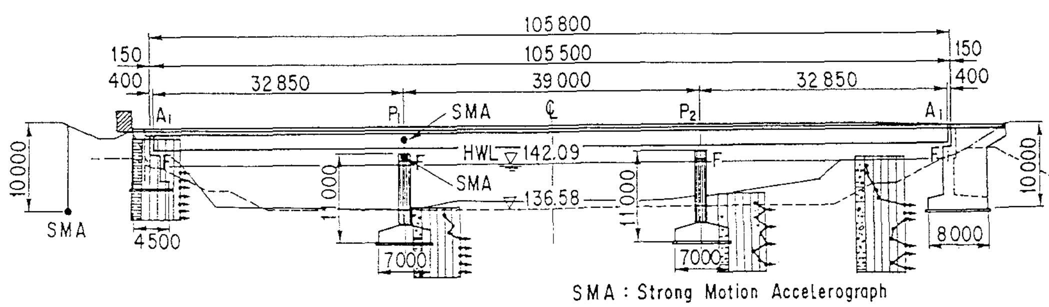

Fig. 1 Miyagawa Bridge and Location of Accelerometers

3. ANALYSIS OF MEASURED RECORDS

Fig. 2 shows the accelerations recorded at the deck, pier crest and 10m below the ground surface. Because intensity was so small, first part of the response could not be recorded. But main part could be successfully recorded.

Table 1 shows the peak acceleration. At 10m below the ground surface, the peak acceleration

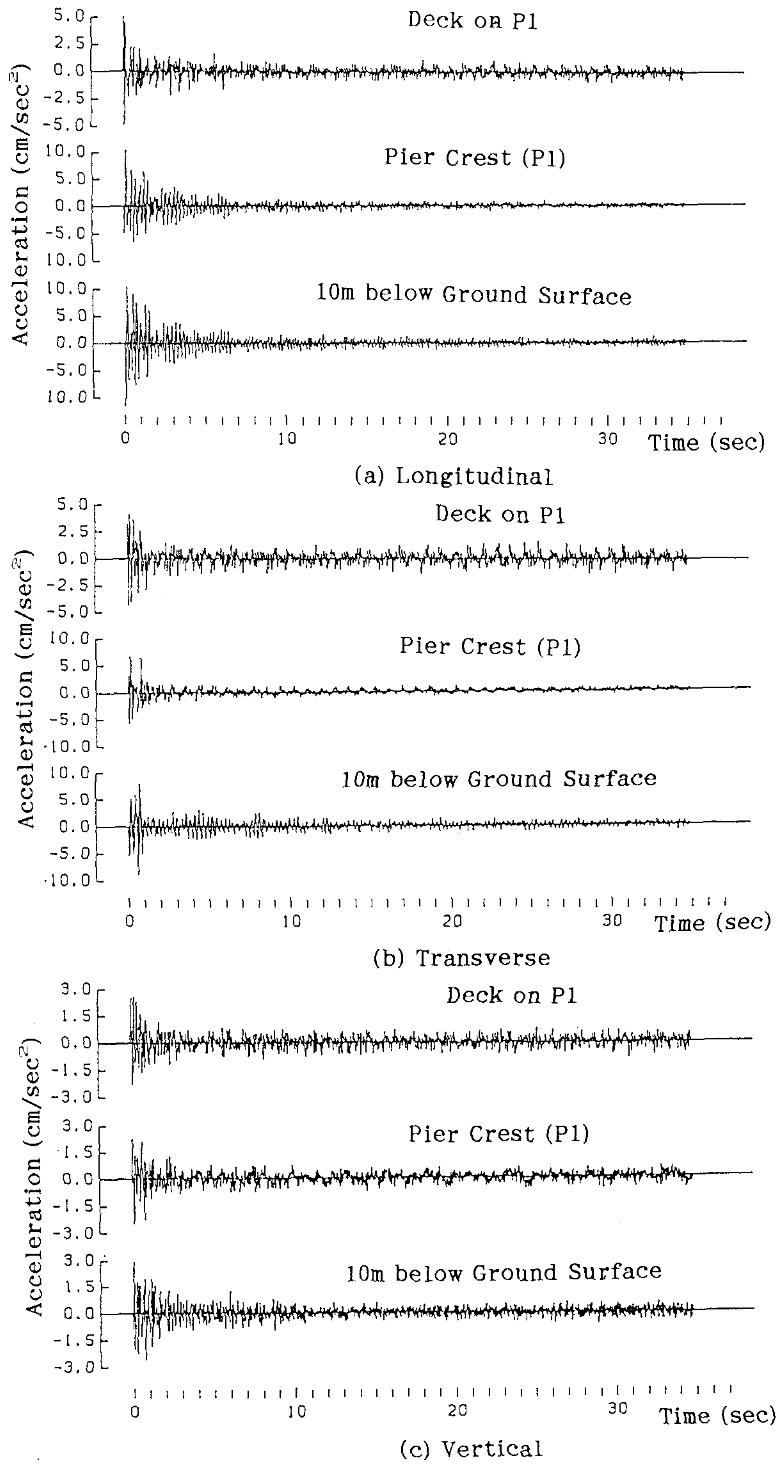

Fig. 2 Measured Accelerations

Table 1 Peak Accelerations Measured

Location	Longitudinal Direction	Transverse Direction	Vertical Direction
Deck on P1	5.0	4.3	2.6
Pier Crest of P1	10.4	6.6	2.5
10 m underground	11.6	8.9	2.9

was 11.6 cm/sec ^{2} in longitudinal direction and 8.9 cm/sec ^{2} in transverse direction. This decreased to 10.4 cm/sec ^{2} in longitudinal direction and 6.6 cm/sec ^{2} in transverse direction on the pier crest. Because the amplification of the peak acceleration from the ground to pier crest is generally about 2.5 for this type of bridges [6], the decrease of pier response in comparison with the ground acceleration at 10 m below the surface is apparent. Furthermore, it decreased to 5.0 cm/sec ^{2} in longitudinal direction and 4.3 cm/sec ^{2} in transverse direction on the deck.

Fig. 3 shows such decrease of amplification in terms of acceleration response spectra with damping ratio of 0.05. It is apparent that although ground motion at 10 m below the surface had the predominant period of 0.2 ~ 0.3 second, it is not predominant in the horizontal deck motion.

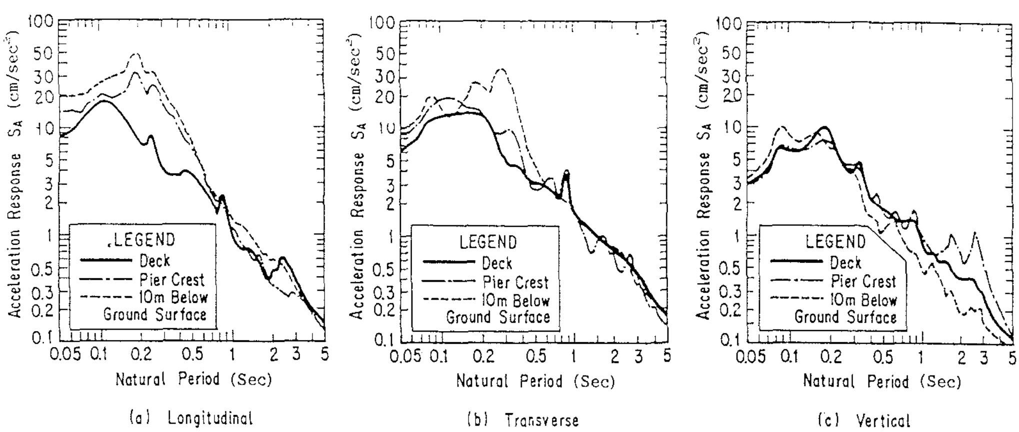

Fig. 3 Acceleration Response Spectra of Measured Acceleration Record (Damping Ratio of 0.05)

For studying the amplification from the 10 m below the ground surface to the pier crest and the deck, ratios of the acceleration response spectra presented in **Fig. 3** are shown in **Fig. 4**. It is seen in **Fig. 4** that the acceleration response spectrum ratio R defined as the acceleration response spectrum of the deck motion divided by the acceleration response spectrum of the ground motion measured at 10 m below the ground surface is approximately 0.2 at natural period of 0.2 ~ 0.3 second. The motion at this period is filtered out at the deck due to the effect of seismic isolation.

On the other hand, the ratio R defined as the acceleration response spectrum of the deck motion divided by the acceleration response spectrum of the ground motion measured 10 m below the ground surface increases to 1.4 in longitudinal direction and 2.0 in transverse direction at natural

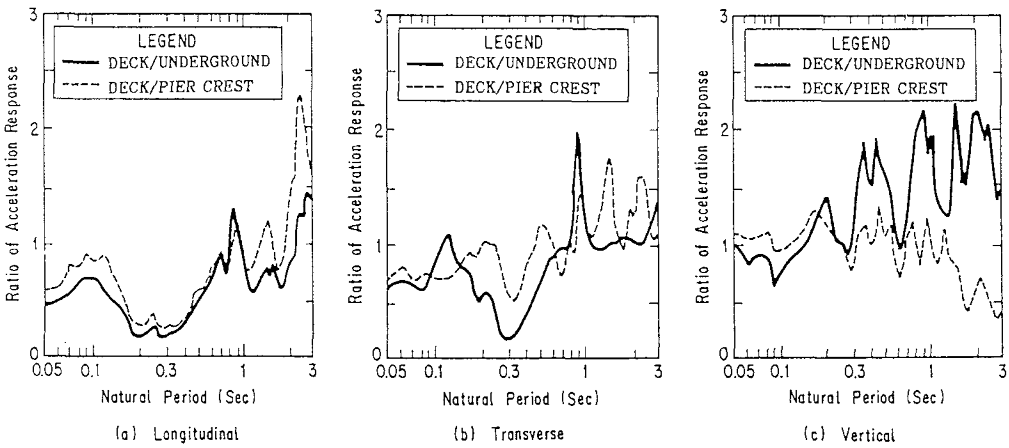

Fig. 4 Ratios of Acceleration Response Spectra

period of about 0.85 second. Reminding the fact that the natural period assumed in the Seismic Coefficient Method was 0.76 second, this represents the amplification of deck response due to resonance of the first mode. Reflecting the fact that the deformation of LRBs developed during the earthquake is much smaller than that assumed in the design by means of the Seismic Coefficient Method, stiffness of LRBs is approximately 25 % higher than that assumed in the design.

The ratio R in vertical component takes about 1.0 for natural period up to 0.3 second. This implies that the amplification from either 10 m below the ground surface or the ground surface to the deck is small. Although the ratio R from 10 m below the ground surface to the deck becomes large for natural period longer than 0.3 second, it requires to clarity the reason after more data are accumulated.

The LRBs were designed so that yielding of the lead plug be developed for the lateral force coefficient over 0.12. Of course, because the response during the earthquake was much smaller that

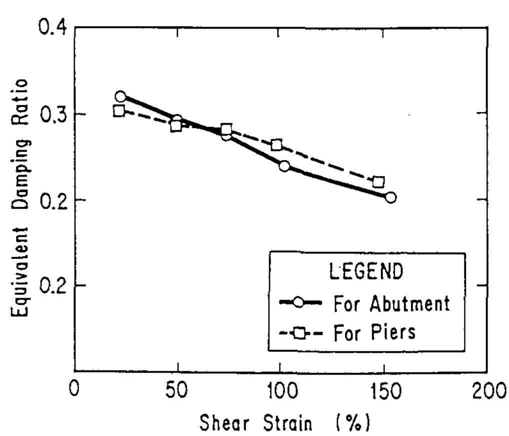

Fig. 5 Equivalent Damping Ratio of Lead Rubber Bearings Based on Full Model Tests

this level, bilinear behavior of LRBs seems not significant. However it should be noted that LRBs show high energy dissipation capability from small displacement. **Fig. 5** shows the equivalent damping ratio of the LRBs obtained from the full size test [7]. Although the equivalent damping ratio for shear strain smaller than 20 % is not presented here, it is known that the equivalent damping ratio for smaller shear range can be approximately evaluated by extrapolating the relation presented in **Fig. 5**. Therefore it may be assumed that the equivalent damping ratio of LRBs was approximately 30 % for the shear strain developed during the earthquake.

4. STUDY OF THE RECORD BY DYNAMIC RESPONSE ANALYSIS

Because dynamic response analysis was made at the design stage for Miyagawa Bridge, it was decided to adopt the same analytical model as shown in **Fig. 6** for studying the record. The acceleration recorded at 10 m below the ground surface was regarded as the input ground motion and the response of the deck and substructure were computed and compared with the recorded accelerations. Because the footings of P1 and P2 are almost on the same level with the down−hole accelerometer placed 10 m below the ground surface (refer to **Fig. 1**), it was considered appropriate to apply the acceleration measured at 10 m below the ground surface to the footings. Although the footing of two abutments, A1 and A2, is slightly higher in level than the down−hole accelerometer, it was disregarded in the analysis because it would cause only minor difference.

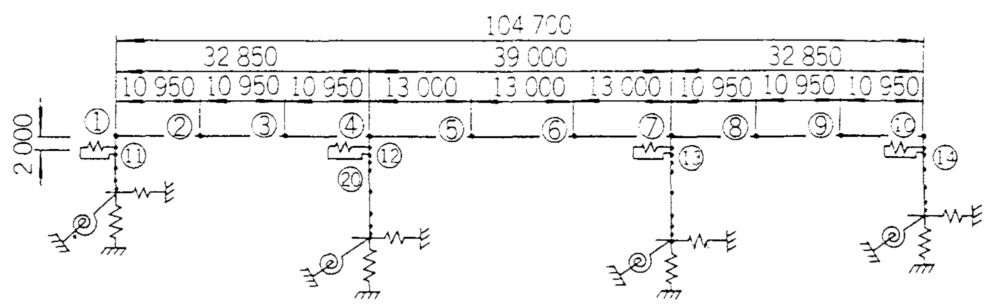

Fig. 6 Dynamic Response Analysis Model of Miyagawa Bridge for Design Purpose

Analysis was made by the equivalent linear analysis. The equivalent linear analysis provides accurate response for the seismic isolated bridges [9]. Because the deformation of the LRBs was quite small during the earthquake, the first stiffness k_1 was assumed as the equivalent stiffness of LRBs as

$$k_1 = 6.5 k_2 \tag{1}$$

$$k_2 = \frac{F - Q_d}{u_B} \tag{2}$$

$$Q_d = A_P \cdot q_0 \tag{3}$$

$$F = A_R \cdot G \cdot \gamma \cdot A_P \cdot q \qquad (4)$$

$$q = -283.6\gamma^2 + 183.8\gamma + 85.0 \qquad (5)$$

where k_1 : 1st stiffness of LRB (kgf/cm), k_2 : 2nd stiffness of LRB (kgf/cm), Q_d : yielding force of lead (kgf), q_0 : shear stress of yielding of lead plug (= 85.0 kgf/cm ^{2}), G : shear modulus of rubber (kgf/cm ^{2}), F : shear force of LRB (kgf), u_B : design displacement (cm), q : shear stress of lead (kgf/cm ^{2}), γ : shear strain, A_P : sectional area of lead plug (cm ^{2}), and A_R : sectional area of rubber (cm ^{2}).

$$h = \frac{\sum_{j=1}^{n} \underline{\phi}_{ij} h_j \underline{k}_j \underline{\phi}_{ij}}{\underline{\Phi}_i^T \underline{K} \underline{\Phi}_i} \qquad (6)$$

where $\underline{\phi}_{ij}$: mode vector of j-th structural component for i-th mode, h_j : damping ratio of j-th structural component , $\underline{k}_j$: stiffness matrix of j-th structural component, $\underline{\Phi}_i$: mode vector of bridge for i-th mode, and $\underline{K}$: stiffness matrix of bridge.

Eq. (6) is effective to evaluate the modal damping ratio of bridges based on an experimental study [8]. Damping ratio of structural components was assumed in design as shown in **Table 2.** Because the deformation developed in LRBs is small during the design earthquake assumed in the Seismic Coefficient Method, it was assumed in the design that the damping ratio of LRBs is zero in the Seismic Coefficient Method. It may be therefore good to assume zero damping ratio for LRBs for the consistency with the seismic design. However it became apparent that zero damping ratio for LRBs gives appreciably larger bridge response than the measured values. Therefore it was assumed as 0.3 based on the above described full model test results on the damping characteristics of LRB.

Fundamental natural period predicted in longitudinal and transverse direction is 0.50 sec and 0.49 sec, respectively. The translation of only the deck is observed in those mode shapes. Because this corresponds to the natural period of about 0.85 second observed in Fig.4, the stiffness of the bridge assumed in the analysis is appreciably higher than that during the earthquake.

Table 2 Damping Ratio Assumed in Seismic Design of Miyagawa Bridge

Structural Components		For Design		For this Analysis
		Seismic Coefficient Method	Bearing Capacity Method	
Super structure		0.03	0.03	0.03
Piers and Abutments		0.05	0.05	0.05
Foundations		0.1	0.1	0.1
Menshin Bearings	A1	0	0.177	0.3
	P1	0	0.137	0.3
	P2	0	0.139	0.3
	A2	0	0.199	0.3

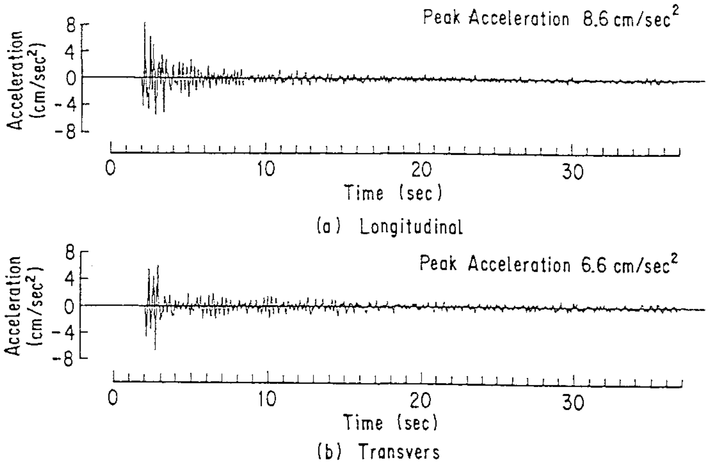

Fig. 7 Computed Deck Acceleration

Fig. 7 shows the response acceleration of the deck thus computed. Although the computed acceleration is 1.5 ~ 1.7 times larger in amplitude than the measured response, overall characteristics of the computed response is similar with the measured acceleration. Overestimation of response acceleration of the deck may be attributed to the fact that the stiffnesse of the model is over estimated and that the damping ratio of the foundation is underestimated in the dynamic analysis. This is for the sake of providing safe-side result for design purpose.

5. CONCLUSION

The first record obtained at the Miyagawa Bridge, the first Menshin bridge in Japan, was analyzed to study the seismic behavior of Menshin bridges. The following conclusions may be deduced based on the analysis presented herein :

1) The deck acceleration is about 50 % in amplitude of the underground acceleration at 10 m below the ground surface. This clearly shows the effectiveness of the seismic isolation.
2) When amplification of the response from the 10 m below the ground surface to the deck is represented in terms of the acceleration response ratio with damping ratio of 0.05, the acceleration response ratio R defined as the response acceleration of the deck motion divided as the response acceleration of ground motion measured at 10 m below the ground surface is only 0.25 at natural period of 0.2 ~ 0.3 second which is the predominant period of the ground motion. However the acceleration response ratio R defined as the response acceleration of the deck motion divided by the response acceleration of ground motion measured at 10 m below the ground surface is 1.4 ~ 2.0 at 0.85 second which is the fundamental natural period of the bridge. Therefore selection of the natural period in consideration with the predominant period of ground motion is very important in the Menshin Design.
3) The dynamic response analysis model which was adopted for the seismic design of Miyagawa Bridge can realistically simulate the response of the deck although the analysis overestimates the measured acceleration.

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