RESPONSE OF ON–NETOH BRIDGE DURING KUSHIRO–OKI EARTHQUAKE OF JANUARY 1993

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ABSTRACT

On–netoh bridge is being constructed in the East of Hokkaido for the replacement of the existing bridge. Menshin design using lead rubber bearings is employed for the On–netoh bridge. In January, 1993, the Kushiro–oki Earthquake with magnitude of 7.8 occurred and it was the first time that a Menshin designed bridge experienced a major earthquake. This paper presents earthquake response characteristics of a Menshin designed bridge during the Kushiro–oki Earthquake. The analysis of acceleration records measured close to the On–netoh bridge and the simulation analysis using the measured record are made.

INTRODUCTION

The On–netoh bridge is now under construction on the national highway No.44 at Nemuro City in Hokkaido[1]. The Menshin design was adopted for the side–span of the On–netoh bridge. Since Hokkaido is located in the North of Japan and is in a cold district, one of the authors have been studied the temperature dependency of the horizontal stiffness of laminated rubber bearings through loading tests of full–size models. It was found that the stiffness of the isolation bearings have the relatively strong temperature dependency[2].

In January, 1993, the Kushiro–oki Earthquake with magnitude of 7.8 on the Richiter scale occurred approximately 100km from the On–netoh bridge as shown in **Fig. 1**. It was the first time that a major earthquake occurred near the Menshin designed bridge and also that it was at a low temperature condition. Although the strong motion observation was not made for the On−netoh bridge, it was obtained at the existing bridge which is located less than 200m from the On−netoh bridge. Earthquake response characteristics of the On−netoh bridge is studied through the analysis of measured acceleration records and the simulation analysis of the On−netoh bridge.

ON−NETOH BRIDGE

The existing bridge was constructed in 1961 on National Highway No.44 at On−netoh in Nemuro, Hokkaido. **Fig. 2** shows the side view of the existing bridge. The bridge length is 96.95m long which consists of a steel Warren−type truss bridge with span length of 60m and a steel composite girder bridge with that of 35m. Width of the deck is 7.5m. Since the existing bridge becomes too old for use and the On−netoh bridge is now being constructed for the replacement of the existing bridge. **Fig. 3** shows the locations of two bridges.

Fig. 4 shows the On−netoh bridge which is constructed by the Hokkaido Development Bureau. Bridge length is 456m long and the bridge consists of 3 four−span continuous plate girder bridges with bridge length of 102.2m, 104.8m and 104.8m and Nielsen−type Lohse girder bridge with span length of 140m. The construction will be completed in 1995. The Menshin design using lead rubber bearings is employed for the side bridge at the Kushiro side. **Photo 1** shows the general view of the bridge. The site is in a cold region where the mean of the lowest temperature for the last 30 years at the site is −9.1℃. Therefore, it is a distinctive characteristics of the bridge that the Menshin design is adopted for a bridge in a cold district.

Rubber bearings are used for the both ends of the girder of the On−netoh bridge, i.e., at A1 abutment and P4 pier. Lead rubber bearings are used at Piers P1, P2 and P3. The stiffness of the bearings are designed so that the distribution ratio of the inertia force of a superstructure in the longitudinal direction be 5% for A1 and P4, and 30% for P1, P2 and P3. On the other hand, the stoppers to prevent displacement in the transverse direction are installed at all of the bearings. **Fig. 5** shows the bearings used for the bridge.

Fig. 6 shows a cross section of the strata of the construction site of the On−netoh bridge. It consists of sand (AS1), silt (AC), sand (AS2) and loam (Nm) layers. The loam layer which is selected as the supporting layer of the foundations is inclined with angle of about 10% from the Kushiro side to the central span. The supporting layer is located at 10m for A1 to 19m for P1 under the sea level.

The On−netoh bridge is designed according to the Design Specifications of Highway Bridges issued in 1980[3]. The seismic lateral force was not decreased considering the damping effects of the Menshin bearings. Table 1 shows the natural periods of substructures and the corresponding seismic lateral force coefficients. **Table 2** shows the design conditions for the Menshin bearings. Menshin design was made according to Guidelines for base Isolation of Highway Bridges[4] and Manual for Menshin Design of

Highway Bridges[5].

Table 1 Natural Period and Lateral Force Coefficient

	Ground Condition	Natural Period (sec)	Lateral Force Coefficient (Kh)
A1	Type I	0.964	0.23
P1	Type II	0.970	0.25
P2	Type II	1.044	0.28
P3	Type II	1.062	0.28

Table 2 Design Conditions of Bearings

		RB	LRB
Maximum Vertical Reaction Force	R (tf)	50.0	135.0
Dead-Load Reaction Force	W (tf)	23.0	92.0
Live-Load Reaction Force	W (tf)	27.0	43.0
Design Displacement (Bearing Capacity Method, Type II Ground)	U_B (cm)		15.0
Equivalent Stiffness	K_B (kgf/cm)	615	2280
Equivalent Damping Ratio	h_{eq}		0.21

Note) RB and LRB represent rubber bearing and lead rubber bearing, respectively

EARTHQUAKE RESPONSE OF ON-NETOH BRIDGE DURING KUSHIRO-OKI EARTHQUAKE OF 1993

The Kushiro-oki Earthquake occurred at 8:06 PM, January 15, 1993. The earthquake was located in the center of eastern Hokkaido and affected over a wide area including Tohoku and Kanto regions. The earthquake registered magnitude of 7.8 on the Richiter scale and the epicenter was reported as approximately 30km off the south coast of Kushiro city at lat. 42° 51' N, long. 144° 23' E. Depth was 107km. A number of aftershocks with epicenters of over 90km deep occurred within one day after the main shock and a few occurred after two days.

Three SMAC-Q type strong motion seismographs are installed at the pier crest, abutment crest and ground surface near the abutment of the existing bridge. Strong motions were successively measured at three seismographs. **Fig. 7** shows the measured acceleration records. Duration time is over 240 seconds. **Table 3** shows the peak acceleration measured at the ground surface and the abutment. Peak accelerations of the ground surface are 341.5cm/sec^2 in the longitudinal direction and 363.1cm/sec^2 in the transverse direction. The peak acceleration of UD component is 114.4cm/sec^2 which corresponds to about 33% of horizontal peak accelerations. Peak accelerations of the abutment in the longitudinal and transverse directions are about 425cm/sec^2 and 300cm/sec^2, respectively. **Fig. 8** shows acceleration response spectrum of the measured strong motion at the ground surface. The analysis was made for the main shaking of 40 seconds from the time of 90 second to 130 second. The predominant period of the response spectrum of horizontal components was found to be in the range of 0.5 to 4.0 second, which contains relatively long period components and the peak is around 0.8 second. The predominant period of UD component is less than 1 sec.

Table 3 Measured Peak Acceleration (cm/sec^2)

	LG	TR	UD
Ground	342	363	114
Abutment	425	300	–

Photo 2 shows the situation of the bearings of the On–netoh bridge after the earthquake. The rubbing between the sole plate of the bearing and the side stopper occurred and the coming off of the painting was found. According to the marks, peak relative displacement of the bearings is estimated as about 4 to 5cm. Larger displacement was found at Pier P4 than the other piers and abutment. No damage or mark were found at expansion joints or hand rails at A1 abutment side.

RESPONSE ANALYSIS OF THE ON–NETOH BRIDGE

Nonlinear time history response analysis for the On–netoh bridge was made. Direct integration method using Newmark β algorithm was used for the analysis. Reyleigh–type damping was assumed to represent the damping characteristics of the bridge. **Fig. 9** shows the analytical idealization of the On–netoh bridge. It was assumed that the piers be in the elastic range and that nonlinear effect of Menshin bearings be expressed as a bi–linear model.

The foundations of A1, P1～P3, which are the pile foundation, are modelled as a linear spring model. On the other hand, that of P4, which is a direct foundation, is assumed as fixed. The characteristics of the bearings used for the analysis are shown in **Table 4**. Although the mean and lowest temperature in the Nemuro region at the time of the earthquake were reported as –4.8℃ and –7.9℃, respectively, the stiffness of Menshin bearings at the temperature of 20℃ and –20℃ was assumed in this analysis. Stiffness of the Menshin bearings at those temperature condition were computed based on the loading tests of bearings. Analysis was made for the longitudinal direction which corresponded to the direction of Menshin design. The strong motion data with duration time of 120 seconds between the time of 50 second to 170 second was used as an input ground motion.

Table 4 Characteristics of Bearings for Analysis

Bearings	Characteristics		20°	−20°
RB	Stiffness	K_{RB} (tf/m)	363.0	472.0
LRB	First Stiffness	K_1 (tf/m)	5188.0	8953.2
	Second Stiffness	K_2 (tf/m)	471.1	645.0
	Ratio of Stiffness	K_2/K_1	0.07212	0.07204
	Yield Displacement	δ (m)	0.01832	0.01244

Note) RB and LRB represent rubber bearing and lead rubber bearing, respectively

Fig. 10 shows the time histories of acceleration and displacement of deck and relative displacement of bearing at pier P2 when the temperature is assumed as −20℃. The responses of the deck were not elongated comparing with the ground motion because the stiffness of the bearings become relatively large in the cold district. Peak deck acceleration is computed as 505cm/sec^2. Amplification of the ground motion is estimated as 1.48. However, the response acceleration of the deck seems to be decreased during the time of 45 to 60 second when the excitation of the strong ground motion. The response of deck displacement and relative displacement of bearings are almost the same and the peak response occurred around the peak of the input ground motion. The residual displacement with about 2mm is found at the bearings.

Table 5 shows the computed peak bending moment at the bottom of piers. Although the peak bending moment at the bottom of pier P1 to P3 is in the range of 1,200 to 1,500tf·m when the temperature is assumed as 20℃, on the other hand, they become twice when the temperature is assumed as −20℃. However, those for Abutment A1 and Pier P4 do not change drastically depending on the temperature, because the variation of the stiffness of rubber bearings with temperature is relatively small. Comparing the bending moment of the piers between computed through the nonlinear analysis and designed, the computed bending moment is about twice the designed bending moment.

Relative displacement of bearings are also shown in **Table 5**. As mentioned before, the peak relative displacement was estimated as 4 to 5cm from the observation of the bridge after the Earthquake, however, that computed with assumption of temperature of −20℃ is about 8 to 9cm. Therefore, relative displacement of expansion joints is also estimated as 8 to 9cm. Design displacement according to seismic lateral force coefficient method is 5.5 to 6.6cm. Based on the analysis, it is estimated that the inertia force beyond the design seismic lateral force acted to the piers. However, the actual bridge response is much smaller than the computed one.

Table 5 Computed Peak Response

	Peak Bending Moment at the Base of Pier (tfm)			Peak Relative Displacement of Bearing (cm)
	20°	−20°	design value	−20°
A1	485.7	475.8		
P1	1175.6	2227.5	806.0	9.0
P2	1295.2	2464.8	1053.0	8.6
P3	1528.1	2733.9	1203.0	8.0
P4	58488	60157	19129	

CONCLUSIONS

The On−netoh bridge, which employs the Menshin design in the cold and snowy district at Nemuro, Hokkaido, experienced the Kushiro−oki Earthquake of 1993. The peak acceleration measured on the ground surface is over 350cm/sec^2. The analytical simulation using the measured ground motion shows that the bending moment at the bottom of pier is estimated to be over the designed one. The effect of the earthquake may be small and the seismic isolation may be successively functioned.

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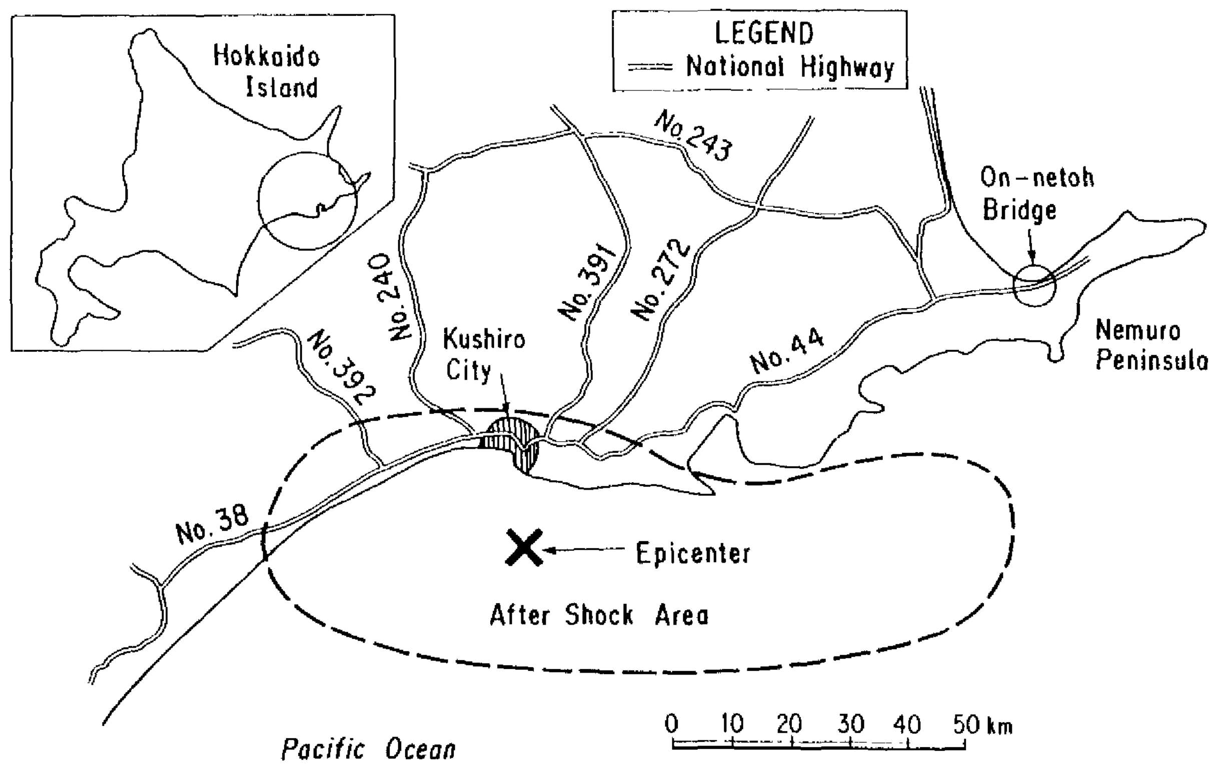

Fig. 1 Locations of the On–netoh Bridge and the Earthquake Epicenter

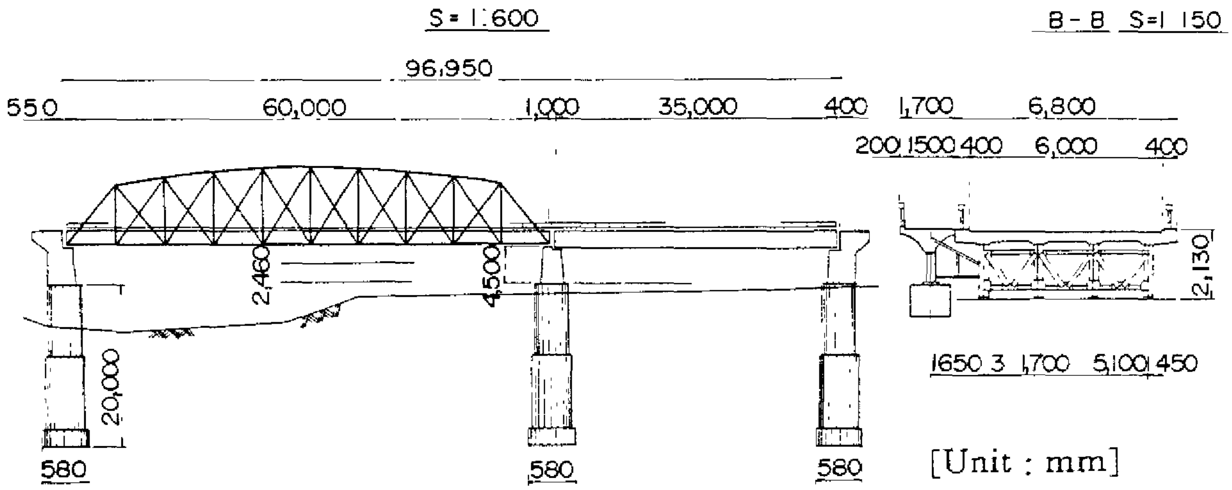

Fig. 2 Side View of the Existing Bridge

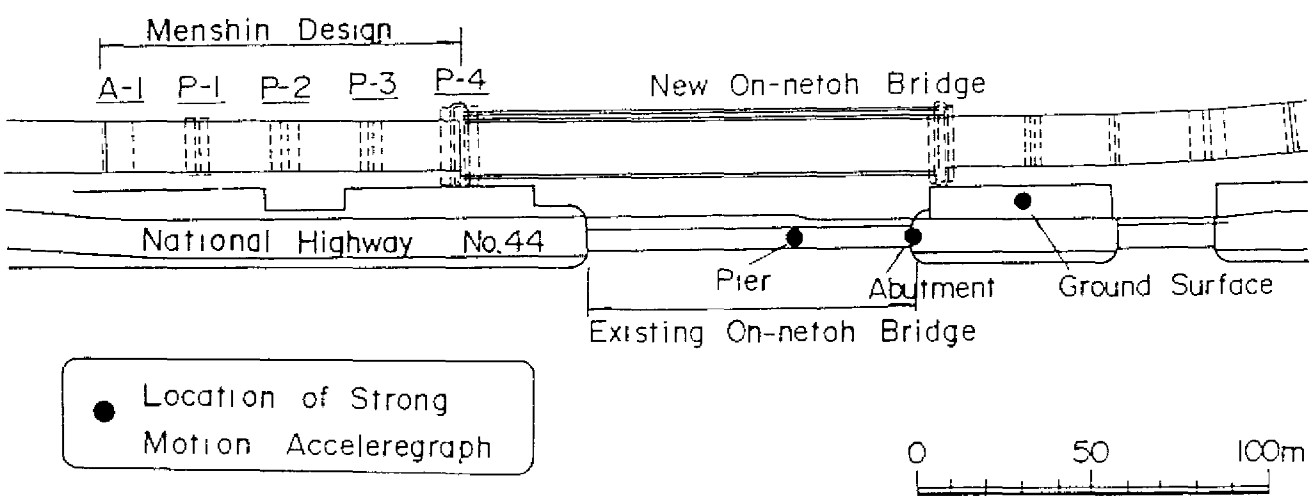

Fig. 3 Locations of two Bridges

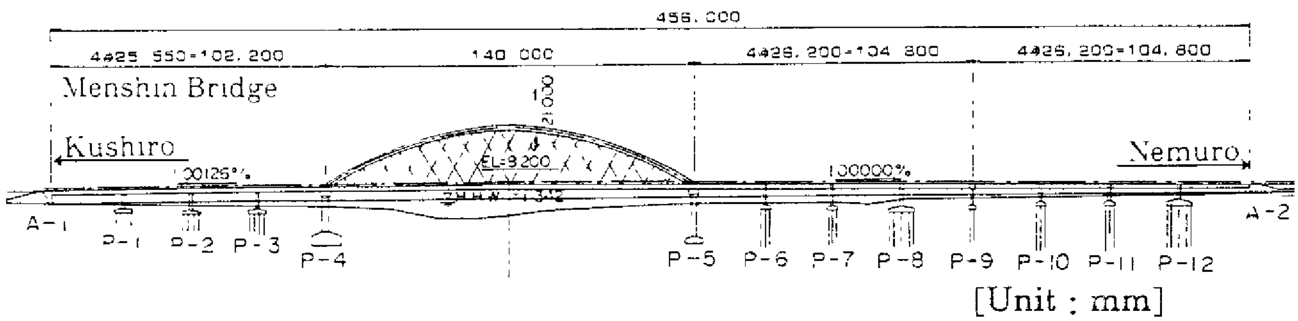

Fig. 4 Side View of the On−netoh Bridge

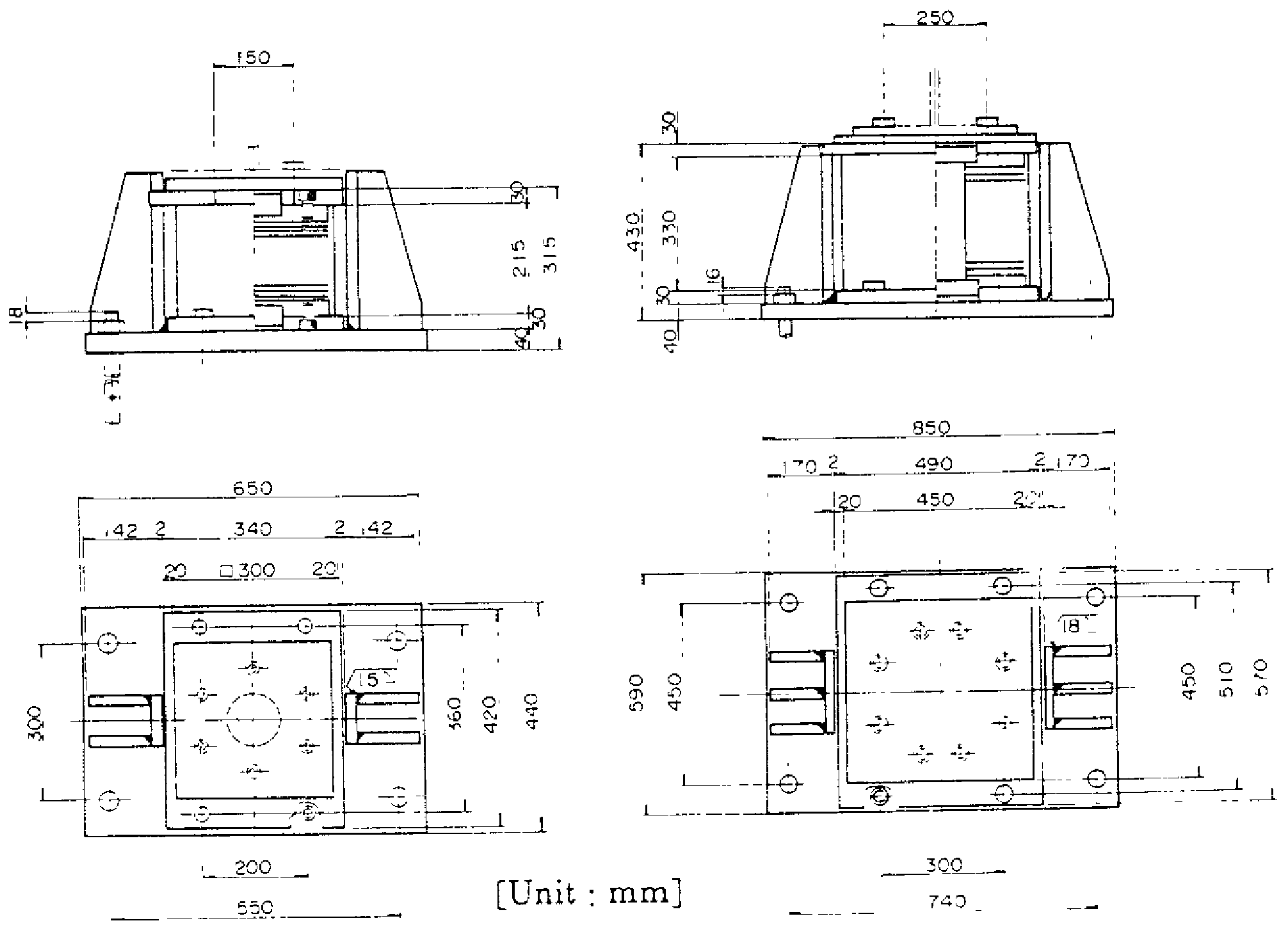

Fig. 5 Bearings used for the On−netoh Bridge

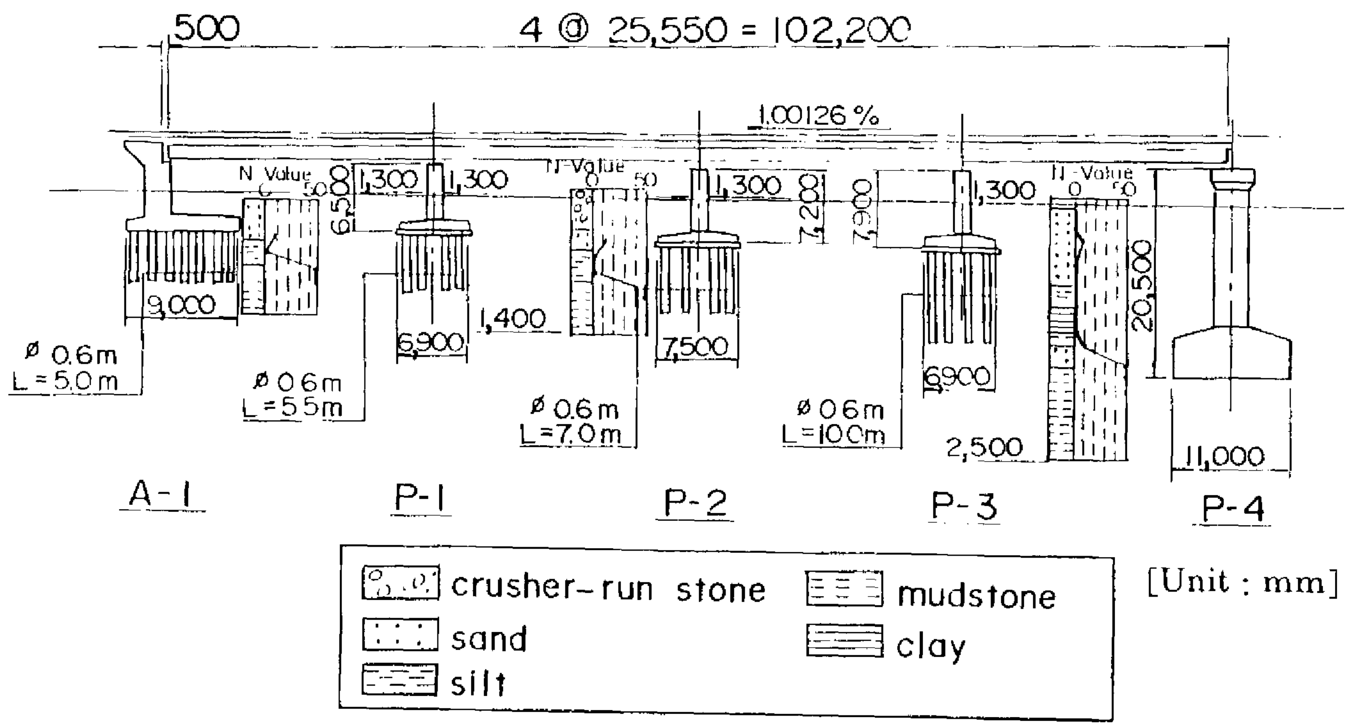

Fig. 6 Ground Condition of the Or- metoh Bridge

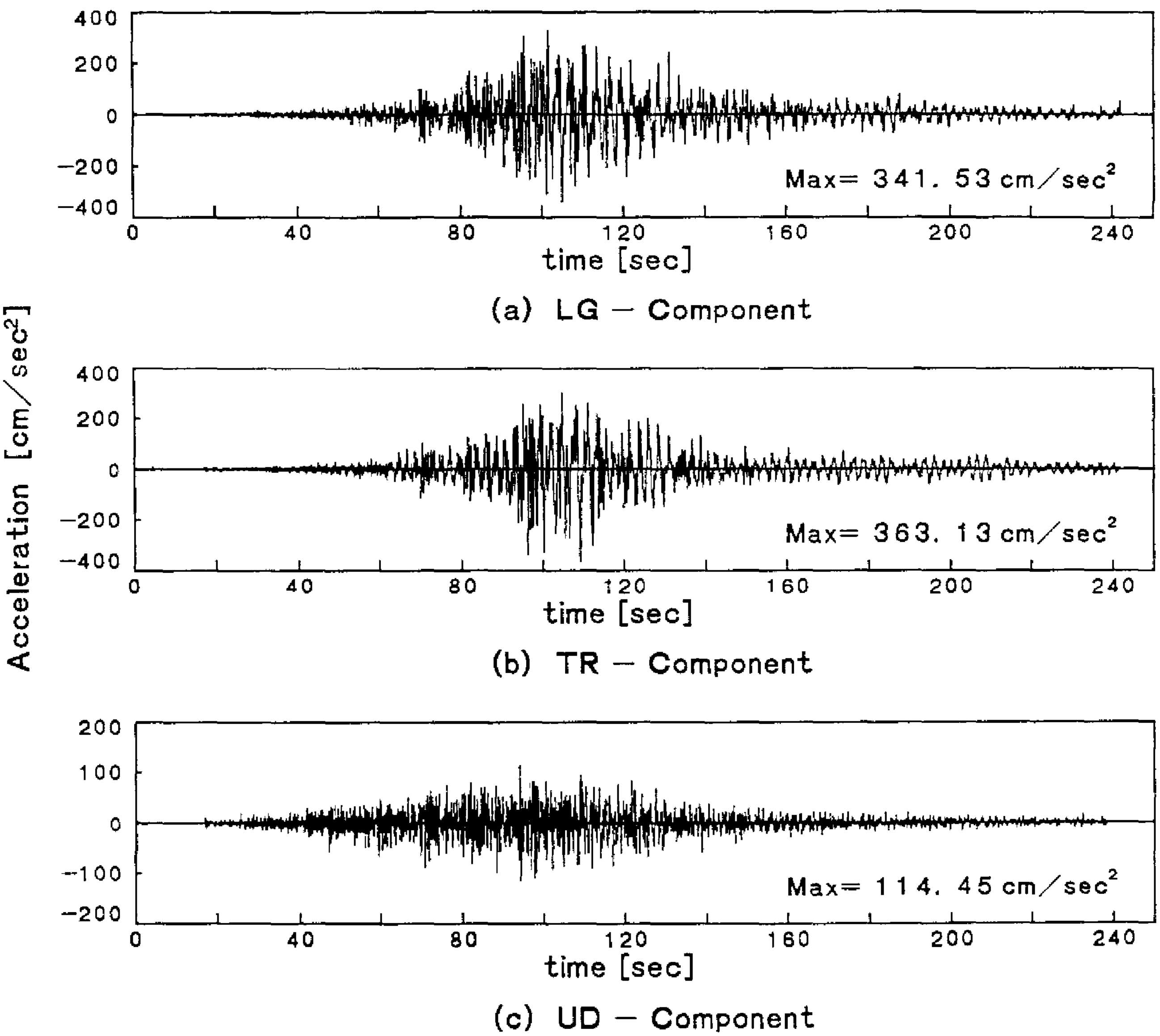

Fig. 7 Strong Motion Record on the Ground Surface

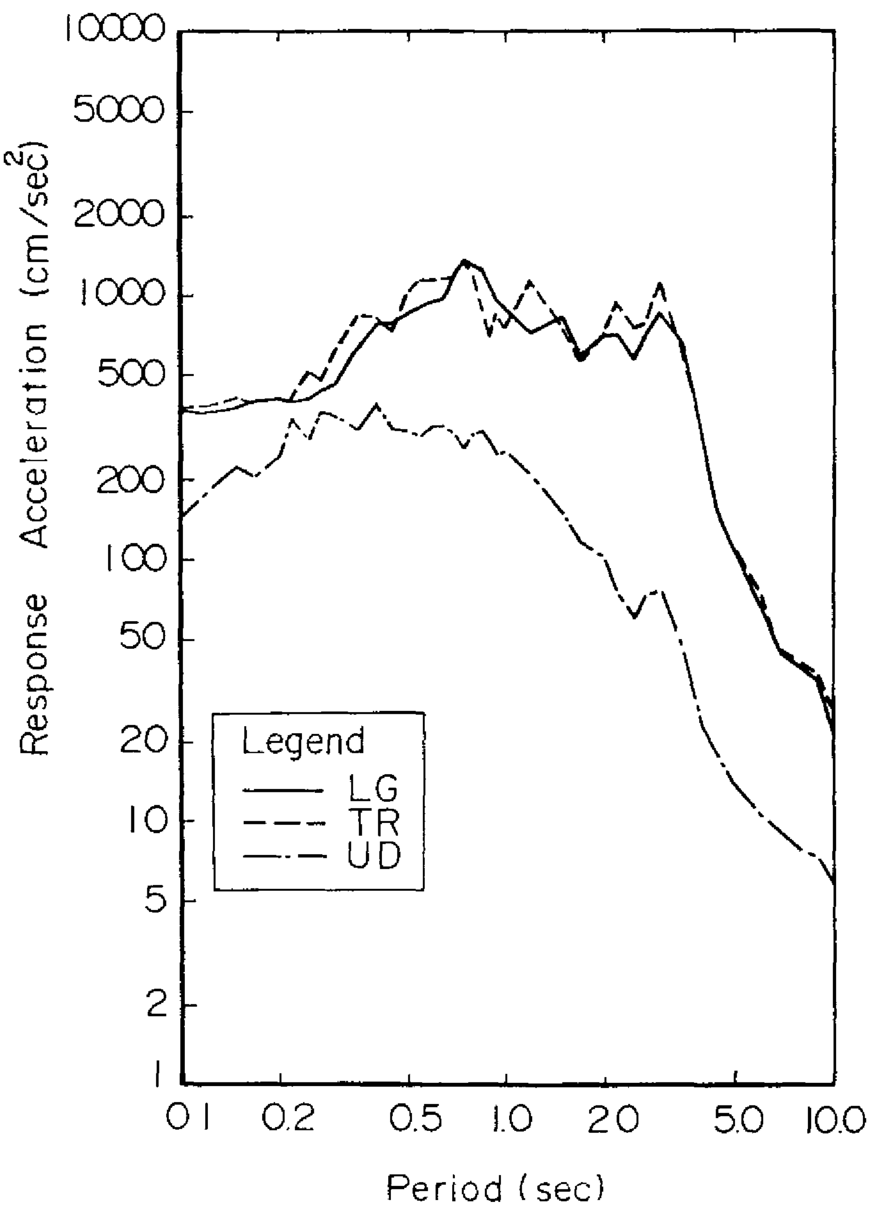

Fig. 8 Acceleration Response Spectrum

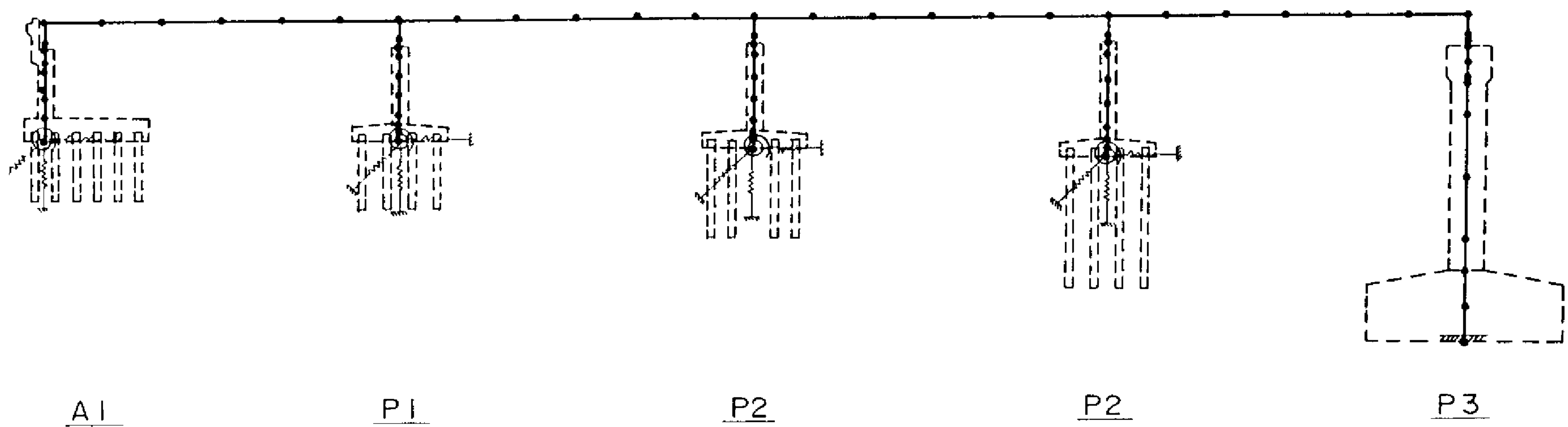

Fig. 9 Analytical Idealization

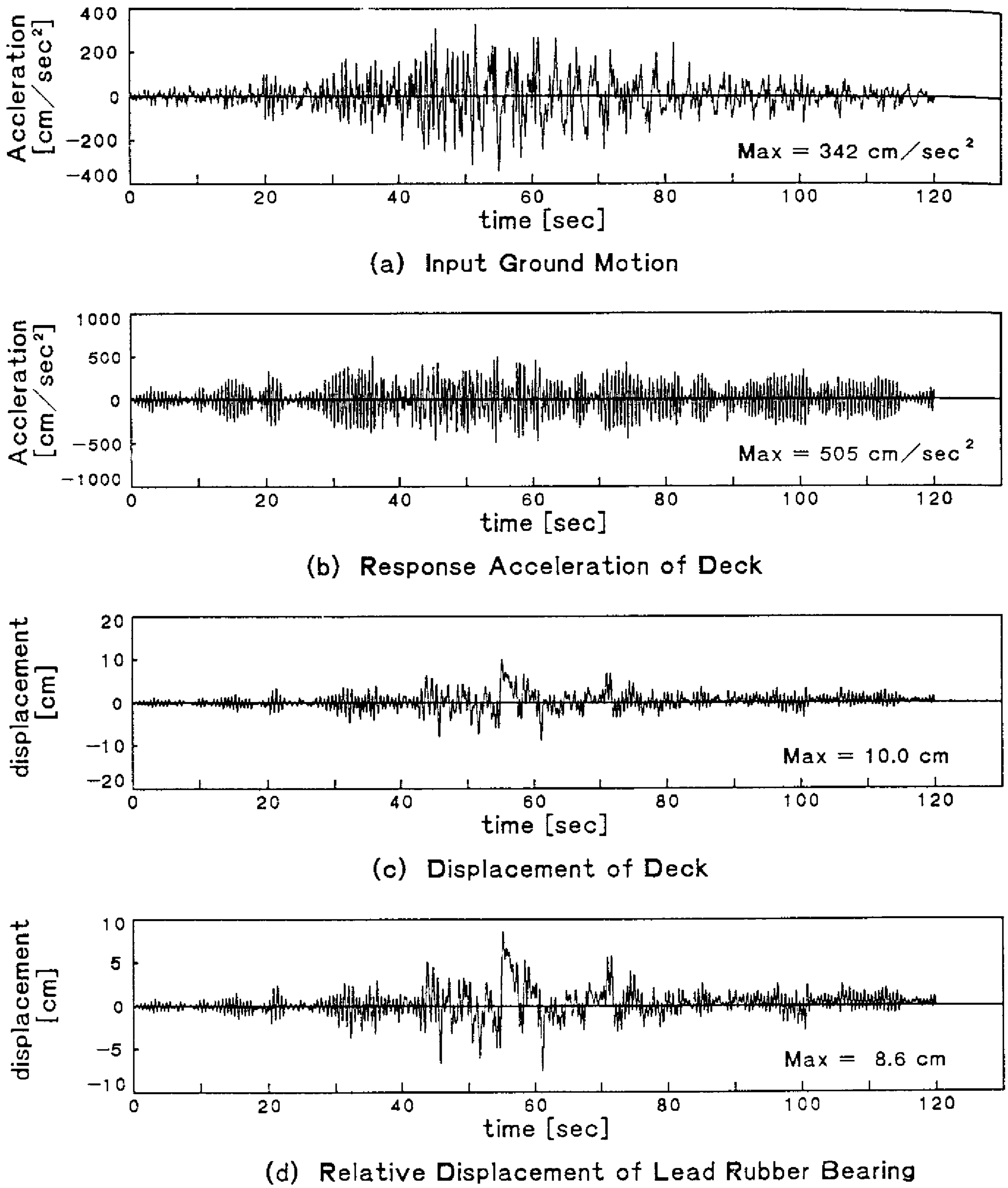

Fig. 10 Time Histories computed by Simulation Analysis

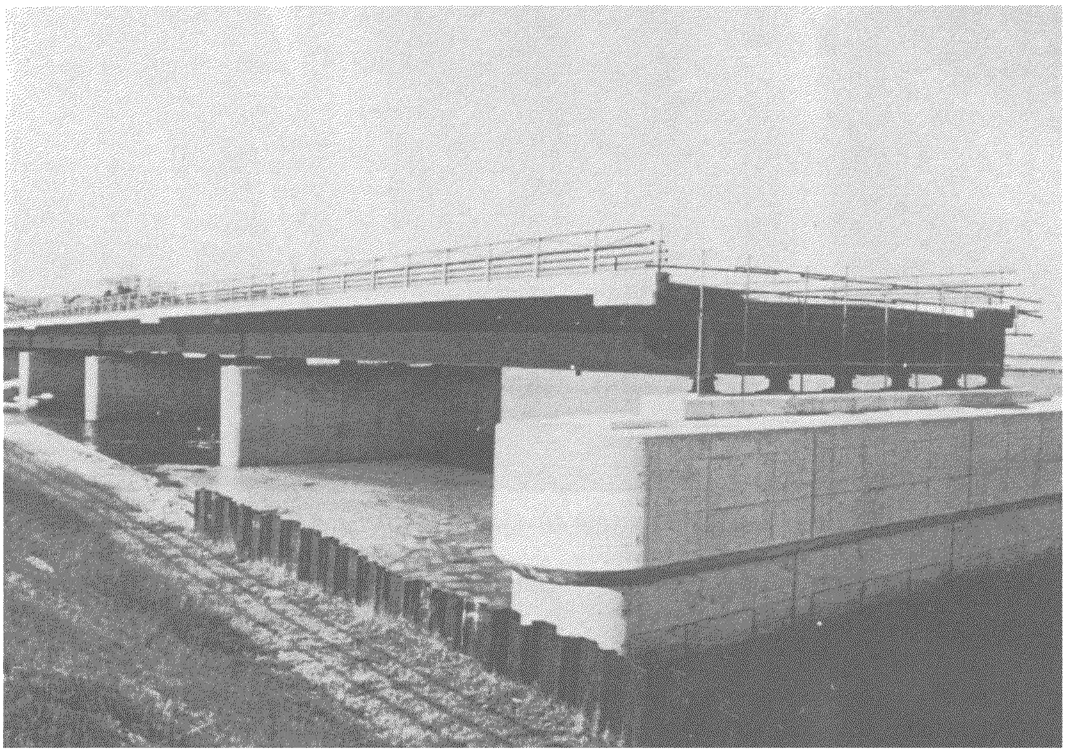

Photo 1 On−netoh Bridge