INSPECTION OF SEISMIC RESPONSE ANALYSIS OF CONTINUOUS GIRDER BRIDGES USING STRONG-MOTION SEISMOGRAM

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Abstract

In order to clearly specify a technique that uses a dynamic verification method to more accurately carry out the seismic design of continuous girder bridges, the Public Works Research Institute (PWRI) for Cold Region has examined the modeling method for seismic response analysis, focusing on the girder bridges of which strong-motion seismograms were obtained in the past. This paper describes the summary of this examination. One of two girder bridges used for this examination is the Onnetô Bridge, which experienced the 1994 Hokkaido Touhou-oki earthquake, while the other is the Tokachi-kakou Bridge, which experienced the 2003 Tokachi-oki earthquake.

Introduction

Recently, a growing number of bridges in Japan are making use of rubber bearings, etc. For the seismic design of continuous girder bridges, the use of a dynamic verification method using seismic response analysis, taking into account material non-linearity, has become the mainstream [1]. However, only a few case studies have examined the accuracy level of the seismic performance of an entire bridge, as evaluated using seismic response analysis.

For this reason, we performed an accuracy verification of the seismic response analysis of the Onnetô Bridge (a seismic isolation bridge, Figure 1) of which strong-motion seismograms were obtained during the 1994 Hokkaido Touhou-oki earthquake. In addition, using the strong-motion seismograms of the Tokachi-kakou Bridge (a PC continuous girder bridge, Figure 2) of which bearings were damaged by the 2003 Tokachi-oki earthquake, we simulated the damage of this bridge.

The Outline of Onnetô Bridge

Photo 1 shows a panorama of the Onnetô Bridge used for this examination. This bridge is composed of a Nielsen Lohse bridge for the main span and four-span continuous plate girder bridges for the side spans. The bridge uses isolation bearings and LRBs for the side spans. The isolation bearings act only in the longitudinal direction, and there is a clearance of 2 mm on one side in the transverse direction. These bearings are interfered with by the side blocks. The ground classification from A1 to P1 is Type I, while from P2 to P4 it is of Type II.

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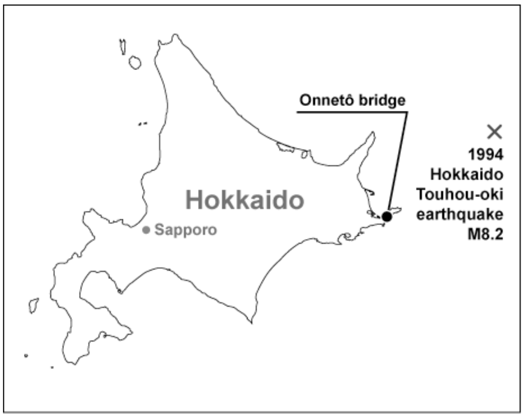


Fig.1 Location of Onnetô Bridge

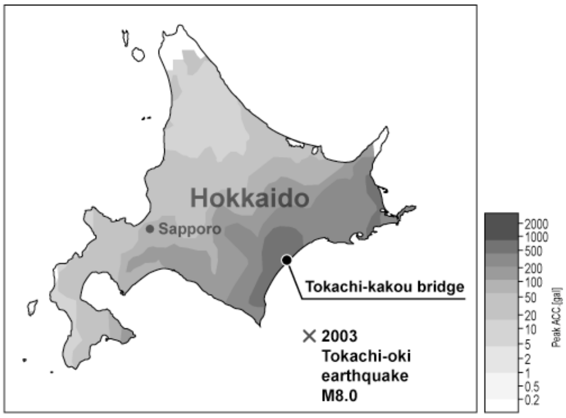


Fig.2 Location of Tokachi-kakou Bridge

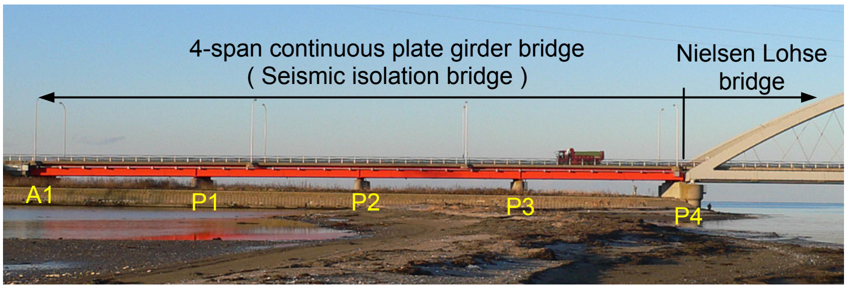


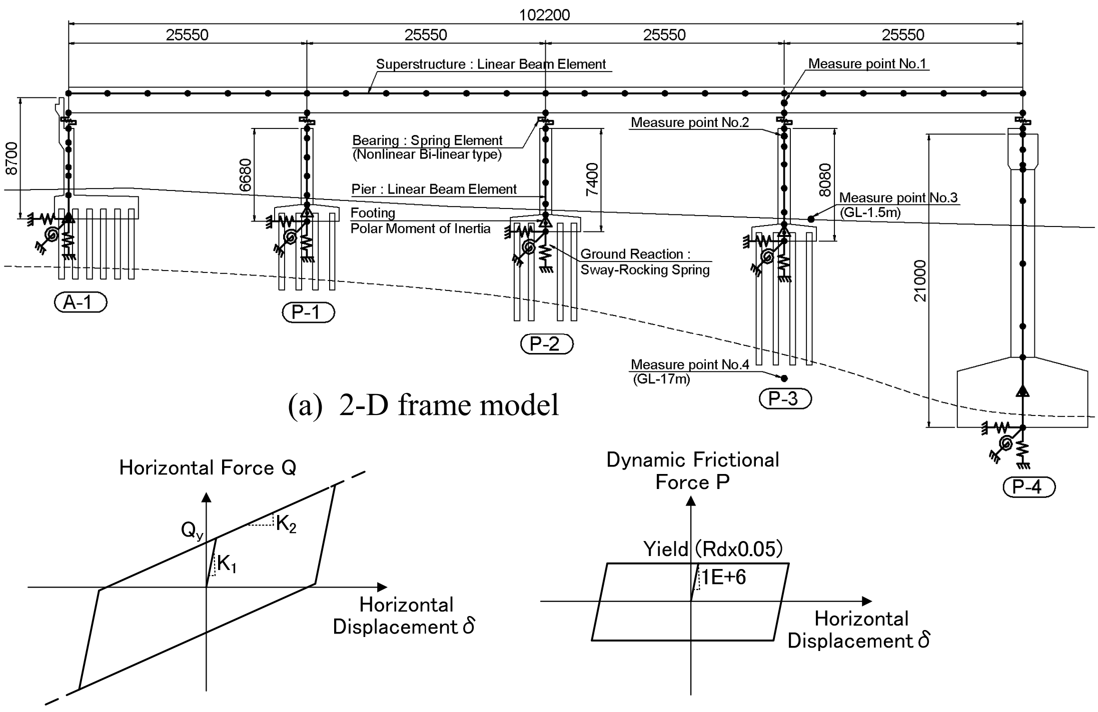
Photo.1 Panorama of Onnetô Bridge

The seismographs are installed on four points on the P3 pier; inside the girder, on the top of the pier, and in the ground 1.5 m and 17 m below ground level. When the bridge was affected by the 1994 Hokkaido Touhou-oki earthquake, only the zones using the isolation bearings were in the same condition as they were when constructed, and strong-motion seismograms were recorded during this earthquake.

Analytical model of Onnetô Bridge

Two analytical models were prepared for this examination. Figure 3 shows a two-dimensional frame model, which is one of two models. In this paper, this model is referred to as Model A. The superstructure and the piers are modeled with an elastic beam element, and the isolation bearings are modeled with nonlinear, bi-linear spring elements. The bearing sections also use nonlinear springs, which simulate the friction resistance caused by the collision between the isolation bearing and the side block. The modeling of the system from the pile foundation to the ground utilizes an S-R model replaced with a sway-locking liner spring, which is often used for actual design work. The acceleration waveform in the longitudinal direction recorded 1.5 m below ground level (the No. 3 measuring point) is used for the input earthquake motion.

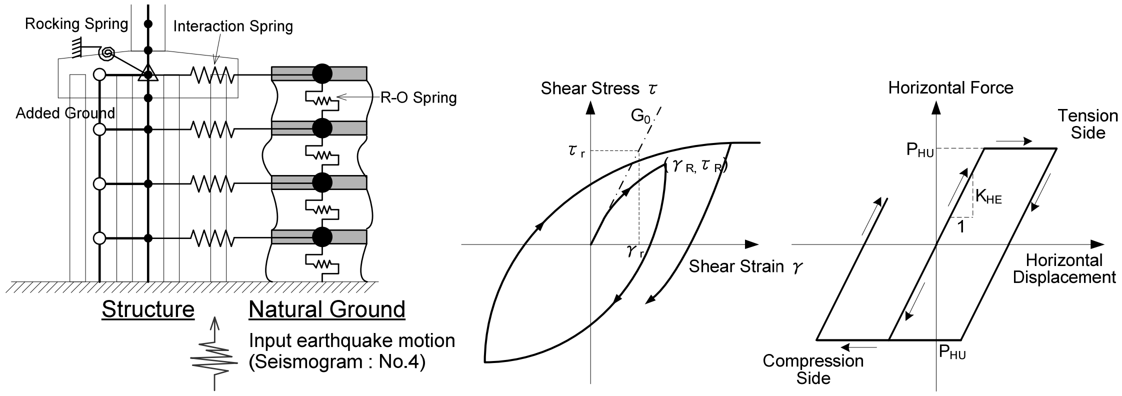
Another analytical model is the two-dimensional frame-coupled model (Figure 4a), which takes into account dynamic interaction with the natural ground. In this paper, this model is referred to as Model B. Although the bridge model is the almost same as that shown in Figure 3, the piles are modeled with elastic beam elements, and is connected to the natural ground via interaction springs. The natural ground is a one-dimensional model composed of a shear spring and the natural ground's mass. The shear spring is the modified Ramberg-Osgood (R-O) model, which takes into account the non-linearity of the ground, as shown in Figure 4b. Regarding the strain dependence properties of the ground, a laboratory cyclic triaxial test on this soil layer was not conducted, thus a stiffness decrease curve $G/G_0 - \gamma$ and a damping increase curve $h - \gamma$—described in the references [2]—were used for preparing the modified R-O model. The mass of the natural ground is estimated to be 200 times that of the mass of the footing. The interaction spring connecting the pile foundation and the natural ground is the perfect bi-linear model of which the upper limit is the passive earth pressure capacity (Figure 4c). The acceleration waveform recorded on the base ground surface in the ground 17 m below ground level (the No. 4 measuring point) is used for the input earthquake motion.



(a) 2-D frame model

(b) Isolation bearing spring (c) Friction spring by interfering with the side

Fig.3 2-D frame model (Model A)



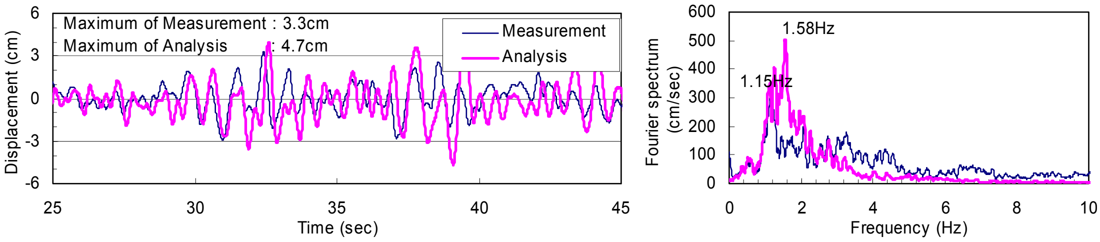
(a) Coupled model (b) Modified R-O model (c) Interaction spring

Fig.4 2-D frame ground coupled model (Model B)

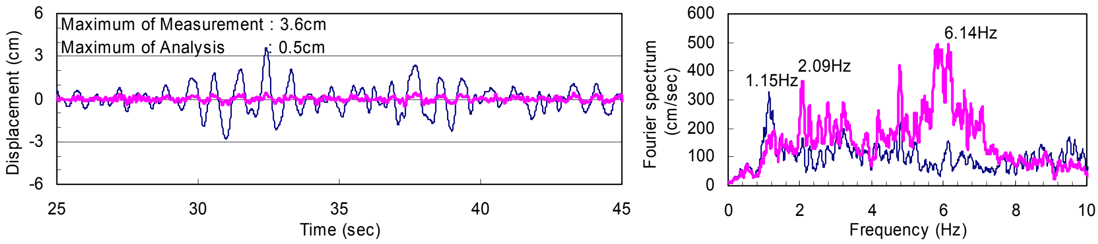
Lastly, Rayleigh damping is used for the damping in the dynamic analysis of these two models.

<u>Comparison between the Analysis Results and Strong-motion Seismograms</u>

Figure 5 shows a comparison between the strong-motion seismograms and the response waveform of the P3 pier, obtained from the seismic response analysis of Model A. To make the comparison clearer, this figure shows the seismic response of only 25 to 45 seconds when the maximum value of seismic response occurs. The comparison of the acceleration spectrums of the pier top shows that the measured value becomes great at 1.15 Hz, which is the same as the superstructure. However, the analysis value of Model A becomes great in the high-frequency region, such as at 2.09 Hz and 5.62 Hz, which means that the measured value and analysis value show a clearly different tendency. Regarding the displacement waveform of the pier top, the measured value is nearly equal in displacement to that of the superstructure; however, the displacement waveform of the analysis value of the pier top is smaller than that of the superstructure, due to the effect of seismic isolation. It can be said that the analysis of Model A fails to perfectly show the actual vibration characteristics.



(a) Superstructure (Measuring point No.1)

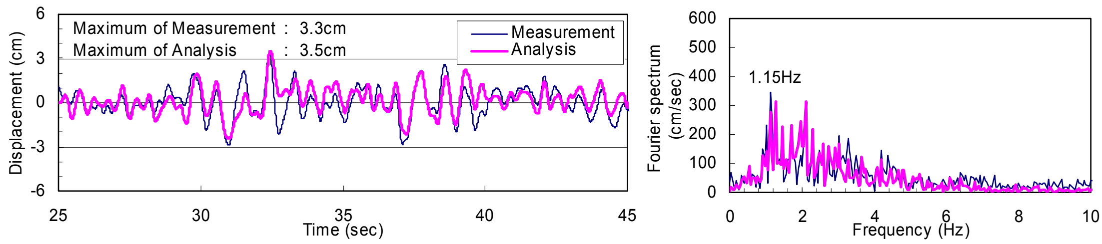


(b) Top of the P3 (Measuring point No.2)

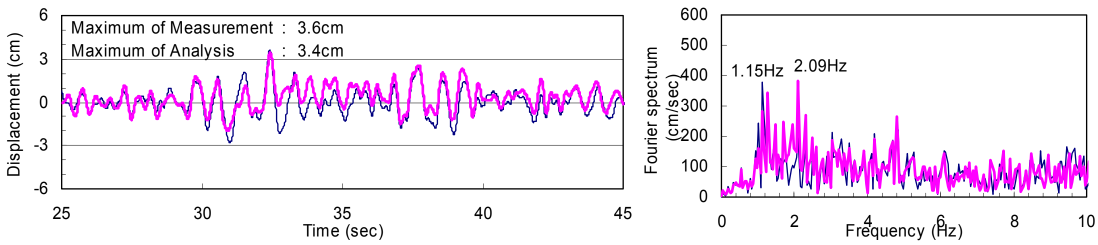
Fig.5 Comparison of the displacement time series and acceleration spectrum (Model A)

Figure 6 shows a comparison between the strong-motion seismograms and the analysis values of Model B, which takes into account dynamic interaction with the natural ground. The acceleration spectrum of the pier top shows that the analysis value in the range from 2–3 Hz is slightly greater than the measured value; however, the analysis value is consistent with the measured value up to the higher frequency zone. Consequently, it can be said that the effect of dynamic interaction with the pile foundation, as well as the vibration transfer characteristics of the bridge, are simulated accurately. Regarding the displacement waveforms of both the superstructure and the pier top, the analysis values of Model B are consistent with the measured values.

As a result, it was verified that the analysis values can be consistent with the strong-motion seismograms by means of taking into account both the non-linearity of the ground and the dynamic interaction between the ground and the bridge.



(a) Superstructure (Measuring point No.1)



(b) Top of the P3 (Measuring point No.2)

Fig.6 Comparison of the displacement time series and acceleration spectrum (Model B)

The Outline of Tokachi-kakou Bridge

The Tokachi-kakou Bridge is composed of the main bridge, which is a three-span hinged PC rigid frame-box girder bridge, and the side bridges, which utilize three-span continuous PC box girders × 3. The piers are oval-shaped RC wall-type piers. The liner plate and the horizontal roller of the steel bearing of this bridge were damaged by the 2003 Tokachi-oki earthquake (M8.0) (Photo 2). In addition, residual displacement up to 67 cm in the transverse direction occurred at the girder edge, and a displacement up to 10 cm occurred on the surface of the bridge [3]. Further, the accelerometers are installed at six points on this bridge: in the girders of the P4 and P5 piers, on the center hinge, on the A2 abutment, and in the ground 5 m and 50 m below ground level. These accelerometers provided the valuable strong-motion seismograms of the Tokachi-oki earthquake.

Photo.2 Damage situation of Tokachi-kakou Bridge

Examination of Analytical Models Using Strong-motion Seismograms

The analytical models used for the examination consisted of three-dimensional frame models (Figure 7) generally used for actual design work. The superstructure is modeled with an elastic girder element that passes through the neutral axis of the PC box girder. The understructure is the tri-liner M-φ model. For the model of the steel bearing, we used a perfect elasticity and plasticity bi-linear model of which the yield point is the design shear strength (hereinafter referred to as “Model A”), and a bi-liner-type slipping model that can take into account slippage after destruction (hereinafter referred to as Model B). Using these two analytical models, we reproduced and analyzed the damage situations. The vibration was applied in the transverse direction, taking into account actual damage. For the input earthquake motion, we used the acceleration waveform that was recorded during the Tokachi-oki earthquake and took measurements at a point 5.0 m below ground level in the middle between P11 and A2 of the Tokachi-kakou Bridge.

Figure 8 shows a comparison between the girder residual displacement measured at the bridge immediately after the earthquake and that obtained by analysis.

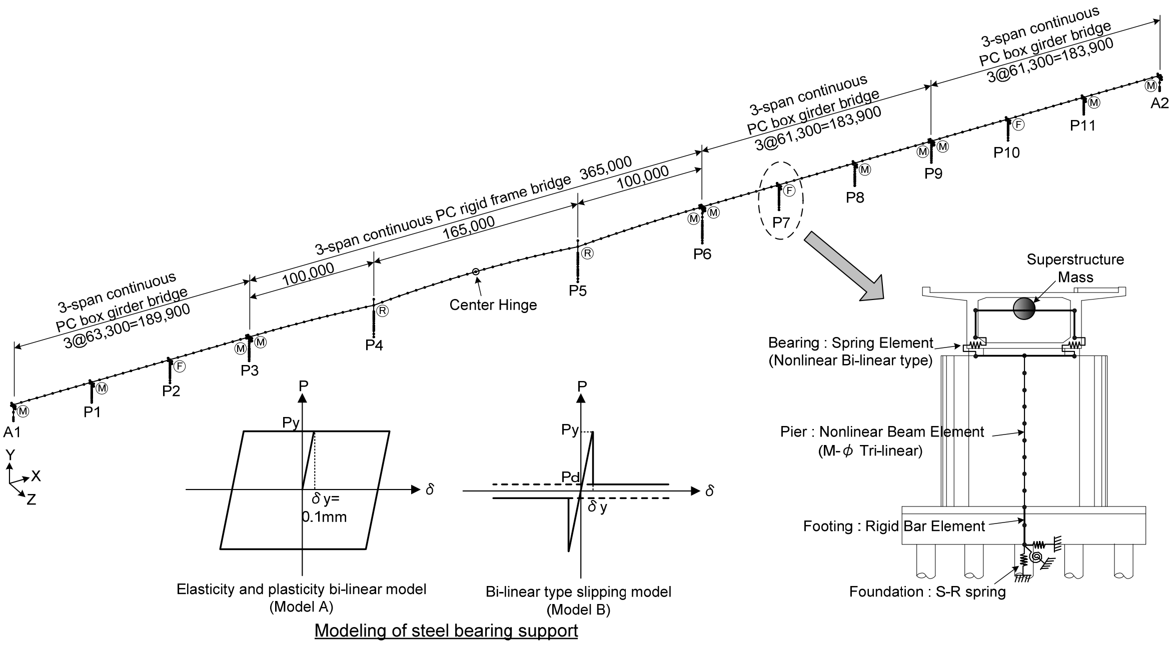


Fig.7 3-D frame model of Tokachi-kakou bridge

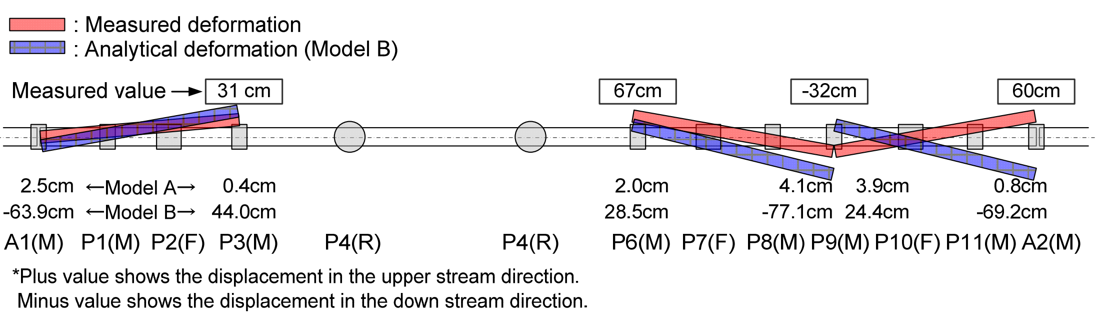


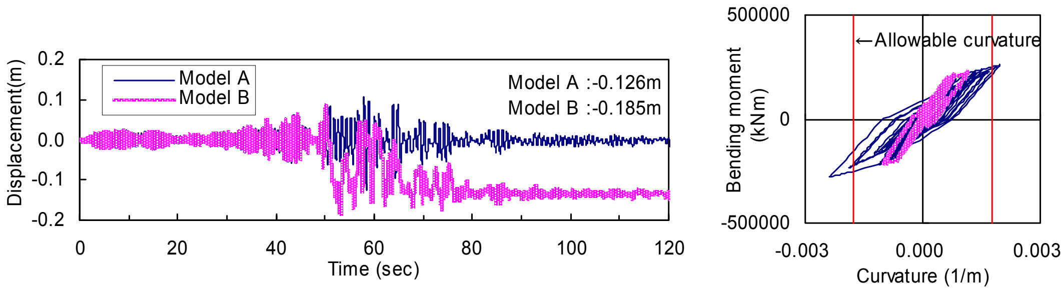
Fig.8 Comparison of residual displacement between measured and analyzed results

While Model A failed to show girder slippage because the strength of the bearing did not decrease after the bearing was broken, Model B reproduced the residual displacement of the site even though it showed a reversal of displacement direction at par with the side bridge.

Seismic Performance Verification of the Pier

Using the abovementioned two models, we verified seismic performance against a Level 2 earthquake motion. For the input earthquake motion, we used the Hokkaido scenario wave that was equivalent to Type I earthquake motion. Figure 9 shows a comparison of the analysis results between Models A and B, and Table 1 shows a comparison of the seismic performance verification results of the pier bases. The verification of Model B in the longitudinal direction shows that the maximum curvatures means that those piers satisfy the seismic performance verification.

During the verification in the transverse direction, the use of Model B allowed the inertial force acting on the understructure to reach its peak when the inertial force becomes twice the strength of the bearing; and therefore, some piers satisfied shear at the secured bases of P2, P7, and P10 are below the allowable curvatures, which force verification. In fact, during the 2003 Tokachi-oki earthquake, it was considered that the



(a) Time series of displacement (Main girder on P6)
(b) Relation of Moment and curvature (Column's bottom of P7)

Fig.9 Comparison of analyzed results of model cases (Level 2 earthquake motion in the longitudinal direction)

Table.1 Performance verification result of Pier (Level 2 earthquake motion)

Pier No.	Longitudinal direction				Transverse direction			
	Maximum of curvature		Maximum of shear force (kN)		Maximum of curvature		Maximum of shear force (kN)	
	Model A	Model B	Model A	Model B	Model A	Model B	Model A	Model B
P1 (M)	0.00045	0.00045	5771	5771	0.00046	0.00002	11571	8913
P2 (F)	0.00244	0.00124	21247	18094	0.00014	0.00004	22323	12203
P3 (M)	0.00035	0.00035	8556	8609	0.00043	0.00015	15015	12227
P4 (R)	0.00069	0.00064	62216	65649	0.00039	0.00023	44893	33386
P5 (R)	0.00064	0.00069	51890	52905	0.00039	0.00023	40764	31062
P6 (M)	0.00119	0.00116	11664	11838	0.00034	0.00014	18579	13966
P7 (F)	0.00224	0.00140	20605	18799	0.00012	0.00006	19588	14057
P8 (M)	0.00088	0.00088	5605	5605	0.00051	0.00005	11289	8820
P9 (M)	0.00074	0.00075	7061	7059	0.00031	0.00003	12042	9819
P10 (F)	0.00253	0.00098	21343	17530	0.00011	0.00005	19720	13129
P11 (M)	0.00008	0.00008	6444	6444	0.00032	0.00001	13459	7898

* The hatching cells show the pier that exceeded allowable value.

piers escaped earthquake damage because bearings were broken.

Conclusion

Through the examination of the Onnetô Bridge, we were able to verify consistency with the strong-motion seismograms by taking into account both the non-linearity of the ground and the dynamic interaction between the ground and the bridge. In addition, the model that took into account the steel bearing failure of the Tokachi-kakou Bridge showed the possibilities of both reproducing earthquake damage on an actual bridge and reducing the scale of the seismic retrofitting of piers.

For the purposes of accurately estimating the behavior of the existing bridges during an earthquake, as well as for determining the damaged sections on a desk immediately after an earthquake, the PWRI for Cold Region is now preparing a manual for modeling methods, analysis methods, and damage determination methods.

References

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[3] 2003 Tokachi-oki Earthquake Review Meeting Report, October 2004 (in Japanese).