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ELABORATE SIMULATION AND PREDICTION OF SEISMIC BEHAVIOR OF A TRUSS BRIDGE

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ABSTRACT

In this study, the time-history analysis procedure is elaborately carried out for a truss bridge by identifying the actual structural vibrating characteristics measured by on-site tests and employing the input ground motions at different sites converted from the motions recorded at other observation spot. This procedure is first validated by simulating the seismic behavior of a deck-type truss bridge subjected to a moderate earthquake (M5.7) happened near the bridge in 2002. Then an extreme earthquake (M7.5) is assumed on the basis of ground motions recorded in the moderate earthquake, and the corresponding seismic responses of the bridge are predicted, by consistently estimating the ground motions at hypocenter with assumption of the ground faults and formulating ground motions at the site with consideration of spreading procedure from the hypocenter.

INTRODUCTION

After the 1995 Kobe earthquake, many research efforts in Japan are attracted on the improvement of seismic design methodology based on dynamic analysis, and consequently some design procedures as well as practical analysis programs are proposed or improved for steel bridges (e.g., References [1]-[3]). Furthermore, in the latest Japan road design codes updated in 2002 (JRA 2002 [4]), the dynamic analysis is required for many practical design cases.

For such dynamic analysis, structural data such as stiffness and masses are formulated on design drawings or design statements. Moreover, the damping is assumed on this basis and the standard seismic ground motions specified in codes are employed as the input earthquakes. However, by using such analysis model and input ground motions, the responses obtained from the dynamic analyses could be quite different from the real responses of actual structures subjected to actual ground motions spread from the hypocenter. The reasons lie in that the measured characteristics of actual structure usually differ from those of the analysis model as mentioned above, which based on the nominal design data, and that the input ground motions are significantly influenced by the oscillating mechanism of hypocenter, the characteristics of spreading

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path, and especially the ground surface characteristics around the structure location. Therefore, it is difficult for the present dynamic analysis procedure to accurately establish each term of the motion equations (i.e., structural stiffness, masses, damping and the input ground motions).

In this study, by identifying the characteristics of actual structure through tests and applying the actually observed ground motions, an elaborate procedure of dynamic analysis is carried out to predict the seismic behavior of the studied bridge. Such predictions are obtained by consistently considering the oscillation of the hypocenter and the ground motions spreading from hypocenter to structural location. The calculated results are compared with those from the conventional dynamic method.

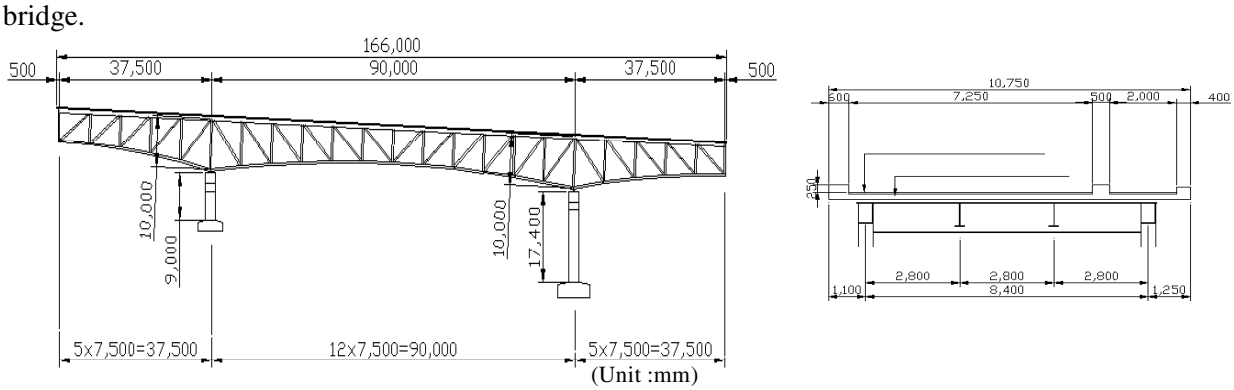
Such elaborate structural model data and input ground motions are formulated through following steps. First, the analysis mode is formulated for a truss bridge [5], where the structural stiffness and mass are more accurately modified by using dynamic characteristics measured in tests. Then, the input ground motions are built for an assumed severe earthquake, which is based a moderate earthquake happened in 2002 at Hyuganadaoki, a place near the studied bridge. To build the ground motions, the empirical Green's function is employed to simulate ground motions of assumed severe earthquake from recorded medium earthquake, and the ground motions of an assumed hypocenter is estimated by using the H/V spectrum ratio in the microtremor condition [6]. Moreover, the phase difference as well as material geometrical nonlinearity are taken into account. The bridge is an upper-deck steel truss bridge, which has complex seismic behaviors.

STUDIED BRIDGE AND 2002 HYUGANADAOKI EARTHQUAKE

The layout and general information of the studied upper-deck truss bridge is presented in Fig. 1 and Table 1. This bridge was built in Miyazaki-ken of Japan in 1981, based on the 1972 Japan design codes. A moderate earthquake happened on Nov. 4, 2002 and its hypocenter was the Hyuganadaoki area of Miyazaki-ken of Japan, which is near to the location of the bridge. Before it, an earlier severe earthquake (M7.5) also happened in this area in 1968 and the recorded ground motions are adopted in JRA (2002) as one of the Level-2 standard design ground motions. Figure 2 shows the ground accelerations recorded in 2002 Hyuganadaoki earthquake at an observation point nearest to the studied bridge.

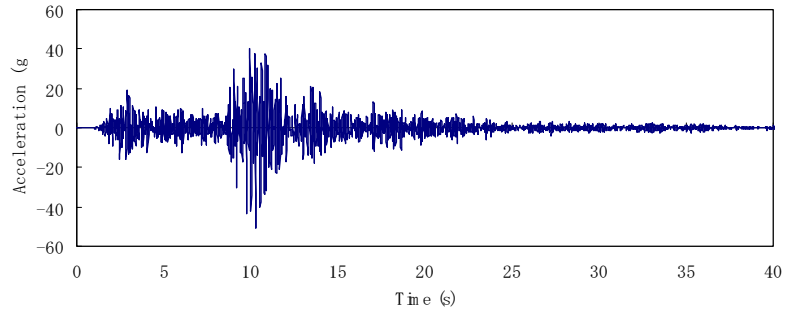
Table 1. General Information of the Studied Bridge

Bridge Type	Three Span Deck-type Truss Bridge
Length of Bridge	166.0 m
Spans	37.5m + 90.0m + 17.5m
Width of Deck Section	9.75m
Region Modification Factor	0.85

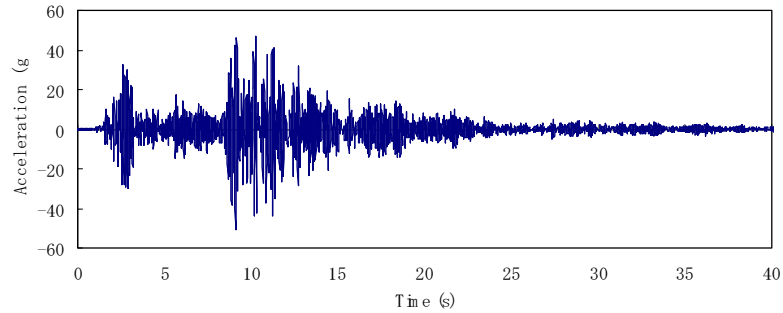


(a) Layout (b) Cross Section

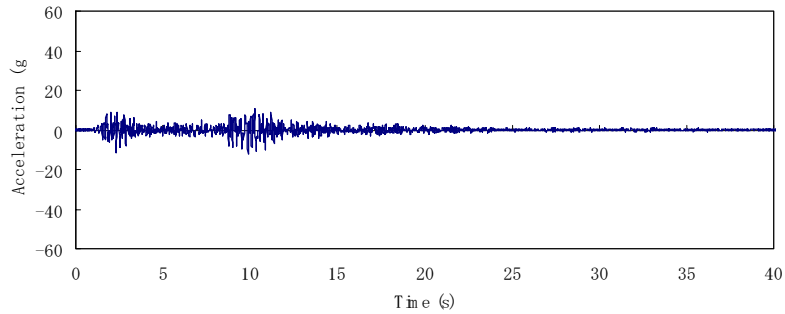
Fig. 1 Studied Bridge



(a) NS Components



(b) EW Components



(c) UD Components

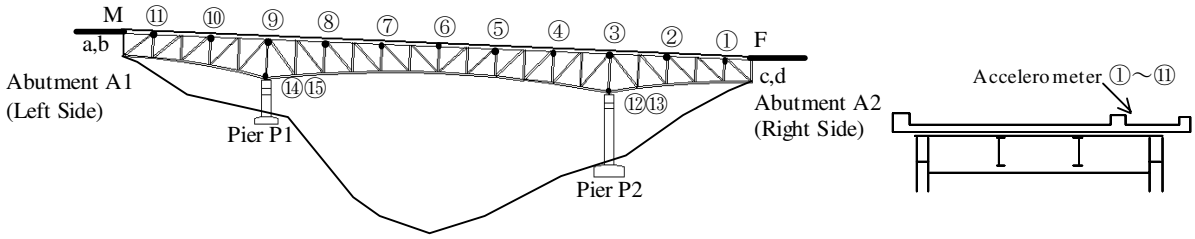
Fig. 2 Observed Ground Accelerations in 2002 Hyuganadaoki Earthquake

SUMMARY OF ON-SITE TEST FOR ACTUAL VIBRATION CHARACTERISTICS

To obtain the actual vibration characteristics of the studied bridge, the measuring experiment was carried out in the common tremor state. Servo-accelerometers are set at locations ①~⑭ (see Fig. 3) and the slight vibrations in longitudinal, transverse, and vertical directions are recorded for 120 seconds. An example of recorded accelerations in transverse direction (30 sec) is shown in Fig. 4(a). The power spectra of vibration in transverse direction at one recording location (location ② in Fig. 3) are illustrated in Fig. 5 as an example. From this figure, it is observed that the frequency in transverse direction is 1.66Hz (period = 0.602 sec) for the first mode and 4.27Hz (period = 0.234sec) for the second mode.

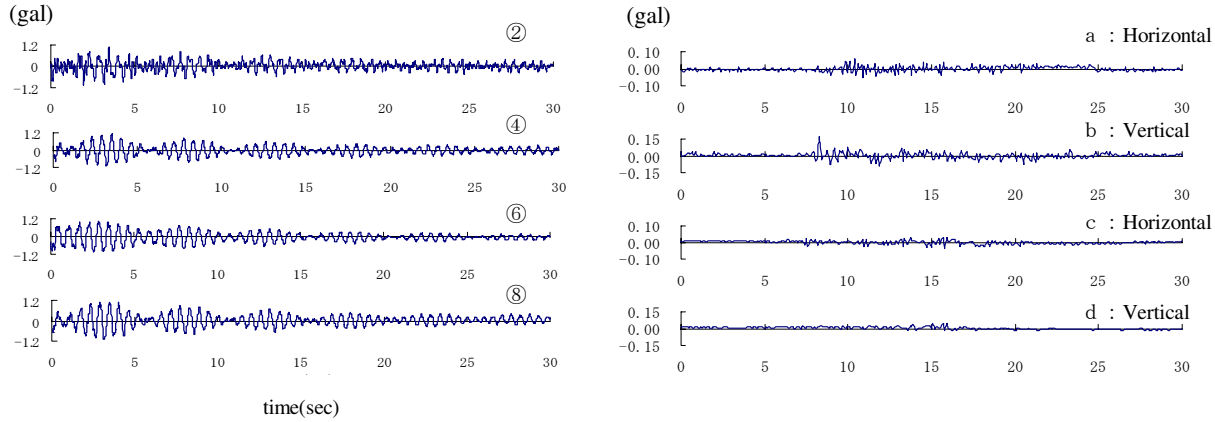
To measure the ground characteristics, same servo-accelerometers are located at position a~d in Fig. 3. However, to obtain ground vibration characteristics for the H/V spectrum ratio used later, the

accelerations in horizontal and vertical directions have to be recorded simultaneously. Figure 4(b) presents one example of the recorded motions (30sec). The measuring results are summarized in Table 2. Since the vibration in vertical direction is coupled with that in longitudinal direction, the first modal frequency in longitudinal direction is also used for first mode in vertical direction.



(a) Side View (b) Cross Section

Fig. 3 Locations of Servo-accelerometers



(a) At Locations on Bridge (②, ④, ⑥, ⑧) (b)At Locations on Ground (a, b, c, d)

Fig. 4 Samples of Recorded Accelerations

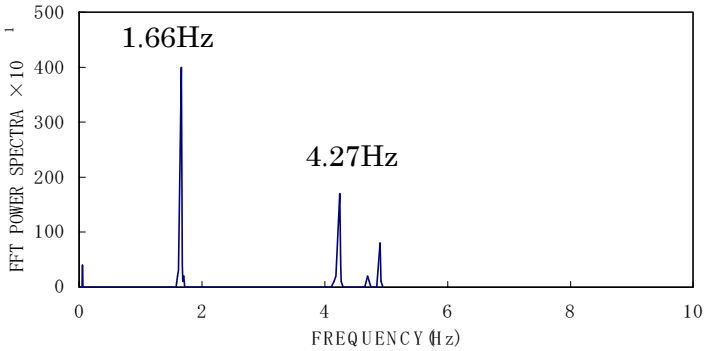


Fig. 5 FFT Power Spectra of Location ① (in Transverse Direction)

Table 2. Measured Frequency from On-site Test

Direction	Longitudinal	Transverse	Vertical
1st Mode	1.86 Hz	1.66 Hz	1.86 Hz
2nd Mode	4.10 Hz	4.27 Hz	-

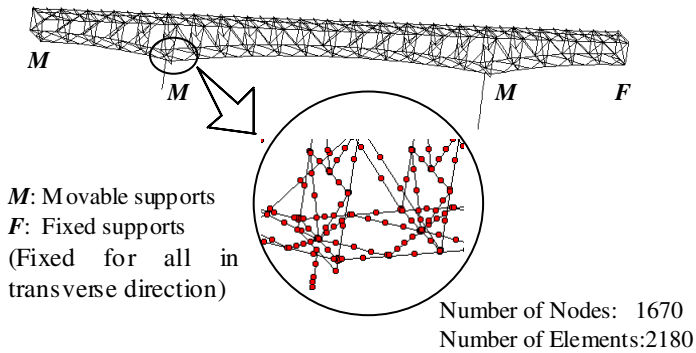


Fig. 6 Analysis Model

VERIFICATION OF ANALYSIS MODEL

Analysis Model

The three dimensional analysis model of the truss bridge is built as Fig. 6. Based on the previous research [5], not only the steel members but also the concrete deck are accurately modeled and the actual dynamic characteristics measured as mentioned above are taken into account in the model. The material nonlinearity is considered by using fiber element and the buckling of members (i.e., geometrical nonlinearity) is also accounted for by dividing one member to several elements, which can simulate the buckling mode of the member.

Comparison Between Test Results and Analysis Results

Before the time-history analysis, the static analysis subjected to dead loads is carried out and based on this initial state the material and geometrical nonlinearity is included in the following dynamic analysis. More details of analysis are referred to the previous paper [5].

The modal analysis results of the model used below are summarized in Table 3. In this table, the test results are also included for comparison. It is observed that the computed modal periods in longitudinal, transverse and vertical directions match the test results well.

Table 3. Modal Periods and Modal Participation Factors

Mode	Period (sec)	Frequency (Hz) (1)	Participation Factor			Measured Frequency (Hz) (2)	(1)/(2)
			Longitudinal Direction	Transverse Direction	Vertical Direction		
1	0.592	1.690	2.524	0.521	-26.664	1.860	0.91
2	0.565	1.770	-0.038	38.361	0.375	1.660	1.07
3	0.301	3.325	0.044	1.464	-0.009	-	-
4	0.272	3.680	-1.510	-0.017	1.613	-	-
5	0.261	3.829	-40.223	0.019	-7.523	4.100	0.93
6	0.226	4.419	0.072	23.779	-0.099	4.270	1.03
7	0.202	4.955	5.954	-0.068	-24.997	-	-

ASSUMPTION OF INPUT GROUND MOTIONS BASED ON AVAILABLE EARTHQUAKE RECORDS

Assumption of Ground Motions at Different Sites

To utilize the available earthquake records to simulate the seismic behavior of the studied bridge, the original ground motions are modified. This is because the location of the recorded earthquake motions is 2.5km away from the site of the studied bridge. Although the ground classification of both locations is the rock soil, to more accurately predict the seismic responses of the structure, the ground motions according to the location of the structure should be adopted. However, there are no earthquake records of the location, such ground motions are assumed by the method proposed by Oukuma et al. [6]. This method is summarized below.

In this method, it is assumed that the vibration amplitude translation ratio between locations A and B caused by earthquakes are equal to the ratio between H/V spectrum ratios of two locations in common microtremor state. This assumption can be presented by Eqs. (1) and (2), where the translation ratio for vertical vibrations between two places of same ground condition can be approximately assumed as 1.0 [6].

$$H_B(\omega) = R_{B/A}^{H/V}(\omega)\, H_A(\omega) \tag{1}$$

$$V_B(\omega) = V_A(\omega) \tag{2}$$

Here, $H_X(\omega)$ and $V_X(\omega)$ are the Fourier spectrum amplitude of ground motions at location X in horizontal and vertical directions, respectively; $R_{B/A}^{H/V}$ is the ratio between H/V spectrum ratios of locations A and B in tremor state.

In this study, based on the ground motions recorded in 2002 Hyuganadaoki earthquake (location A), the Fourier spectrum amplitude $H_A(\omega)$ and $V_A(\omega)$ can be calculated and the $R_{B/A}^{H/V}$ ratio is obtained by observation at this site (location A) and the location of the studied bridge (location B). By using Eqs. (1) and (2), the Fourier spectrum amplitude of the structure location, $H_B(\omega)$ and $V_B(\omega)$, can be calculated (Refer to Fig. 7). Then based on the assumption that the ground motions of the bridge location have the same Fourier phase as that of the observation spot, the input ground accelerations can be obtained by reverse Fourier transformation.

Figure 8 shows the response spectra (5% damping) of the originally recorded acceleration motions as well as the motions of the bridge's left and right sides, which are estimated by the above H/V spectrum ratio method. It is found that there is obvious difference among the motions at three locations. Especially for bridge's left side, the responses in the phase from 0.2 to 0.4 sec become quite larger.

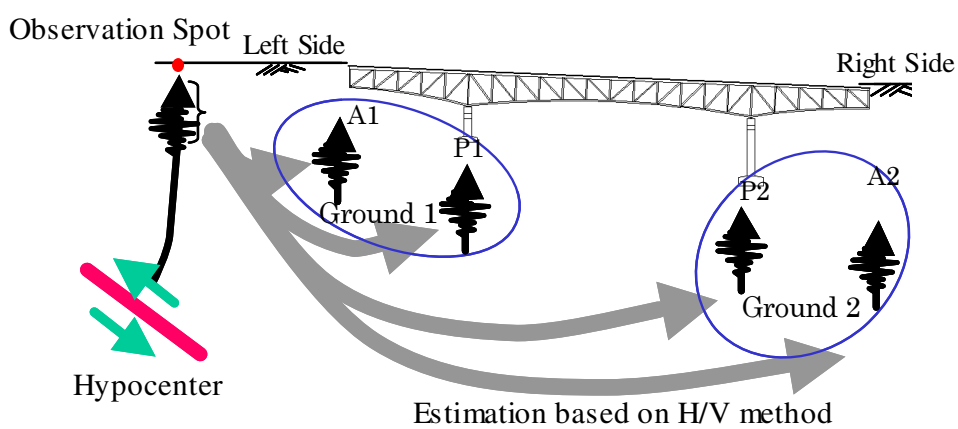


Fig. 7 Estimation of Input Ground Motions Based-on H/V method

Assumption of Ground Motions for a Severe Earthquake

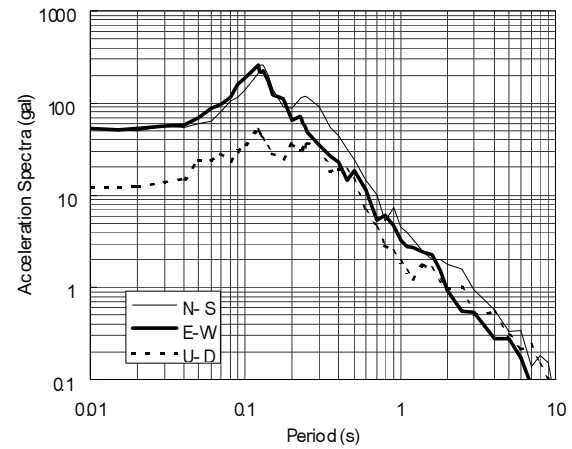
To assume the ground motions of a severe earthquake, there are some methods available, such as composition method based on assumed ground fault models [8] and the empirical Green's function method based on available ground motions of a moderate earthquake [9, 10]. In this study, the empirical Green's function method is adopted because the studied bridge is not near to the faults of North Hyuganadaoki. The ground motions recorded in 2002 Hyuganadaoki earthquake, a moderate earthquake of M5.7, is employed in this method.

The empirical Green's function method applied in this study is that improved by Harada et al. [10], which is based on the procedure proposed by Irikura [9]. Equation (3) shows the translation.

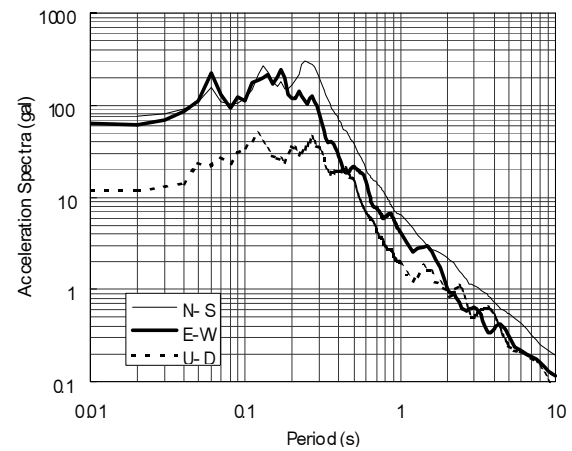
$$u(x,\omega)=\sum_{m=1}^{N_L}\sum_{n=1}^{N_W}\frac{R_0}{R_{mn}}T_{mn}(\omega)e^{-i\omega(\tau_{mn}+t_{mn})}u_0(x,\omega) \qquad (3)$$

Here, N_L and N_W are the numbers of divisions of the fault in length and width directions, respectively; R_0 is the distance from the hypocenter; R_{mn} is distance from a divided part (m, n) of the fault to hypocenter; τ_{mn} is the time delay for dislocation surface spreading from hypocenter to fault part (m, n); t_{mn} is the time of S wave happened in fault part (m, n) spreading to the observation location; T_{mn} is the translation function to account for the difference between the hypocenter time function of the considered fault part during a large earthquake and that during a small earthquake. More details are referred to in the literature [10].

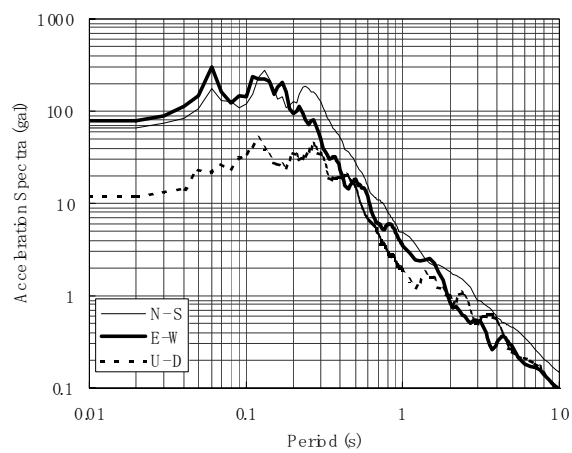
Table 4 shows the fault parameters of the assumed large earthquake (M7.5), which is corresponding to the seismic requirements of the local area. The obtained response spectra of the input ground motions for large earthquake are illustrated in Fig. 10(a). However, these ground motions are still corresponding to the observation spot of the original records and thus by employing H/V spectrum ratio method as stated above, the ground motions corresponding to bridge's left (A1) and right (A2) abutments are assumed, as shown in Figs. 10 (b) and (c), respectively. Figure 9 summarizes the procedure stated above, where the ground motions of a severe earthquake is first assumed for the observation spot and then the corresponding ground motions at structural locations are estimated by H/V spectra ratio method.



(a) At Observation Spot



(b) On Left Side



(c) On Right Side

Fig. 8 Response Spectra Based-on Recorded Medium Earthquake

Table 4. Fault Parameters of the Assumed Large Earthquake (M7.5)

Parameter (Unit)	Value	Parameter (Unit)	Value
Magnitude	7.5	Fault Dips (degree)	20
Fault Length (km)	70	Rise Time (sec)	2.25
Fault Width (km)	40	S-wave Velocity (km /sec)	3.0
Seismic Moment (dyne-cm)	2.8×10^{27}	Rupture Velocity (km /sec)	2.1
Depth of Fault (km)	3.0	Fault Strikes	N20E

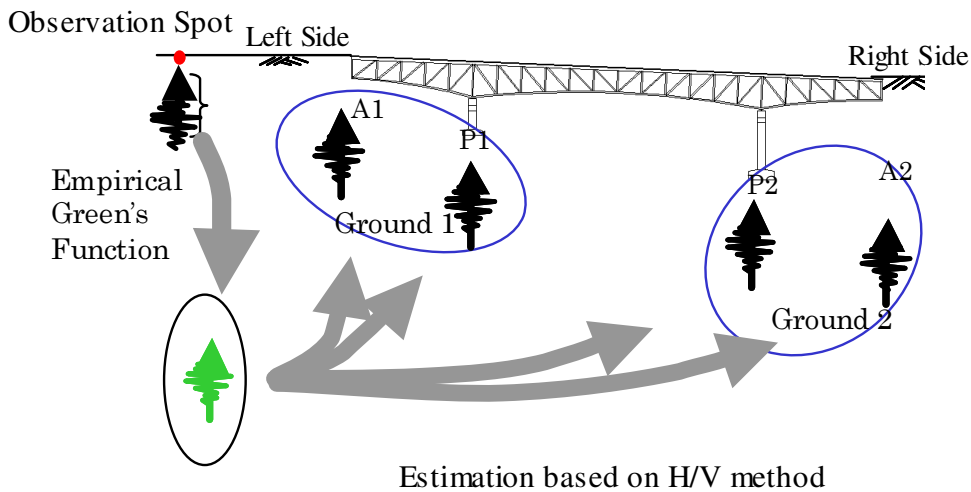


Fig. 9 Estimation of Input Ground Motions Based-on Empirical Green's Function and H/V method

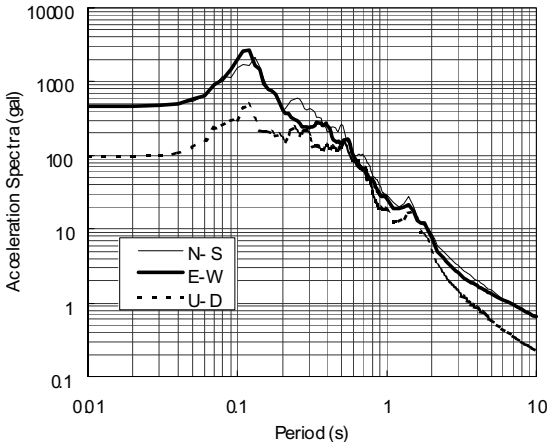
ESTIMATION OF SEISMIC RESPONSES OF STUDIED BRIDGE IN THE 2002 HYUGANADAOKI EARTHQUAKE

Actual Damages of the Bridge Happened in the 2002 Hyuganadaoki Earthquake

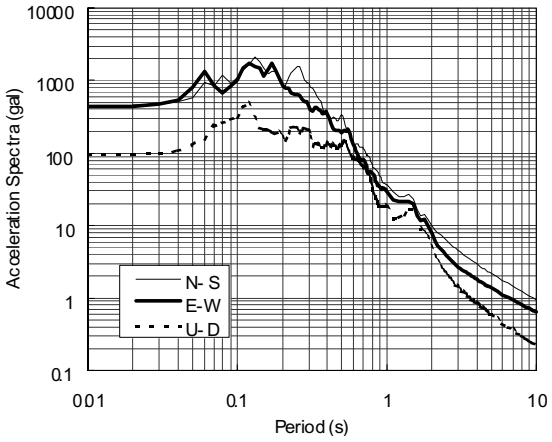
Immediately after the 2002 Hyuganadaoki earthquake, the studied bridge was inspected and the moving trace of the movable bearing at pier P1 was observed (Fig. 11), which was not detected in the examination before the earthquake. The moving distance is about 8 mm, which can be thought as the relative displacement between the superstructure and the top of pier P1.

Analysis Method

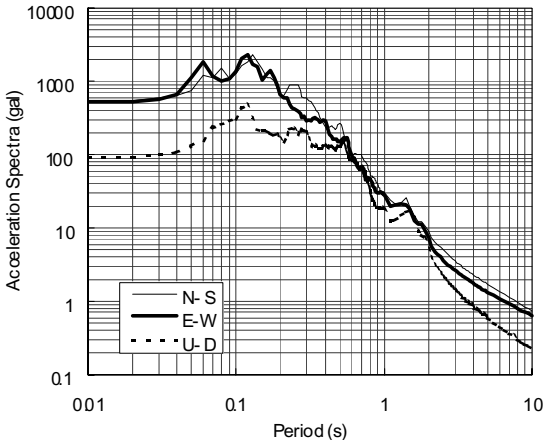
By the method proposed in the above section, the ground motions in NS, EW and UD directions are assumed and applied in the bridge's longitudinal direction (from abutment A1 to A2), transverse direction, and vertical direction, respectively. The ground motions are input simultaneously in three directions. Different motions at the left side (abutment A1 side) from those at right side (abutment A2 side) are adopted (Case 1). Moreover, the phase difference is considered by inputting motions first at the left side (abutment A1 and pier P1), where is nearer to the hypocenter (See Fig. 7), and then after a time delay inputting at the right side (abutment A2 and pier P2). The time delay is assumed as 0.03 sec, which is calculated by considering the distance from the hypocenter (about 130 km), the depth of the hypocenter (35km), the angle from hypocenter to location of the bridge, the velocity of the motion waves, and the distance between two sides of the bridge. In the analysis, zero accelerations for the time delay are inserted at the beginning of ground motion data of right side and such modified motions are input at the same time as the left side.



(a) At Observation Spot



(b) On Left Side



(c) On Right Side

Fig. 10 Response Spectra of Assumed Severe Earthquake

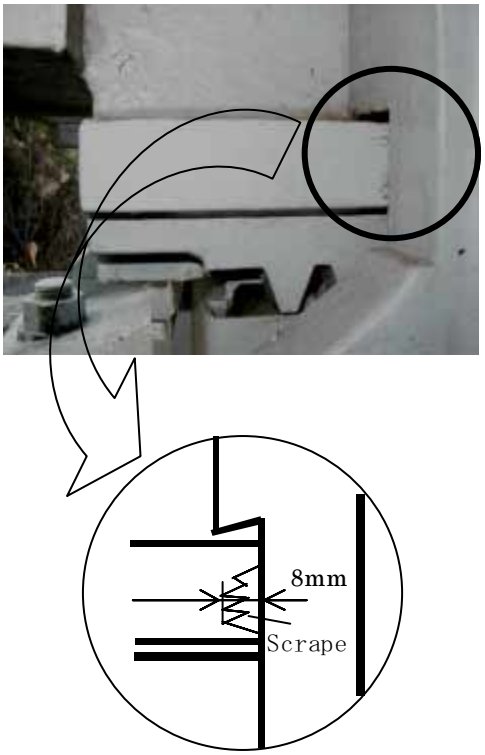
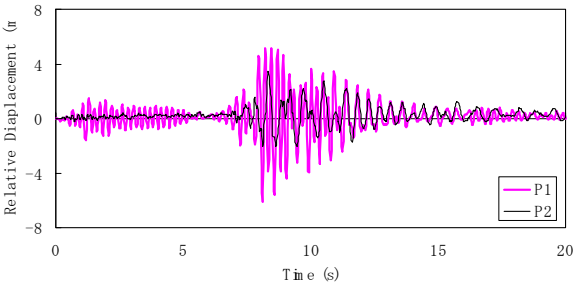
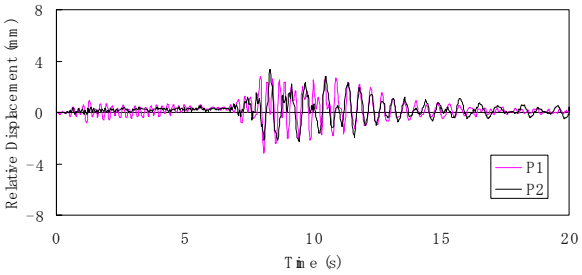


Fig. 11 Moving Trace Left in the 2002 Hyuganadaoki Earthquake



(a) Response History of Case 1



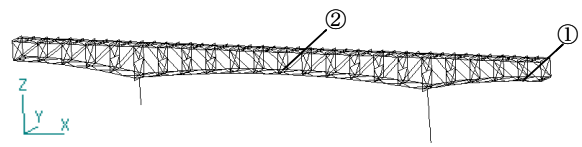
(b) Response History of Case 2

Fig. 12 Comparison of Relative Displacements at Movable Supports

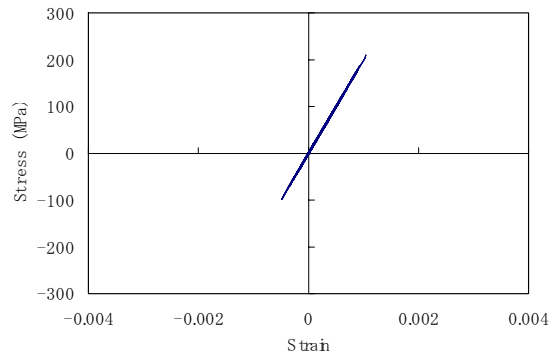
Another analysis case (Case 2) is also carried out by directly using the ground motions recorded at the observation location in 2002 Hyuganadaoki earthquake for both the two sides of the bridge. In this case, the ground motions are also input simultaneously in three directions and the phase difference is also considered. The comparison between results of two cases is investigated.

Analysis Results

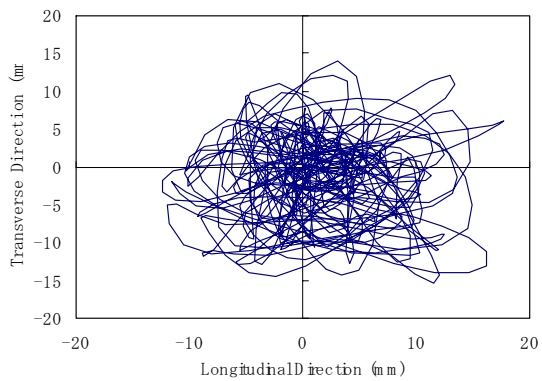
The results of Case 1, where different ground motions are applied at two sides of bridge, are shown in Fig. 12(a), where the ordinate is corresponding to the relative displacement between the superstructure and top of pier. It is found that the maximum relative displacement of pier P1 is about 6 mm, which is somewhat smaller than the actual relative displacement, 8mm, happened in the 2002 Hyuganadoki earthquake. Nevertheless, the predictions are considered to be approximately accurate. The reasons for the difference may lie in that the stiffness of the joints on truss members is not so rigid as assumed in the analysis and that 2% stiffness damping considered in the analysis could be overestimated compared with the actual damping of the bridge in the actual moderate earthquake.



(a)Yielded Parts (No Yields Here)



(b) Stress-Strain Responses of Lower Chord Member ①



(c) Response Displacements of Node ②

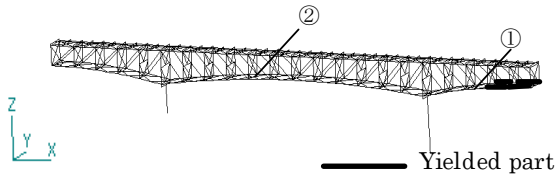
Fig. 13 Results based on Assumed Severe Earthquake (Case 3)

Figure 12(b) illustrates the results of Case 2. By comparing results of two cases, considerable difference between results of two cases is observed. Furthermore, it is observed that in Case 1 the relative displacement at pier P1, which is shorter, is larger than that of P2, while in Case 2 the relative displacements of two piers are almost same. The reason of this phenomenon lies in that in Case 1 the response spectra of the input acceleration motions at P1 side have larger values around 0.25 sec (Fig. 8(b)), which is close to the fundamental period of pier P1, 0.23 sec. From Fig. 12(b), it is also found that the maximum relative displacement of pier P1 is about 3mm, considerably smaller than the actual value observed in the earthquake.

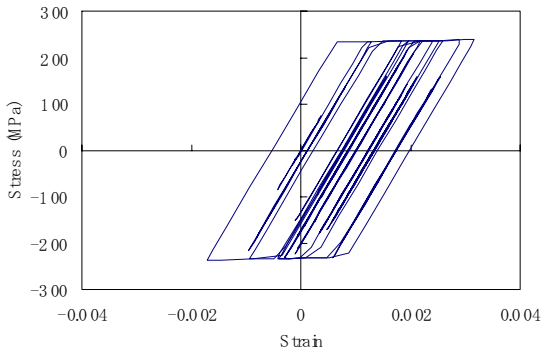
ESTIMATION OF SEISMIC RESPONSES IN AN ASSUMED SEVERE EARTHQUAKE

Analysis Method

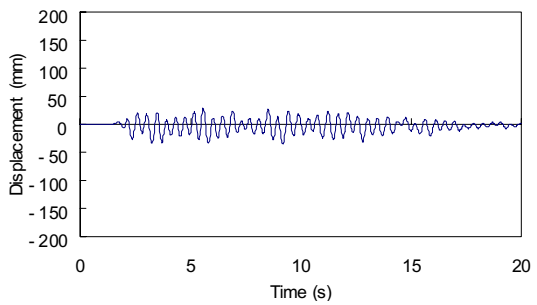
By using the assumed ground motions for a severe earthquake as presented above, the responses of the studied bridge are predicted. The application of input ground motions is same as that of the above section (Case 3), which for the actual moderate earthquake. For comparison, the standard ground motions for the severe earthquake specified in JRA [4] are also implemented in the analysis. The application of the standard motions is following the usual method, where the phase difference is not accounted for.



(a)Yielded Parts

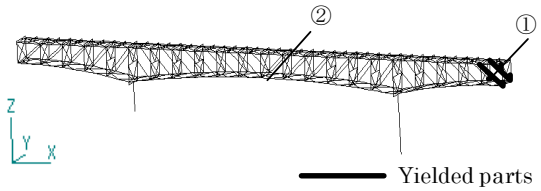


(b) Stress-Strain Responses of Lower Chord Member ①

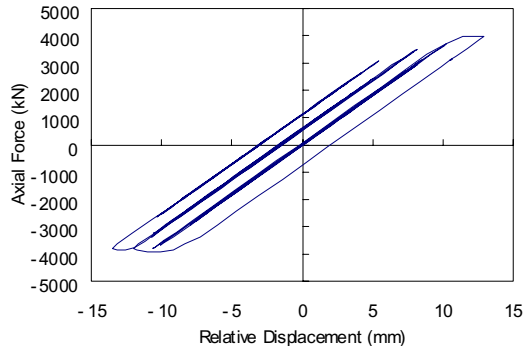


(c) Displacement Responses in Longitudinal Direction (Node ②)

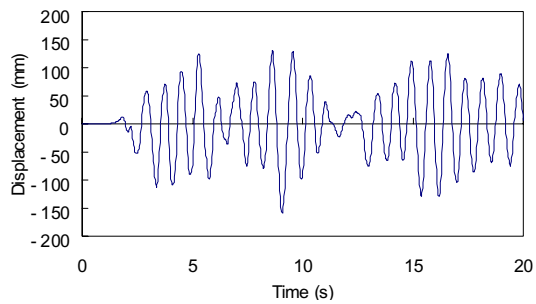
Fig. 14 Results in Longitudinal Direction based on Standard Ground Motions (Case 4)



(a)Yielded Parts



(b) Stress-Strain Responses of Lower Chord Member ①



(c) Displacement Responses in Transverse Direction (Node ②)

Fig. 15 Results in Transverse Direction based on Standard Ground Motions (Case 5)

Though the above discussions are focused on the seismic behavior in longitudinal direction, the standard ground motions are applied separately in both longitudinal direction (Case 4) and transverse direction (Case 5), as the usual design use.

In common design procedure where the standard ground motions are used, the structural model is usually based on nominal design data. However, in this study the structural model modified by actual measurement is also used for the Case 4 and Case 5.

Analysis Results

The results of Case 3, where the assumed ground motions for severe earthquake is applied, are presented in Fig. 13. It is found that the members of the bridge do not yield (Fig. 13(a)). The largest strain happens in the lower chord member near fixed bearing at right abutment (Fig. 13(b)). Figure 13(c) shows the response history of the bridge center. It is observed that the maximum displacements in longitudinal and transverse directions are similar and such analysis simultaneously considering input ground motions in three directions can better simulate the actual seismic behavior of structures in earthquake. The maximum

relative displacement at top of pier P1 is about 61mm, which is about 10 times of the displacement happened in the 2002 Hyuganadaoki moderate earthquake.

The results by using the standard ground motions in JRA [4] (i.e., Case 4 and Case 5) are shown in Figs. 14 and 15. Yielding is found in both cases, which happens in the lower chord member near fixed abutment for longitudinal direction and in the diagonal member near the fixed abutment for transverse direction. Nevertheless, no parts reach the ultimate state. The maximum response displacements are larger than the results by using the assumed ground motions (Case3), especially as much as 10 times for displacements in transverse direction.

SUMMARY

In this study, by using actual structural characteristics modified by measurement, the seismic behaviors of a truss bridge have been investigated according to the observed ground motions in a moderate earthquake happened in 2002 as well as the motions assumed for a severe earthquake. The following observations have been obtained.

(1) By identifying structural characteristics through on-site measurement and estimating the ground motions away from the observation spot through H/V spectrum ratio method, more accurate motion equation can be formulated to predict the seismic responses.
(2) In this study, the analyses are carried out by consistently estimating the hypocenter ground motions based on assumed faults and formulating ground motions at structural sites spread from the hypocenter.
(3) Compared with the usual design method, where structural characteristics based on nominal design data and standard acceleration motions are used, the analysis procedure proposed in this study, as stated in (1) and (2), can more accurately predict the structural responses.
(4) Based on the 2002 Hyuganadaoki moderate earthquake records, the ground motions of the bridge's location, which is away from the observation spot, are estimated through H/V spectrum ratio method. It is thought that based on such an analysis procedure, the actual seismic responses can be fairly simulated. On the other side, if the original motions recorded at the site away from the structural location are directly employed in the analyses, the simulation results could be quite inaccurate. Thus it can be concluded that it is necessary to estimate ground motions by considering the ground characteristics of the site.
(5) The analyses results are considerably different when inputting simultaneously in three directions the ground motions estimated in this study and when, as the usual design use, applying accelerations separately in longitudinal or transverse direction.

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