

Free vibrations and seismic responses of Shin Saikai Bridge and Saikai Bridge

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ABSTRACT: This paper discusses natural vibration properties of the Shin Saikai Bridge and the Saikai Bridge which are famous arch bridges in Japan, based on the original cross sections using 3D FE model and compared with experimental results. The seismic response characteristics and seismic safety are examined by carrying out nonlinear seismic analysis under the Level 2 earthquakes in order to check seismic safety of the Shin Saikai Arch Bridger and gasp whether the Saikai Arch Bridge should be reinforced against the strong earthquake.

1 INTRODUCTION

The Shin Saikai Bridge is a road bridge that was opened to traffic in March 2006. This CFT arch bridge marks the first use in Japan of concrete-filled steel tubes (CFT) for the arch ribs of a road bridge (Takaharu et al 2006). The natural frequency and dynamic response properties (seismic response and running-vehicle load response) of the Shin Saikai Bridge have been determined through analysis (Yoshimura et al 2006, Wu et al 2006). In addition, vibration tests were carried out at the time of the bridge's completion to determine the running-vehicle load response. However, tests have not yet been performed to determine the natural vibration properties (natural frequency and damping constant).

The Saikai Bridge, which is a steel arch bridge constructed in 1955 with the span of 216m, is known as not only the first long-span bridge in Japan but also the historical bridge. However, since the design traffic load was small as 130kN and the concrete slab is also thin with 13cm, there is a recent problem in the maintenance accompanying the increase of traffic loading. Because the construction of the bridge was more than 60 years ago, various analyses and tests that are widely used in recent years could not be carried out.

In order to achieve the objective of this study, which is to determine the validity of the analysis model for Shin Saikai CFT arch bridges and the Saikai steel arch bridge and the damping constants, the natural vibration characteristics of the actual bridge must be determined. Accordingly, the authors performed microtremor measurements of the two arch bridges and estimated the natural vibration properties (natural frequency, natural vibration shapes and damping constant) through data analysis using the subspace identification (SSI) method. The results were compared with the natural frequency determined in the analysis in order to evaluate the validity of the analysis model. The damping constants for the bridge were also estimated.

The seismic response characteristics and seismic safety are examined by carrying out nonlinear seismic analysis under the Level 2 earthquakes in order to check seismic safety of the Shin Saikai Arch Bridger and gasp whether the Saikai Arch Bridge should be reinforced against the strong earthquake.

2 VIBRATION OF THE SHIN SAIKAI BRIDGE

2.1 Overview of the bridge

The Shin Saikai Bridge is a highway bridge on the Nishisonogi Region Expressway. The bridge spans the Harioseto channel that divides Saikai City from Sasebo City in Nagasaki Prefecture. This new highway bridge was planned as the first CFT arch bridge in Japan and began service on March 5, 2006.

Fig.1 shows a general view of the bridge. The arch ribs of the bridge comprise two parallel arch ribs, each of which has a triangular truss consisting of three steel tubes measuring 812.8 mm in diameter as chords. The arch ribs on the right and left are connected by two sets of transverse beams. Other than the transverse beams, there are no members connecting the left and right arch ribs. At the time of erection, a pin structure was used for the base of the arch ribs, but following closure the arch ribs were fastened in place by means of reinforced concrete jacketing.

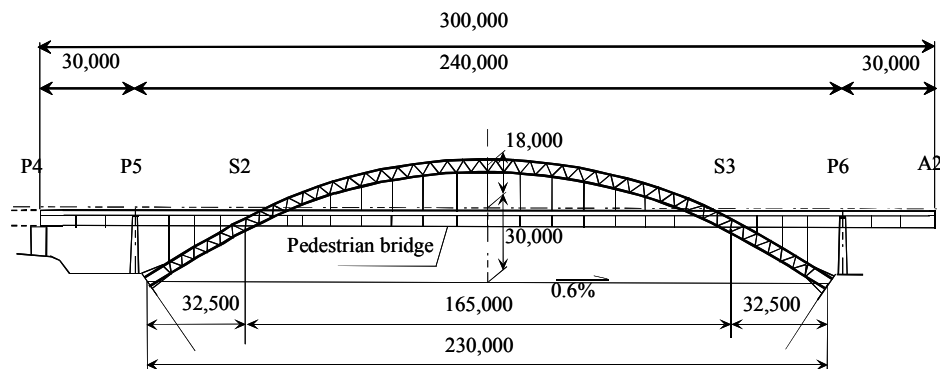


Figure 1 : General view of the Shin Saikai Bridge (unit: mm)

The stiffening girders and crossbeams are box girders measuring 2 m in height and 1.5 m in width. The vertical beams are I-beams. The stiffening girders are supported by the arch ribs by means of bearings at the position of the transverse beams, by means of supports at the deck section, by means of hanger cables at the through section, and by means of brackets at the location of the crossbeam sections.

This bridge was planned as a vehicle-only bridge. However, as both banks at the location of the bridge have been designated a prefectural park, a pedestrian bridge has been added beneath the girder to provide convenient access for park users.

2.2 Natural vibration analysis

Fig.2 shows a structural model of the main bridge section of the Shin Saikai Bridge. The model is a 3-dimensional finite element model (FEM) created using beam elements and truss elements for all members including the pedestrian bridge. The overall shape of the arch ribs of the actual bridge is that of a three-centered circle; in the structural model, this is approximated through the use of straight lines to connect the nodes. In a steel tube truss structure comprising arch ribs and truss transverse beams, normally pins are used to connect the members to form axial members. In this study, however, steel connection that is nearer to the actual connection conditions was used, and beam elements were used for the members so bending could be taken into consideration. Modeling was carried out as follows:

(1) A steel and concrete composite structure was used for the arch rib chord members, and adhesion of the steel tubes and concrete to one another was not taken into consideration;

(2) Care was taken to ensure that the composite slab would be able to resist horizontal force through the use of virtual members for the floor system. In addition, rigidity was taken into consideration for the main girders;

(3) The cables were modeled as axial members that do not resist compression.

Moreover, with respect to the arch ribs, stiffening girders and pedestrian bridge, the values for the section performance of each structure will differ depending on location, so the values for the center positions were used as representative values. As changes to drawings and the like were made at the actual bridge design stage and only the weight of the slab was provided at the time of design, the section at the time of completion was used for the evaluation of rigidity and mass. The bridge had 1086 nodes and contained 2,295 members.

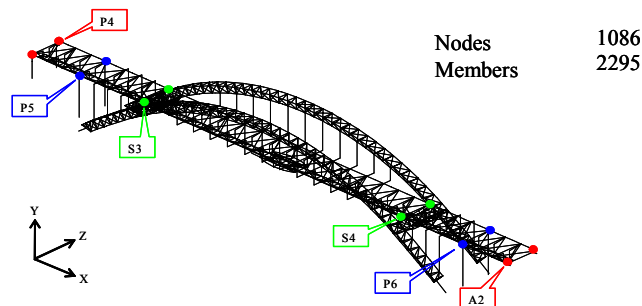


Figure 2 : Analysis model

The base for the arch ribs and bridge piers are fixed. Moreover, as rubber bearings to distribute reaction force are used for the joints between stiffening girders and bridge piers (S1 and P4, S2 and P5 and S5 and P6), between stiffening girders and abutments (S6 and A2) and between stiffening girders and truss transverse beams (S3 and S4), these were considered to be elastic supports produced by spring elements, and the spring constant was determined.

With regard to the bearing conditions for the pedestrian bridge, the displacement in the direction perpendicular to the bridge axis was fixed at the location of the horizontal bearing in the truss transverse beam section.

2.3 Estimation of natural vibration characteristics through microtremor measurement

Microtremor measurements were performed for the main bridge and side span sections of the Shin Saikai Bridge. Measurement was performed for 1,238 seconds, with a sampling interval of 0.2 millisecond. In preliminary tests for microtremor measurements at the site, stable data with a sampling rate of 5 KHz were obtained, so this rate was used. In contrast to "output-only" structural identification as in the case of the microtremor vibration tests, the SSI method was used to identify the natural frequency, damping constant and natural vibration shapes.

2.4 Result of analysis

Table 1 shows the natural frequency and damping constant determined using the SSI method from the results of microtremor measurements of vertical vibrations and out-of-plane vibrations on the main span and vertical vibrations on the side span. The table also shows the natural frequency derived through analysis. Table 5 includes all vibration values derived through multiple measurements. The natural vibration shapes were identified from the amplitude and phase of the measurements at three locations.

There are no measurements for 1st, 3rd, 6th, 7th, 10th and 12th modes. This is because arch rib out-of-plane vibrations were dominant, so these natural vibration shapes could not be determined from stiffening girder measurements alone. As the two arch ribs on this bridge are not connected to one another, low-mode vibrations possess many natural out-of-plane frequencies.

A comparison of the analysis values with the measured values reveals that, with the exception of the 4th mode and 13th mode vibrations, the values were generally in agreement, with a disparity of no more than 10%. Accordingly, modeling of this bridge was judged to be appropriate.

As shown in Table 1, the values for damping constant obtained through measurement were large in the case of 2nd mode and 5th mode vibrations. For these two vibration shapes, displacement in the bridge axial direction is dominant in the vibration. This is presumed to result from damping by the rubber bearings used to disperse the reaction force. In structural analysis, these rubber bearings are evaluated using only spring constants, but these results suggest that the effect of damping should be taken into consideration when performing seismic response analysis. The other damping constants for out-of-plane and vertical vibrations were low (approximately 0.01). The damping constant was about the same as in the case of a steel bridge.

Table 1: Natural frequency, damping constant and natural vibration mode of the Shin Saikai Bridge

| Mode | Frequency (Hz) | | | Damping constant | Natural vibration shape |
|------|----------------|-------------------|------------|------------------|--|
| | Analysis value | Measurement value | Difference | | |
| 1st | 0.365 | — | — | — | 1st mode out-of-plane (sym) |
| 2nd | 0.47 | 0.434 | -7.7 | 0.065 | 1st mode axial (inverse sym) |
| 3rd | 0.473 | — | — | — | 2nd mode out-of-plane (sym) |
| 4th | 0.574 | 0.49 | -14.6 | 0.01 | 3rd mode out-of-plane (sym) |
| 5th | 0.64 | 0.581 | -9.2 | 0.029 | 2nd mode axial (inverse sym) |
| 6th | 0.683 | — | — | — | 4th mode out-of-plane (inverse sym) |
| 7th | 0.833 | — | — | — | 5th mode out-of-plane (inverse sym) |
| 8th | 0.85 | 0.836 | -1.6 | 0.006 | 6th mode out-of-plane (sym) |
| 9th | 0.927 | 0.885 | -4.5 | 0.006 | 1st mode vertical (sym) |
| 10th | 1.113 | — | — | — | 7th mode out-of-plane (inverse sym) |
| 11th | 1.128 | 1.11 | -1.6 | 0.003 | 8th mode out-of-plane (inverse sym) |
| 12th | 1.253 | — | — | — | 9th mode out-of-plane (sym) |
| 13th | 1.455 | 1.307 | -10.2 | 0.002 | 2nd mode vertical (sym) |
| 14th | 1.475 | 1.365 | -7.5 | 0.008 | 10th mode out-of-plane (inverse sym) |
| 15th | 1.51 | 1.369 | -9.3 | 0.003 | 4th mode axial (inverse sym) |
| 16th | 1.867 | 1.766 | -5.4 | 0.013 | 11th mode out-of-plane (sym) |
| 17th | 1.944 | 1.92 | -1.2 | 0.007 | 12th mode out-of-plane (sym) |
| 18th | 1.948 | 1.915 | -1.7 | 0.005 | 3rd mode vertical (sym) |
| 19th | 2.012 | 1.969 | -2.1 | 0.006 | 13th mode out-of-plane (inverse sym) |
| 20th | 2.026 | 1.983 | -2.1 | 0.004 | 4th mode vertical (sym) |
| 27th | 2.644 | 2.766 | 4.6 | 0.012 | 1st mode vertical in side span (sym) |
| 34th | 3.2 | 3.308 | 3.4 | 0.01 | 2nd mode vertical in side span (inverse sym) |

*Difference = (Measurement value - Analysis value) / Analysis value × 100 (%)

Sym = Symmetric

3 VIBRATION OF THE SAIKAI BRIDGE

3.1 Overview of the bridge

The Saikai Bridge is a road bridge spanning the Inoura Seto at the mouth of the Ohmura Bay. The road bridge connects Sasebo City and Seihi Town, Nishi Sonogi County. The bridge is of fixed deck arch type. The bridge is 316.20 m long, with a span of 216.0 m, and 7.5 m wide, with the bridge deck 43.31 m above the sea (Fig.3).

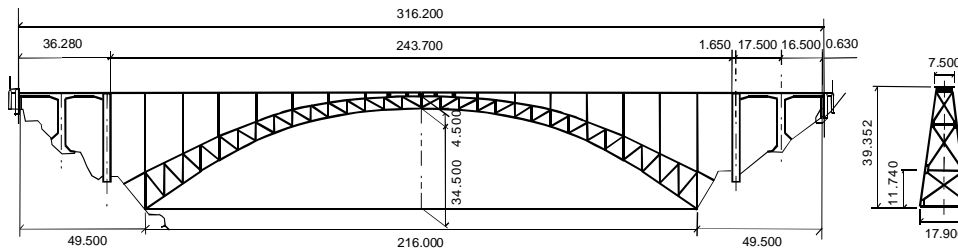


Figure 3 : General view of the Saikai Bridge (unit:m)

3.2 Natural Vibration analysis

The following elements were used for the finite element (FE) models: a nonlinear beam element was used for the member tangent to the arch rib, and stringer; a linear beam element for the vertical member of the arch rib, diagonal member, post and cross beam; a truss element for the lateral bracing and the diagonal member of the post (Fig.3). Two models were prepared: Model A that takes into account the stiffness of the deck and longitudinal girder; Model B that neglects the stiffness of the deck and longitudinal girder but takes into account only the mass.

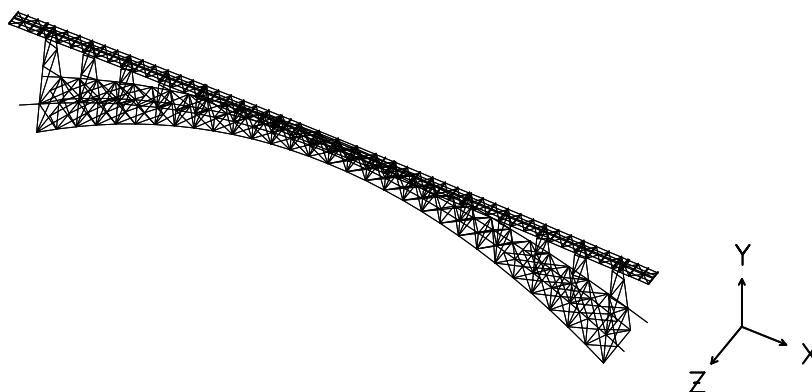


Figure 4 : Analysis model

3.3 Comparison of microtremor measurements with analytical value

The microtremors of the main bridge section were measured. The results of the measurement of natural frequency were compared with those of the analysis. Table 2 lists the natural frequencies obtained from the analysis using Models A and B and the measured natural frequencies, the difference between measurements and analytical values, and calculated damping coefficients.

Focusing on the natural frequencies obtained from the analysis, the higher natural frequencies were obtained from the model that took into account the stiffness of the deck, the effects of which are noticeable on the out-of-plane vibration modes. When the difference between measurements and analytical values obtained from the natural frequency analysis using the two analysis models was focused, the difference was very small. From this result, it can be judged that the models are valid. When the measured natural frequencies were compared with analytical values of in-plane vibrations, the values obtained from the analysis using Model A

were closer to the measurements than Model B. In contrast, when the measurements were compared with analytical values of out-of-plane vibrations, the natural frequency obtained from the analysis using Model A was greatly different from the measurement, and the difference of which was the largest among others. On the whole, the difference between measurements and analytical values obtained from the natural frequency analysis using Model A was smaller than Model B. This indicates that the stiffness of the deck should be taken into account in modeling the bridge. However, the deck was modeled as a single beam and likely to be overestimated. The calculated damping coefficient of the bridge obtained from the analysis was 0.01-0.03.

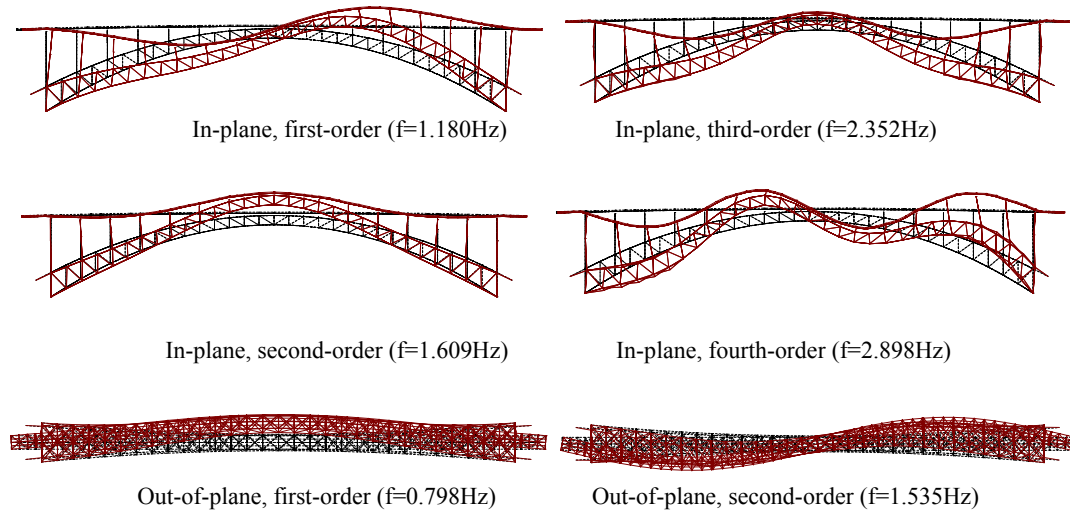


Figure 5 : Natural frequency and vibration mode

Table 2 : Natural frequency and damping coefficient of the Saikai Bridge

| Natural frequency | Natural frequency (Hz) | | Measurement | Difference (%) | | Damping coefficient | |
|-------------------|---|---|-------------|----------------|---------------|---------------------|-------|
| | Analytical value obtained from analysis using Model A | Analytical value obtained from analysis using Model B | | Using Model A | Using Model B | | |
| | In-plane | 1 | | 1.180 | 1.147 | | 1.304 |
| | 2 | 1.609 | 1.483 | 1.626 | -1.0 | -8.8 | 0.006 |
| | 3 | 2.352 | 2.306 | 2.380 | -1.2 | -3.1 | 0.006 |
| | 4 | 2.898 | 2.639 | 2.983 | -2.8 | -11.5 | 0.012 |
| Out-of-plane | 1 | 0.798 | 0.723 | 0.761 | 4.9 | -5.0 | 0.008 |
| | 2 | 1.535 | 1.225 | 1.240 | 23.8 | -1.2 | 0.032 |
| | 3 | 1.859 | 1.604 | 1.812 | 2.6 | -11.5 | 0.027 |
| | 4 | 2.666 | 2.394 | 2.603 | 2.4 | -8.0 | 0.007 |

* Difference = (analytical value - measurement)/measurement x 100%.

4 SEISMIC ANALYSIS

4.1 Analysis model

In order to consider the non-linearity of the material of the arch rib chords (which are CFT structures), a fiber model that enabled automatic consideration of axial force fluctuations and biaxial bending was used.

The material properties of the steel tubes were evaluated using a perfect elastoplasticity model based on Specifications for Highway Bridges (Japan Road Association 2002). Localized buckling was not considered, as the deformation of the steel tubes is confined effectively by the infill concrete. The stress-strain curve and equation which add in the effect of the steel tubes, were used to determine the material properties of the infill concrete (Sato 1993).

4.2 Non-linear seismic response analysis

To analyze the seismic response, non-linear dynamic analysis by means of direct integration using the Newmark β method ($\beta = 1/4$) was carried out. The time interval for numerical integration was 1/400 second and the continuous duration was 40 seconds.

Rayleigh damping was used. As the historical characteristics of the material were not considered, a damping constant of 0.02 was used for this study in accordance with Specifications for Highway Bridges (Seismic Design).

Under the classification system in Specifications for Highway Bridges, the ground directly beneath the Shin Bridge is Class I ground. Accordingly, six standard waveforms -- three Type I earthquake motions (T111, T112 and T113) and three Type II earthquake motions (T211, T212 and T213) for Class I ground -- whose amplitudes had been corrected with a correction coefficient for that particular region (Nagasaki Prefecture: $C_z = 0.7$) were used as the input.

4.3 Results

This section describes the results of the nonlinear seismic response analysis of the Saikai Bridge using Model A that was judged to be capable of reproducing the actual structure, with the nonlinear characteristics of materials taken into account. At the same time, this section describes the results of the nonlinear seismic response analysis of the Shin Saikai Bridge using the model that uses a fiber element for the arch rib. The earthquake resistance of the Saikai Bridge was compared with that of the Shin Saikai Bridge. The analysis conditions for the two bridges were the same. Fig.6 and 7 show the N-Mz correlation curve at the base of the chord member of the Saikai Bridge and at the base of the outside upper chord member of the Shin Saikai Bridge, respectively. In these figures, the analysis results that were most disadvantageous to the chord member are shown. The member of the Saikai Bridge did not yield. For the Shin Saikai Bridge, steel yielded but the member filled with concrete did not yield. From these results, earthquake forces acting on the Saikai Bridge that weighs less than the Shin Saikai Bridge are less than the Shin Saikai Bridge. The Saikai Bridge was evaluated to be safer against strong earthquakes with their cross sections of structural members at the time of design than the Shin Saikai Bridge.

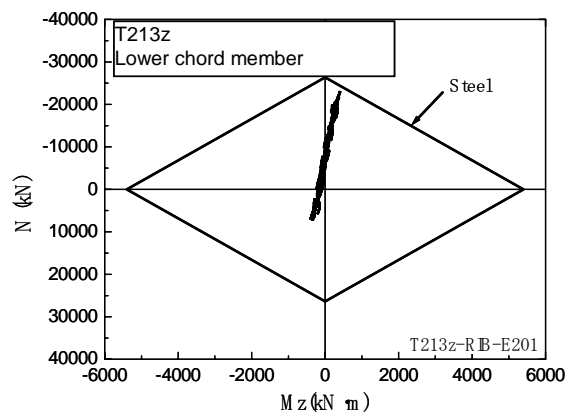


Figure 6: N-Mz correlation curve of Saikai Bridge

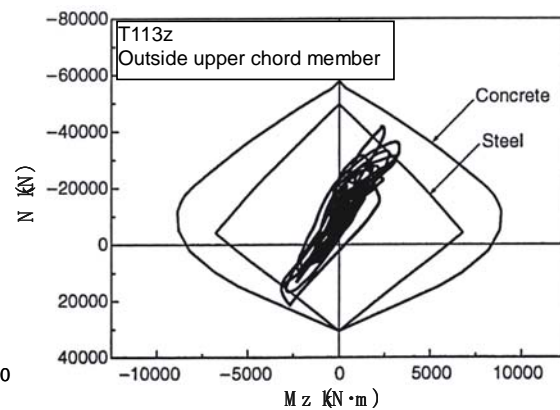


Figure 7: N-Mz correlation curve of Shin Saikai Bridge

5 CONCLUSIONS

The results of the study were as follows:

(1) The analysis values and measurement values for natural frequency of the Shin Saikai Bridge are within approximately 10% of one another, confirming the validity of the dynamic analysis model;

(2) The damping constant for vertical and out-of-plane vibrations was approximately 0.01, closer to that of a steel bridge than that of a concrete bridge;

(3) A damping constant on the high side was obtained for vibrations in the bridge axial direction. This is thought to be the effect of the rubber bearings. In seismic response analysis in the in-plane direction, it will be necessary to take the damping of the rubber bearings into account;

(4) The main structures of the Saikai Bridge were modeled appropriately. Because the stiffness of the deck has effects on the natural frequency of the bridge, the effects need to be taken into account in the natural frequency analysis of the bridge. The calculated damping coefficient of the bridge obtained from the analysis is 0.01-0.03;

(5) The arch ribs of the Saikai Bridge and Shin Saikai Bridge are sufficiently safe against strong earthquakes. The Saikai Bridge is safe against strong earthquakes with their cross sections of structural members at the time of design.

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