Microtremor Measurements And Earthquake Response Analysis On Urado Bridge, Kochi, JAPAN

Seckin Ozgur CITAK, Michihiro OHORI* & Takeshi NAKAMURA
Earthquake and Tsunami Research Project for Disaster Prevention, JAMSTEC, Yokohama, Japan

Minoru SAKAUE, Shunsuke TAKEMURA & Teito TAKEMOTO**
Earthquake Research Institute, The University of Tokyo, Tokyo, Japan

Takashi FURUMURA, & Kazuhisa IWAI***
Graduate School of Interdisciplinary Information Studies, The University of Tokyo, Tokyo, Japan

Atsuki KUBO, Kazuo KAWATANI & Sawa TAJIMA
Faculty of Science, Kochi University, Kochi, Japan

Narumi TAKAHASHI & Yoshiyuki KANEDA
Earthquake and Tsunami Research Project for Disaster Prevention, JAMSTEC, Yokohama, Japan

** Now at University of Fukui, Tsuruga, Japan*
*** Now at Yokohama City University, Yokohama, Japan*
**** Now at Japan Patent Office, Tokyo, Japan*

SUMMARY:
In this paper we introduce natural vibration properties of Urado Bridge based on two-day microtremor measurements and its dynamic behaviour to the earthquake excitation. Urado Bridge is located at the pacific coast of Kochi city in Japan and it shall undertake extensive role of being an evacuation and escape spot for the population in its vicinity from tsunami threat in case of occurrence of Nankai Earthquake. The bridge was completed in 1972 and has five-span continuous PC box girder superstructure. The central span is 230 meters long and consists of 115-meter-long cantilever girders connected at the center with sliding hinges. The numerical model was created by referring to the design drawings and a verification analysis has been conducted to satisfy the mode shapes and predominant frequencies which were obtained from the microtremor measurements. The numerical model was analyzed for scenario Nankai Earthquake simulation wave data.

Urado Bridge, microtremor measurement, earthquake response analysis, natural vibration properties

1. INTRODUCTION

Subduction of the Philippine Sea plate beneath the overlying Eurasian plate has attributed to Magnitude 8 class mega-thrust repeating earthquakes along the Nankai trough (**Figure 1.a**) at sequence interval of about 100-150 years. The sequence of earthquakes occurring along the Nankai trough is called the Tokai, Tonankai, and Nankai earthquakes (**Figure 1**), which has a history to cause severe widespread damages especially in Western Japan (Tsuji, 2003). The Long-Term Evaluations of the Earthquake Research Committee, The Headquarters for Earthquake Research Promotion of Japan (2012), indicate high occurrence probability within the next 30 years (as of January 1, 2011) with 87% for the Tokai earthquake, 70% for the Tonankai earthquake, and 60% for the Nankai earthquake. Moreover, it has been pointed out that according to historical studies, these mega-thrust earthquakes might occur as a seismic linkage (Tsuji, 2003). To understand the possibility of seismic linkage and re-occurrence of these earthquakes, estimation of seismic motion and tsunami heights and damages, large-scale research projects have been conducted by Ministry of Education, Culture, Sports, Science and Technology, Japan (Special Project for Earthquake Disaster Mitigation in Urban Areas (2002-2006), Research for the Tonankai and Nankai earthquakes (2003-2007) and the project for seismic linkage for Tokai, Tonankai and Nankai earthquakes(2008-2012)).

Kochi city, where Urado Bridge, the subject of this research is located, is one of the areas where serious damages are expected in the case of occurrence of the great earthquakes along the Nankai Trough (**Figure 1**). When considering Kochi city's recovery and reconstruction strategies after the earthquakes, Urado Bridge shall undertake extensive role of being an evacuation and escape spot for the population in its vicinity from tsunami threat. The bridge connecting two sides of Kochi bay across the sea lines (gate to the Kochi bay) will be the only route to access to the bay and to the Kochi city (**Figure 1.b**). Therefore, damaging of Urado Bridge will intercept the accessibility to Kochi bay from the sea as well as passages and transportation between the two sides of Kochi bay (Examination of concrete measures to ensure emergency transportation routes, Summary of estimated FY 2010 budget for Nankai earthquake countermeasures).

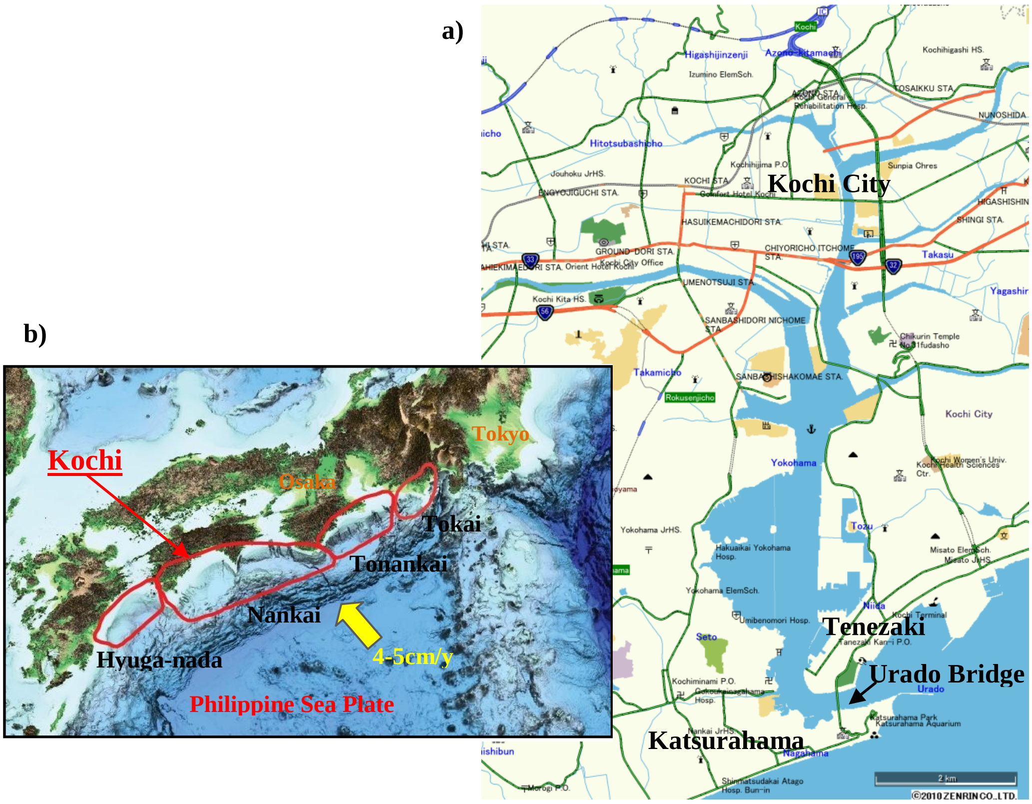

Figure 1. a) Map of the entire Nankai Trough area. b) Locations of Kochi city, Urado Bridge and its vicinity

Urado Bridge, when completed in 1972 was a toll road. According to the survey results of Kochi Prefecture, the traffic volume of a day was 10,439 units in 1999, however, since the bridge became non-toll public road in 2002, the traffic volume drastically increased and reached to 21,749 units a day as of 2005. The estimated average traffic volume in recent days is over 30,000 units a day. Consequently, being in service for 40 years, accumulated fatigue damage due to the increase in traffic loads and volume as well as the corrosion of the internal steel reinforcement might have resulted in severe deterioration of the bridge structure (Kato et al., 2010, Mihashi et al., 2010). Due to the above facts, an evaluation of seismic safety conducted by Kochi Prefecture (Summary of estimated FY 2010 budget for Nankai earthquake countermeasures) is in process and if necessary, there will be an implementation of seismic strengthening of the bridge.

The authors aimed to contribute to the evaluation of seismic safety of Urado Bridge by obtaining the fundamental dynamic data. In order to understand the vibration characteristics the simultaneous microtremor observations using different measurement schemes were carried out on the bridge and its vicinity. A FEM model was created by referring to the design drawings and a verification analysis was conducted to satisfy the mode shapes and predominant frequencies which were obtained from the microtremor measurements. Then the FEM model was used to perform earthquake response analysis for the hypothetical Nankai Earthquake simulation wave data.

2. MICROTREMOR OBSERVATIONS ON URADO BRIDGE

Urado Bridge is located at the entrance to Kochi bay from Pacific Ocean and connects two sides of Kochi city – Tanezaki district on the North and Katsurahama and Urado districts on the South – with a total length of 915m. The location of Kochi city, Urado Bridge and their vicinity are shown in **Figure 1.b** The bridge is a five-span continuous PC box girder superstructure 600 m in length. The central span is 230 meters long and consists of 115-meter-long cantilever girders connected at the center with sliding hinges (**Figure 2**).

Figure 2. Urado Bridge (The picture was taken from North East side of the bridge -Tanezaki district-.)

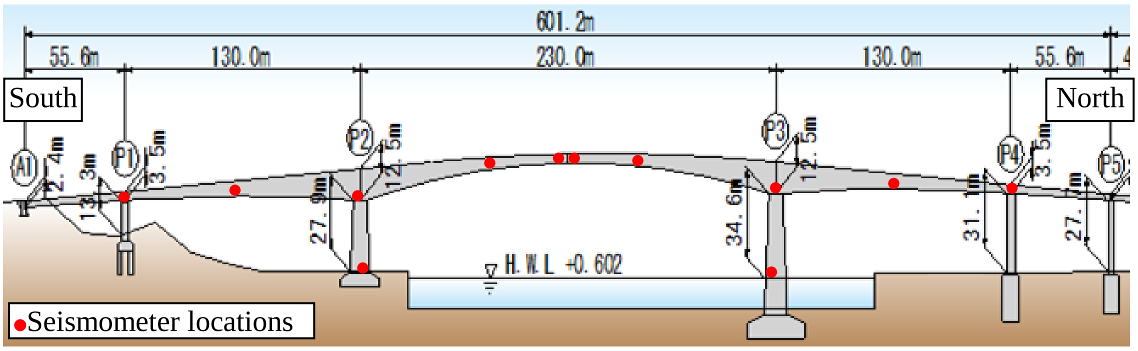

Figure 3. Microtremor observation locations on Urado Bridge (Scheme 1)

The PC box girder is 8m width with two 6.5m lanes and two 0.75m narrow sidewalks (pedestrian) at the both sides. The height of the box girder varies from 2.4m at the center and edges to 12.5m at the main piers. The bridge consists of two sections – the North and the South. Each section consists of 115-meter-long cantilever box girders, main piers (P2 in the South section is 27.9m high measured from the ground level, and P3 in the North section is 49.4m high measured from the sea bed level), 185-meter-long box girders which are supported by two friction sliding bearings at the end and in the middle of the girders. The friction sliding bearings are situated on an abutment (A1) and a pier (P1) in the South section and two piers (P4, P5) in the North section. **Figure 2** shows the central span and P2, P3 piers of Urado Bridge.

In order to understand the dynamic characteristics of Urado Bridge, microtremor observations were conducted using 12 seismographs on 18th and 19th of November, 2010 (**Figure 3**). All 12

Figure 4. Microtremor observation conditions (Left: inside view of box girder. Middle: observation above the box girder. Right: P3 pier which is located in the sea)

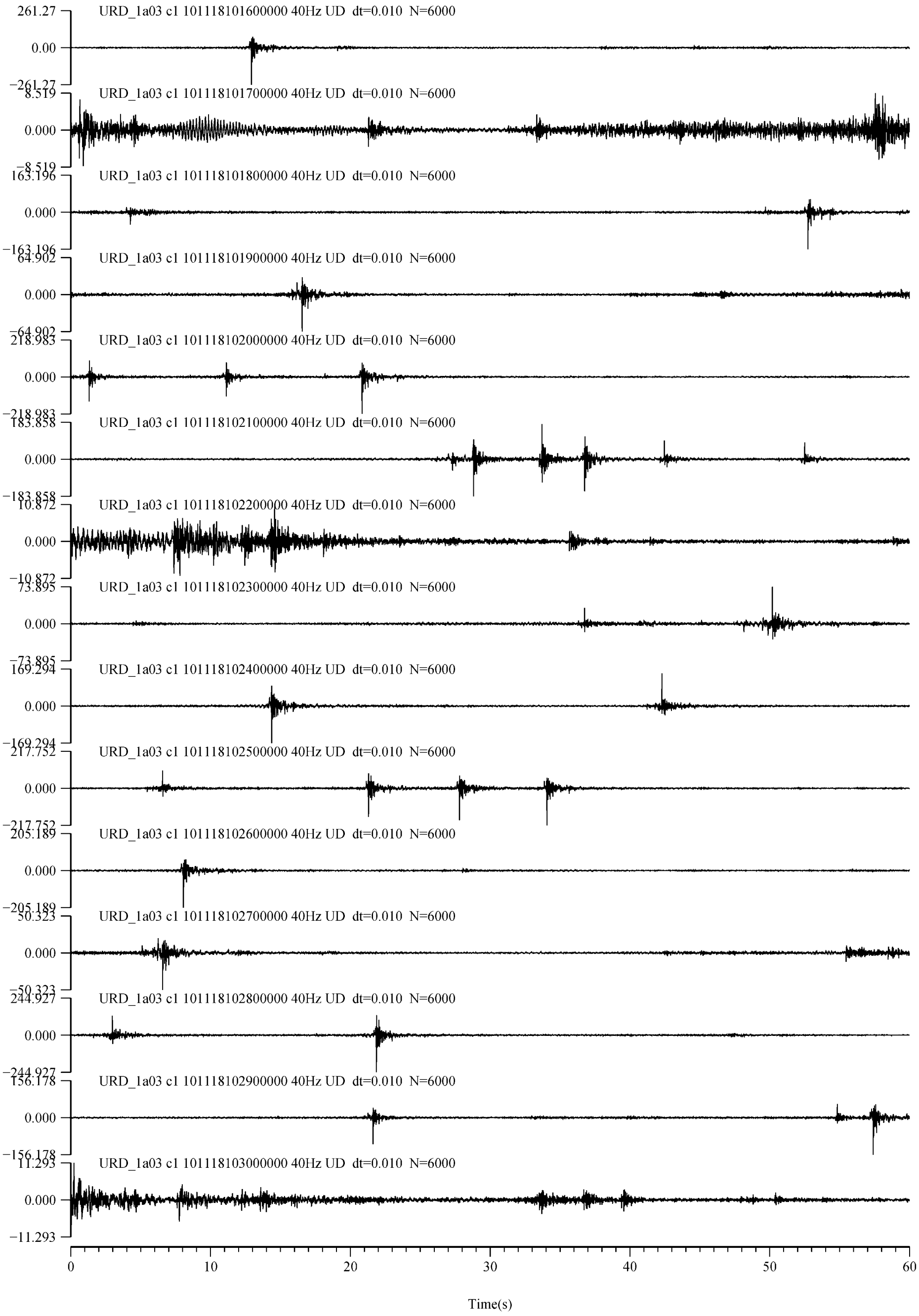

Figure 5. An example of the observed waveform near the center hinge, South side (Gal)

seismometers had LS-7000XT (Hakusan Co., Ltd.) type of 24 bit A/D resolution data loggers with GPS time correction. As for the sensors 11 of those JEP-6A3 (Mitutoyo Corporation) and one JA-40GA04 units (Japan Aviation Electronics Industry Co., Ltd.) both are high resolution strong motion accelerometers were used.

The majority of observations were carried out inside the box girder superstructure due to the several reasons, which are difficulty to enforce the traffic control, consideration of staff safety, convenience of transportation and installation of the equipments. Few observations were carried out above the box girder along the narrow sidewalk, on the pier P2 and P3 foundations (the pier P3 is located in the sea and a charter boat was used for access), and the pier P1, P4 and P5 ground level (**Figure 4**).

Three different observation schemes were adopted. In the first scheme, where overall dynamic behaviour of Urado Bridge was aimed at, simultaneous observations on both North and South sections were conducted. In the Second and Third schemes, where detailed vibration characteristics of the bridge was aimed at, the seismometers were installed densely within each section. Within each scheme further comprehensive observations were conducted by changing location of seismometers. In all cases, three components of 30 minutes data with 100 Hz sampling frequency are recorded. Since we focus on general dynamic characteristics in this paper only scheme one is introduced here (**Figure 3**). In addition to the observations conducted on the bridge we performed array microtremor measurement at the vicinity of the site (Tanezaki area) in order to estimate S wave velocity structure of the sub-soil that was used to calculate site amplification for the input ground motion.

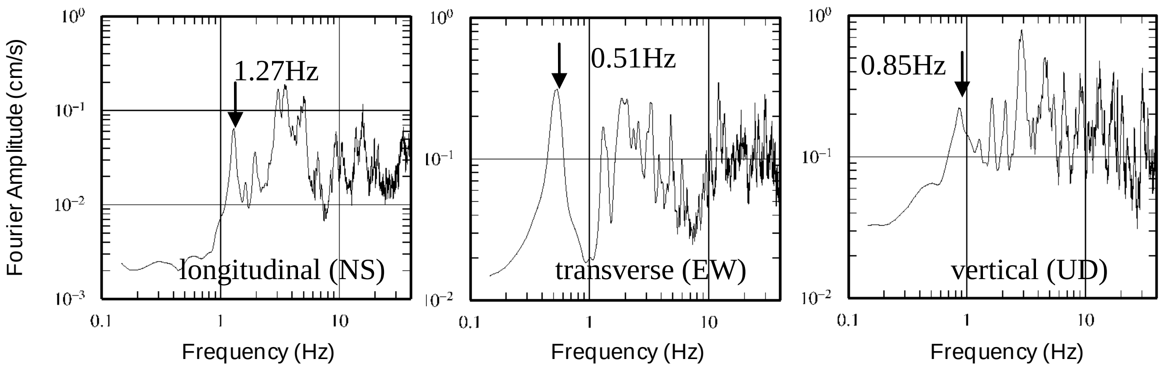

Figure 6. An example of the Fourier spectra of the center hinge microtremor observation results

3. DYNAMIS CHARACTERISTIS OBTAINED FROM MICTOTREMORS

Figure 5 shows examples of waveform measured at the center hinge on the north section. The figure shows the first half of 30 minutes vertical component acceleration measurements. The structure of the center hinge causes strong impact loading as well as impact sound whenever large and heavy vehicle passes over the hinge from one side to the other. In the 15 minutes span of the example recordings (**Figure 5**) there are five that exceed 200 Gal in vertical motion. Except for contamination from large vehicle passing, in general the microtremor measurements of the heavy traffic was within several Gals. In the analysis we excluded the contaminated parts and chose more than ten 20.48-sec data windows, which were zero padded to 40.96-sec and their average Fourier spectra were calculated. An example of the Fourier spectra at the center hinge part is shown in **Figure 6** with clear peaks at 0.51 Hz for transverse (EW), 0.85 Hz for vertical (UD), and 1.27 Hz for longitudinal (NS) direction of Urado Bridge. When spectral results from the overall measurements are compared, common peaks at different location can be detected, which are the vibration mode frequencies (see **Table 1**). **Figure 8** shows the vibration mode shapes obtained from band-pass filtered data (band pass was chosen to be centred at the predominant frequency). In the same figure the measured mode shapes are compared with those obtained from the numerical analysis. The fundamental frequencies of the center span of Urado Bridge are estimated to be 0.51, 0.85 and 1.27 in transverse, vertical and longitudinal directions, respectively. These three predominant frequencies correspond to modes 1, 2, and 4 in **Table 1**. In the same table, mode 3 was obtained in the transverse (EW) direction between piers P3 and P5. As for modes 4, 5, and 6 a coupled dynamic behaviour was observed from the microtremor observation in all directions.

4. NUMERICAL MODELING AND MODE ANALYSIS

The numerical model was created based on the design drawings and used for mode and seismic response analysis. The overall modeling flow is shown in **Figure 7**. Hand written design drawing documents (Kochi Prefecture Road Division) were carefully reproduced and digitalized using AutoCAD software, which were subsequently used to construct analysis model. SAP2000, one of the widely used static and dynamic finite element code was utilized in the analysis. Four-node shell elements (Wilson, 2002) were used for modeling of box girder superstructure, gap elements (Wilson, 2002) with clearance opening of 4-14 cm were used to model the hinge part of the bridge. The clearance opening length was measured several times during different seasons with different weather conditions. The bridge box girders are connected to the four main piers via friction bearings which were modeled using multi linear elements (Wilson, 2002) of analysis code. The material properties of concrete were chosen based on design drawing information (Young's modulus was set to 30,299 N/mm^2, 26,416 N/mm^2 and 24,523 N/mm^2 for box girder, P2 and P3 pier and the rest of the bridge, respectively). In this study for the superstructure of the bridge to save the calculation costs, we performed only linear analysis, thus increase of stiffness due to the conventional and pre-stress reinforcement was not considered. The damping coefficient was calculated using RD method (Tamura, 1993) to be 3%, which is within with generally accepted 3-5% damping value.

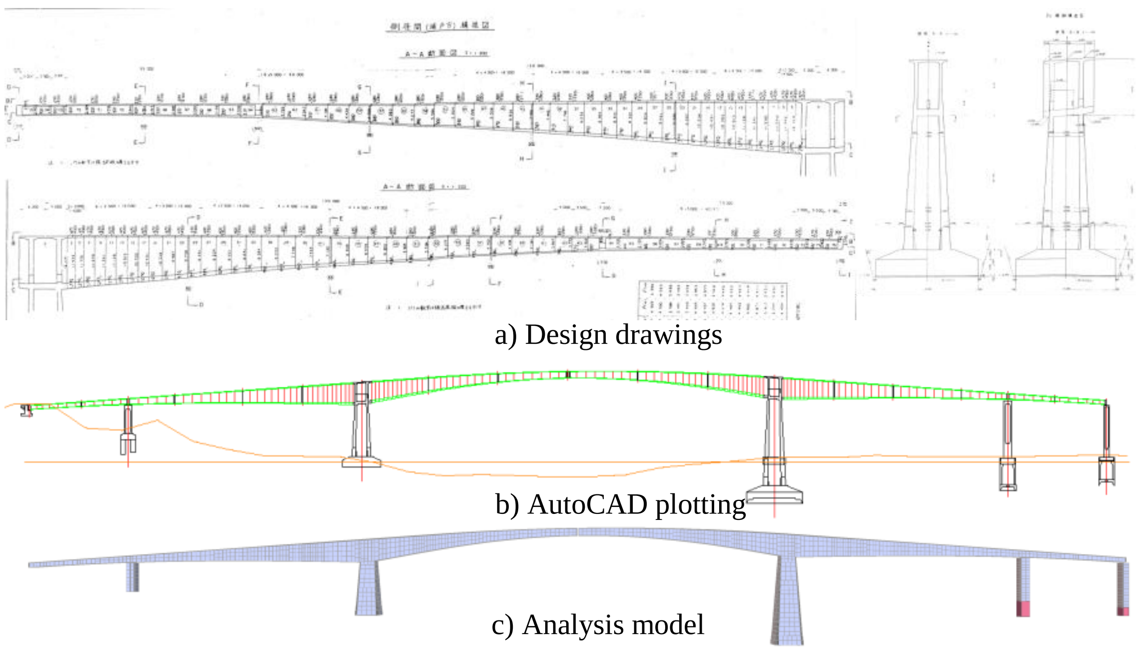

Figure 7. Flow chart of construction of the analysis model (a) referred design drawings. b) digitalised AutoCAD plotting. c) Sap2000 analysis model)

The modal analysis results are given in **Table 1** and **Figure 8**, and are compared with the observed ones. It can be seen from **Table 1** where first seven predominant frequencies are presented, the analysis mode frequency values are slightly lower than those of the observed ones, which might be caused by the inherent property of microtremor measurement to consider small amplitude vibrations, as well as by the neglecting of reinforcement stiffness in the numerical analysis. As for the mode shapes, the consistency of the first 3 mode shapes for central span can be seen from **Figure 8**.

Table 1. Comparison of obtained from measurements and analysed predominant frequencies (Hz)

Mode No	Mode shape	Observed	Analysed	Observed/Analysed
1	(EW) transverse	0.51	0.46	1.11
2	(UD) vertical	0.85	0.63	1.35
3	(EW) transverse	1.07	1.04	1.03
4	(NS) longitudinal	1.27	1.10	1.15
5	(EW) transverse	1.27	1.19	1.07
6	(UD) vertical	1.27	1.31	0.97
7	(EW) transverse	1.31	1.32	0.99

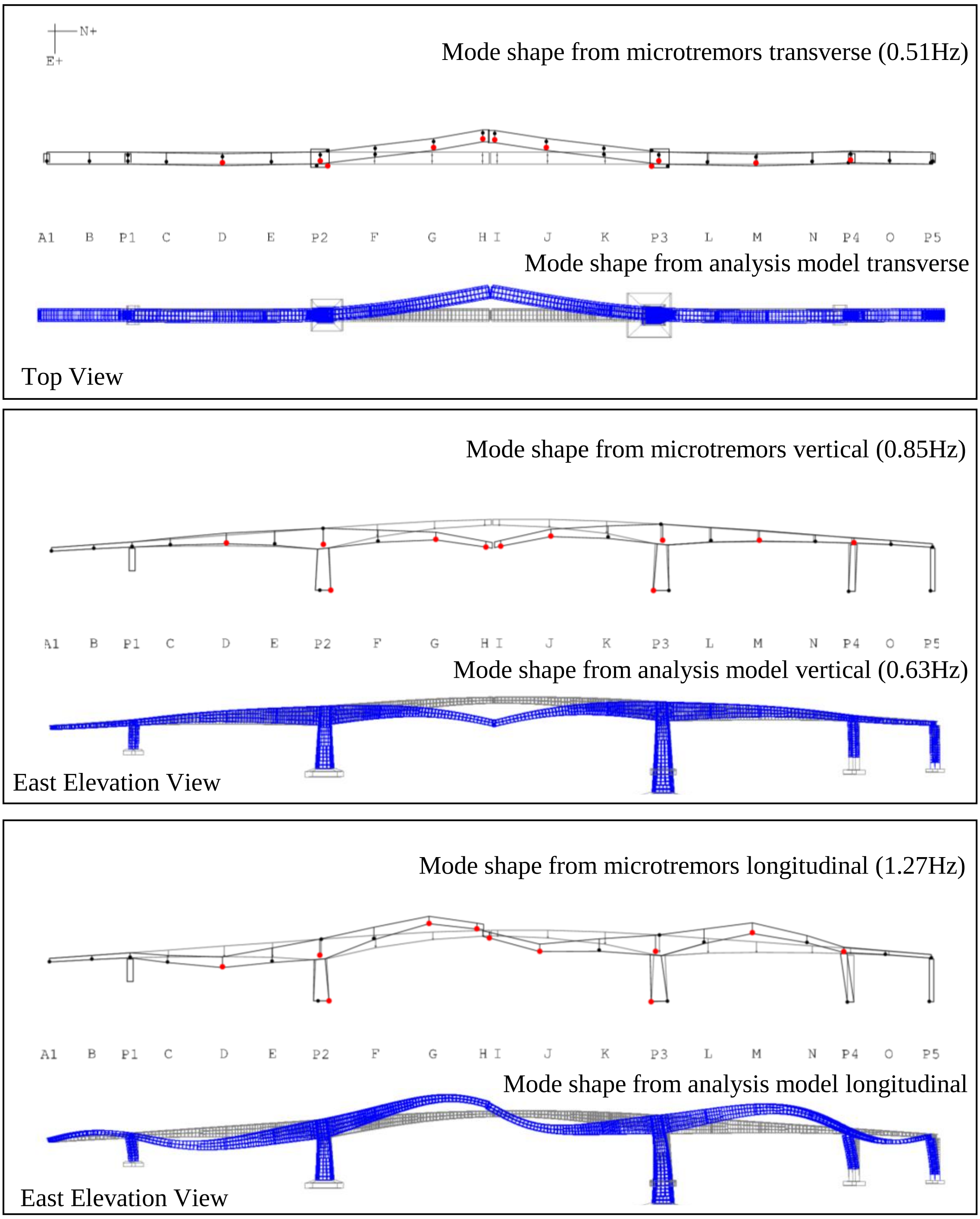

Figure 8. Comparison of mode shapes obtained from the microtremor measurements and the analysis (Top: transverse (EW) direction, Middle: vertical (UD) direction, Bottom: longitudinal (NS) direction)

5. EARTHQUAKE RESPONSE ANALYSIS

In the earthquake response analysis we used hypothetical Nankai earthquake simulated waveforms (Central Disaster Prevention Council, 2003), which are shown in **Figure 9**. The maximum acceleration in the NS direction (the longitudinal direction of Urado Bridge) is 395 gal and JMA intensity is 5.57 (**Table 2**). In the same figure we show pseudo response spectrum where response peaks are within the frequency band of interest as was shown in **Table 1**. Although the box girder superstructure was modelled with linear elements, we did perform nonlinear time history direct integration analysis using above mentioned waveforms, in order to contribute nonlinear effects of sliding hinges at the center and friction sliding bearings between the piers and the box girder. The resulting displacement responses are shown in **Figure 10** for the north side of the center sliding hinge and top of piers P3 and P4. In the figure we can see in the transverse direction (East-West) a long duration oscillation with period of around 2 sec. and 45 cm maximum displacement.

Figure 11 shows the overall displacement and compression stress distribution at the time of maximum compression. The displacement patterns are different for the North and the South sections, due to differences of pier heights. The maximum compression is detected at a distance of about 20 ~ 30 m from the center hinge on the North section, where the stresses repeatedly reach 70% of the concrete material compressive strength (400 Kg/cm^2). We have to note that at the place of the maximum compression maximum tension stresses of the same amplitude occur.

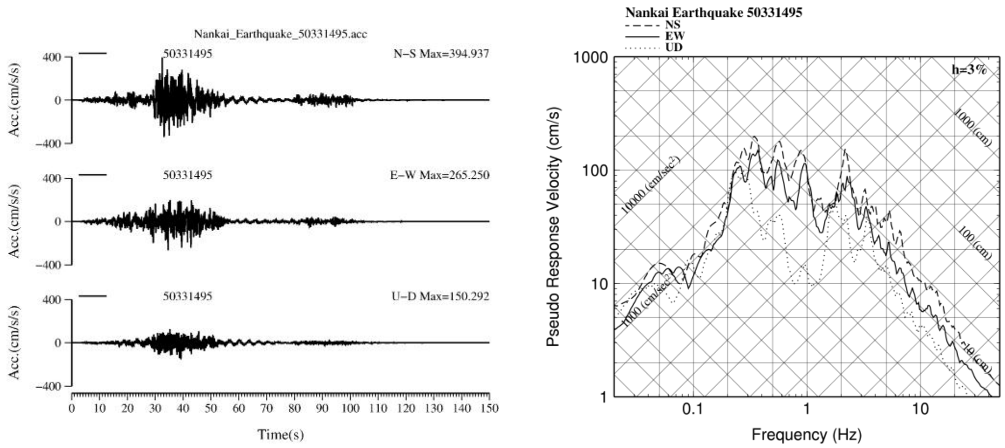

Figure 9. Input ground motion used for the analysis (Central Disaster Prevention Council, 2003). (Left, Acceleration wave forms for three directions, Right: Tripartite logarithmic plot of pseudo response spectrum for three directions)

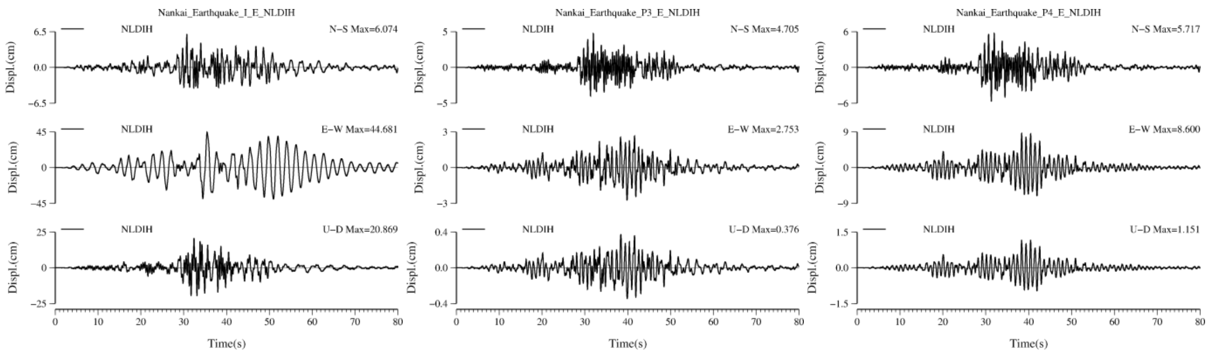

Figure 10. Response displacement waveforms (Left: center hinge (North), middle: pier P3 and pier P4)

As for the center hinge clearance distance, the analysis results show that the minimum clearance shall be at least 6.3 cm. However due to the seasonal changes of the clearance distance as it was noted before, the pounding of the two sections might cause exceeding stress levels around the hinge for smaller clearances. In the analysis the relative displacement at the center hinge in longitudinal direction was calculated to be 7.55cm, which would cause pounding (**Table 2**).

Also an earthquake response analysis was conducted for a wave form Nankaiβ, (**Table 2**). The waveform calculated (amplified from engineering bedrock to the ground surface) using newly estimated sub velocity structure based on microtremor array measurement performed at Tanezaki around 500m north of Urado Bridge (Ohori, 2012). A summary of the input waveforms and the analysis results, [displacements and stress responses] are shown in **Table 2**. The analysis results show that the local site conditions have significant effect on the increased responses. Especially the maximum compression stress reaches 95% of concrete material compressive strength.

Table 2. Summary of input wave forms and analysis results

Wave Form	Maximum acceleration (gal)			JMA Intensity	Center hinge maximum displacement (cm)			Center hinge relative displacement (cm)	Maximum compression stress (kg/cm^2)
	NS	EW	UD		NS	EW	UD	NS	
Nankai	394.9	265.3	150.3	5.57	6.07	44.68	20.87	7.55	278.5
Nankaiβ	482.3	335.7	176.8	5.84	9.35	49.60	28.90	8.64	383.6

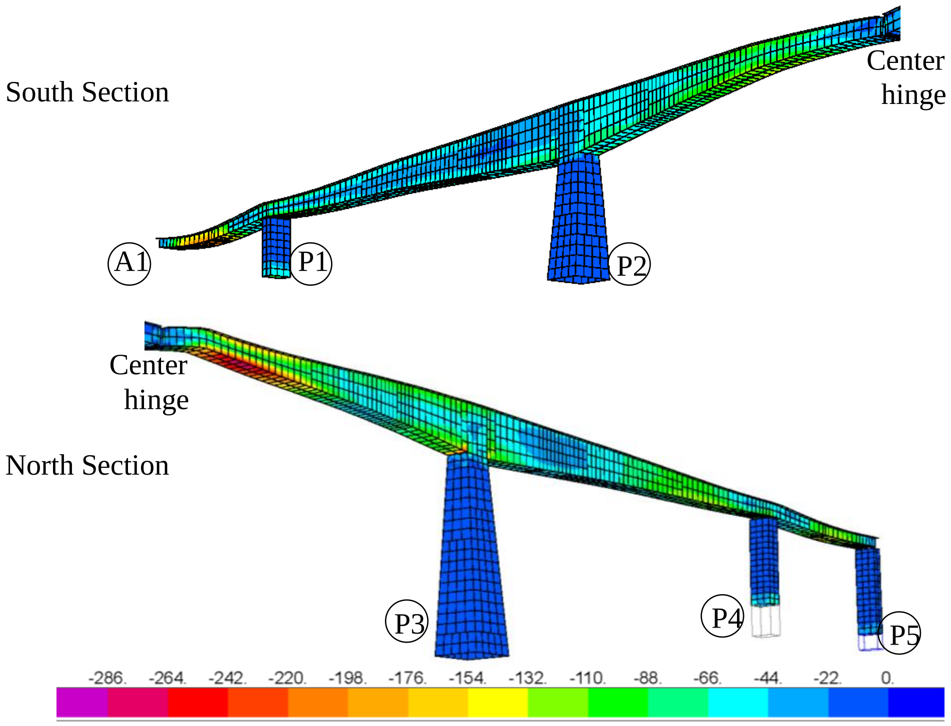

Figure 11. Distribution of maximum compressive stress (Top: South section, Bottom: North section) (kg/cm^2)

6. CONCLUSION

Microtremor measurements were conducted and dynamic properties (predominant frequencies, damping coefficient, and mode shapes) determined for Urado Bridge, one of the most important infrastructure in Kochi city, Japan, where severe damages are expected in case of Nankai earthquake. Numerical model of the bridge was constructed based on design drawings and modal analysis was performed to obtain the basic dynamic characteristics. The results from the modal analysis and from the measurements were similar. Next, earthquake response analyses were conducted using hypothetical Nankai earthquake simulated waveforms (Central Disaster Prevention Council, 2003). The results show that the maximum compression is detected at a distance of about 20 ~ 30 m from the center hinge on the North section, where the stresses repeatedly reach 70% of the concrete material compressive strength (400 Kg/cm^2). Also there is possibility of exceeding stress levels around the center hinge due to pounding of the two sections (North and South) when the clearance of the hinge is less than 6.3 cm. Urado bridge, which has been in service for 40 years, accumulated fatigue damage due to the increase in traffic loads and volume as well as the corrosion of the internal steel reinforcement which might have resulted in severe deterioration of the bridge structure. Therefore, more detailed seismic resistance evaluation of the bridge is crucial.

AKCNOWLEDGEMENT

We are grateful to Kochi Prefecture, Kochi city and other related institutions for their assistance during the measurements. We would like to thank the Committee of Kochi Geo-Hazard Evaluation for providing us with geological cross-section data and sub-layer information. Also, we would like to acknowledge The Central Disaster Prevention Council, Japan for allowing us to use simulated strong ground motion data for Nankai Earthquake. Some figures were made using GMT (P. Wessel and W. H. F. Smith, 1999)

REFERENCES

Central Disaster Prevention Council, Japan, (2003), Tonankai and Nankai Earthquake investigation committee, Referance document for Subsurface Structure 2-3, http://www.bousai.go.jp/jishin/chubou/nankai/16/sankousiryou2_3.pdf

Committee of Kochi Geo-Hazard Evaluation (2011), http://www.geonews.jp/kochi/index.html

Examination of concrete measures to ensure emergency transportation routes, Shikoku Regional Development Bureau, Ministry of Land, Infrastructure and Transport, Japan. http://www.mlit.go.jp/common/000109019.pdf

Headquarters for Earthquake Research Promotion. (2012), Overview of long-term evaluation of subduction earthquakes. http://www.jishin.go.jp/main/choukihyoka/ichiran.pdf

Kato, E., Kawabata, Y., and Iwanami, M. (2010). Probabilistic Aproach for Structural Performance Evaluation of Port RC Strucrure. *Technical Note Of The Port And Airport Research Institute* 1225.

Mihashi, H., Rokugo, K., and Kunieda M. (2010). Fracture Mechanics And Cracks In Concrete, Gihodo publication, Japan.

Ohori, M., Citak, S., Nakamura, T., Sakaue, M., Takemura, S., Furumura, T., Takemoto, T., Iwai, K., Kubo, A., Kawatani, K., Tajima, S., Takahashi, N., and Kaneda, Y. (2012). Modeling the Shallow Subsurface Structure in the Urban Area of Kochi City. *Jurnal of Japan Association for Earthquake Engineering.* (Submitted)

Research for the Tonankai and Nankai earthquakes (2003-2007), Ministry of Education, Culture, Sports, Science and Technology, Japan. http://www.mext.go.jp/a_menu/kaihatu/jishin/04031204.htm

Special Project for Earthquake Disaster Mitigation in Urban Areas (2002-2006), Ministry of Education, Culture, Sports, Science and Technology, Japan. http://www.mext.go.jp/a_menu/kaihatu/jishin/04031203.htm

Summary of estimated FY 2010 budget for Nankai earthquake countermeasures, Secretariat of the Nankai earthquake Prevention Headquarters in Kochi Prefecture, Japan. http://www.pref.kochi.lg.jp/uploaded/attachment/20076.pdf

Tamura, Y., Sasaki, A., and Tsukagoshi, H. (1993). Evaluation Of Damping Ratios Of Randomly Excited Buildings Using The Random Decrement Technique. Journal of structural and construction engineering. Transactions of AIJ: 454, 29-38.

The project for seismic linkage for Tokai, Tonankai and Nankai earthquakes(2008-2012), Ministry of Education, Culture, Sports, Science and Technology,Japan. http://www.mext.go.jp/b_menu/houdou/20/04/08040308.htm

Tsuji, Y. (2003). Disaster Research Forum 2002 Lecture “ Great Earthquake Disaster Scenarios, Tokai, Tonankai and Nankai earthquake -What happens at that time-”, *Lessons learnt from earthquake disasters scenario from past Tokai, Nankai Earthquakes, Non-Life Infrastructure Rating Organization of Japan*, Earthquake Insurance Research : 2, 21-31, 71-76. (In Japanese)

Wilson, E.L., Three Dimensional Static And Dynamic Analysis Of Structures, Computers and Structures, Inc. Berkeley, California, USA, 2002.