Seismic Structural Performance of the Prambanan World Heritage Temple Damaged by The Central Java Earthquake of 2006 in Indonesia

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SUMMARY
The Central Java earthquake of May, 2006 caused severe damage to the World Heritage Prambanan Temple Compounds. For proposing the restoration plan, Japanese experts have been involved in the architectural structural survey with Indonesian experts. In order to assess the structural seismic safety, the structural monitoring, earthquake and crack displacement, has been conducted as an international collaborative project. The main scope of the present paper is to show the seismic behaviours on the basis of the earthquake data, recorded on 21st September 2010, on 24th November 2011 and on 19th March 2012 at the Siwa Temple, the largest structure of the Prambanan Temple Compounds. The earthquake response analysis was also conducted to simulate the dynamic response to the event of September 2010. In the present analysis model, dynamic soil-structure interaction was taken into account under the condition that the structure was so massive and comparatively rigid. The analysis demonstrated successfully that the soil-structure interaction had significant effect on the structural response. Moreover, the structural stability was discussed on the basis of the crack and temperature monitoring data.

Keywords: World Heritage, Stone Structure, Earthquake Monitoring, Soil-structure Interaction

1. INTRODUCTION

A devastating earthquake of magnitude 6.4 that hit Central Java in Indonesia on 27th May 2006 affected a number of architectural heritages in the area of Yogjakarta, one of the historic cities in Indonesia. This near field earthquake caused serious damage to the Prambanan Temple Compounds, World Heritage, being located 25km from the epicentre [1]. For proposing the restoration plan, Japanese team has been involved in the architectural structural survey of the earthquake damage as an international collaborative project since just after the earthquake. In this international project, seismic structural monitoring has been conducted to understand the actual seismic behaviours of the structure, as well as, to assess the structural seismic safety. As structural seismic monitoring, earthquake and crack/temperature have been monitored.

The present study deals with the Siva Temple of which height is 47m, the largest one of 8 monuments of the Prambanan Temple. This apparently masonry structure is characterized by the composite one combining the inner reinforced concrete covered by the exterior andesite stone walls with decorative ratona/stupa stones. Such composite structure was introduced when the monument was reconstructed by Dutch engineers in 1950’s. However, the inner structural condition of the Siva Temple has been unknown so far, although that of the other monuments in the Prambanan site was described in the drawings at the reconstruction in the end of 1980’s.

2. EARTHQUAKE MONITORING

2.1. Outlines

It is essential to understand the actual seismic behaviours of Prambanan Temple, as the inherent characteristics of the earthquake response should be considered to discuss the restoration plan. Monitoring of earthquake response was started at the Hangsa Temple on October 2007. However, the restoration works started at Hangsa Temple in July 2010, therefore, it was needed to remove the seismograph at Hangsa Temple at that time. After new monitoring system was re-installed at the Siva Temple, earthquake monitoring was restarted at the Siva Temple. Shown in Fig. 1, strong-motion seismographs, were installed at 4 points, Top, Middle, Base and Ground. Three components of EW, NS and UD at every point have been recorded.

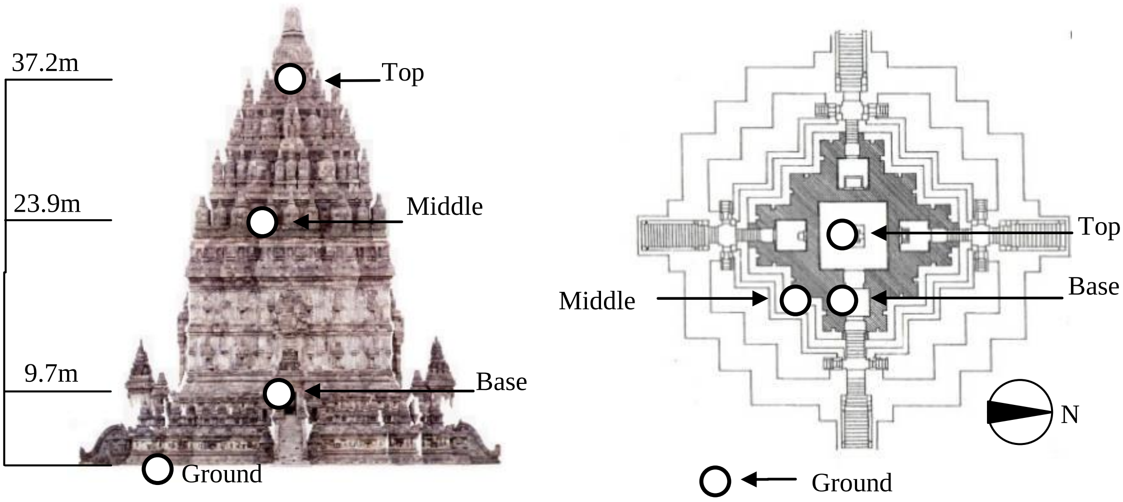


Figure 1 Location of seismograph installed

2.2. Monitoring Records of Earthquake

During the earthquake monitoring period, three earthquakes occurred in the vicinity of Yogjakarta on 12th September 2010, 24th November 2011 and 19th May 2012, listed in Table 1. It can be noticed in this table that the focal depth of 2010 events was much deeper than those of 2011and 2012 events. The earthquake data of those events were successfully recorded at the Siva Temple. The acceleration wave forms and acceleration response spectra of the ground motions of three earthquakes are shown in Figs. 2 through 5. The predominant period of response spectra (See Fig.5) , the seismic response displacement (See Table 2) and the natural frequencies (See Table 2) show that the frequency characteristics of the ground motions of 2010 are different from both those of 2011 and of 2012. Those records show that the natural frequency of soil-structure system was slightly affected by the ground motion characteristics. Furthermore, they show that the natural frequency become longer with increase of the response displacement.

Table 1 Three Earthquakes Recorded

Day	Local Time	M_L	Depth(km)	Epicentral Distance(km)	MMI*	PGA ** (cm/s^2)
12th Sep. 2010	23:37	5.0	48	11	Ⅳ	8.6
24th Nov. 2011	10:55	5.1	10	165	Ⅲ	1.8
19th Mar. 2012	9:19	4.2	10	40	Ⅲ	9.1

*Modefied Mericali Intensity scale estimated by PGA

**PGA : Peak Ground Acceleration

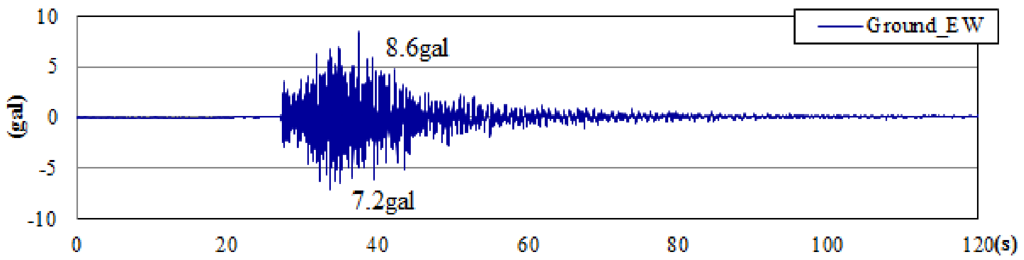


Figure 2 Acceleration wave form of ground motion(12th Sep. 2010)

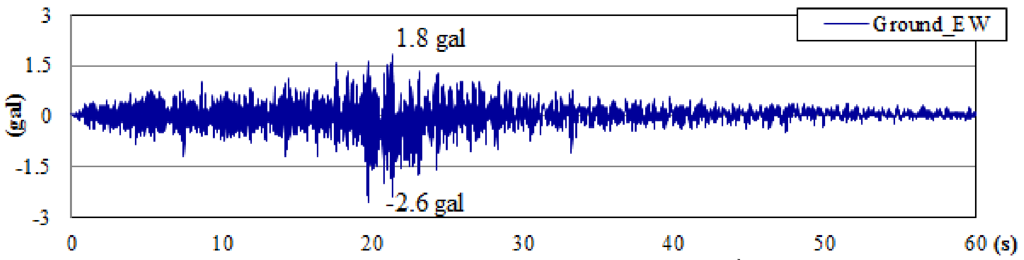


Figure 3 Acceleration wave form of ground motion(24th Nov. 2011)

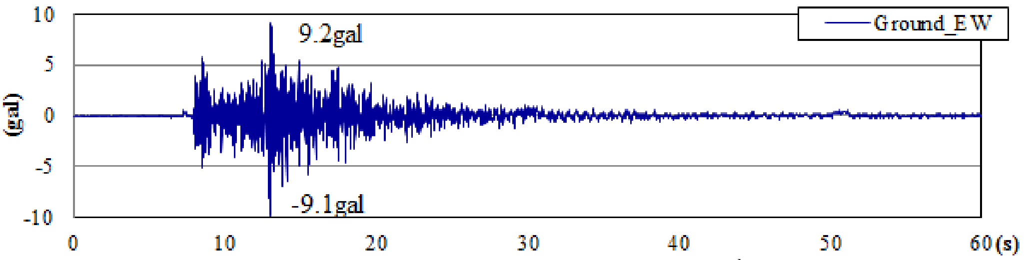


Figure 4 Acceleration wave form of ground motion(19th Mar. 2012)

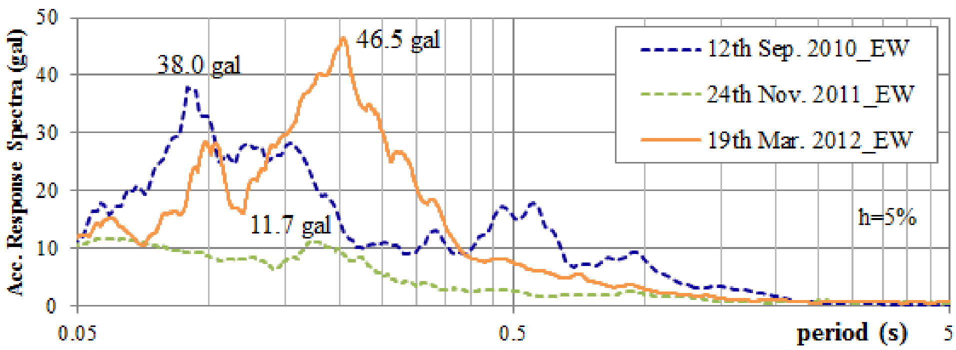


Figure 5 Comparison of Acceleration response spectra of ground motions

Table 2 Peak of Response Displacement (Top, Ground) and Amplitude Factor

		Displacement (cm)					
		12th Sep. 2010		24th Nov. 2011		19th Mar. 2012	
		MAX.	min.	MAX.	min.	MAX.	min.
EW	Top	0.18	-0.22	0.02	-0.02	0.06	-0.04
	Ground	0.06	-0.06	0.09	-0.07	0.08	-0.09
NS	Top	0.02	-0.03	0.02	-0.02	0.02	-0.02
	Ground	0.02	-0.02	0.02	-0.04	0.03	-0.05
Amplification Factor	EW	2.33		0.24		0.80	
	NS	1.14		0.74		0.62	

Table 3 Natural Frequency Evalusted by Transfer Function

		Natural Frequency(Hz)		
		12th Sep. 2010	24th Nov. 2011	19th Mar. 2012
/ground	EW	1.95	2.06	2.01
	NS	1.99	2.05	1.99
/base	EW	2.93	3.35	3.51
	NS	2.67	3.18	3.31

3. MONITORING OF CRACK DISPLACEMENT WITH TEMPERATURE

3.1. Introduction

A lot of cracks were generated at the exterior stone walls and around the entrances to the inner chambers by the Central Java Earthquake. Long term monitoring of these cracks has been conducted to assess structural stability of the monument, at the same time, to make clear the relation between crack displacement and variation of temperature. Therefore, monitoring of crack and temperature has been conducted since October 2008. At the same time, the humidity has been monitored with temperature.

3.2. Method of Monitoring

Shown in Fig. 6, crack displacement has been monitored by using Pi-type displacement gages at 14 points in the inside rooms and on the outside stones wall of the Siva Temple. In addition, 4 pieces of dummy displacement gages were installed just nearby the displacement gages on the outside stone wall. Montoring of temperature and humidity has been conducted by using temperature-humidity loggers in each inner chamber. Monitoring of temperature using thermocouple has been conducted at 2 points, outside and in the central inner chamber. Data logger was put at the central inner chamber and was connected to both the thermocouples and to the Pi-type crack gages.

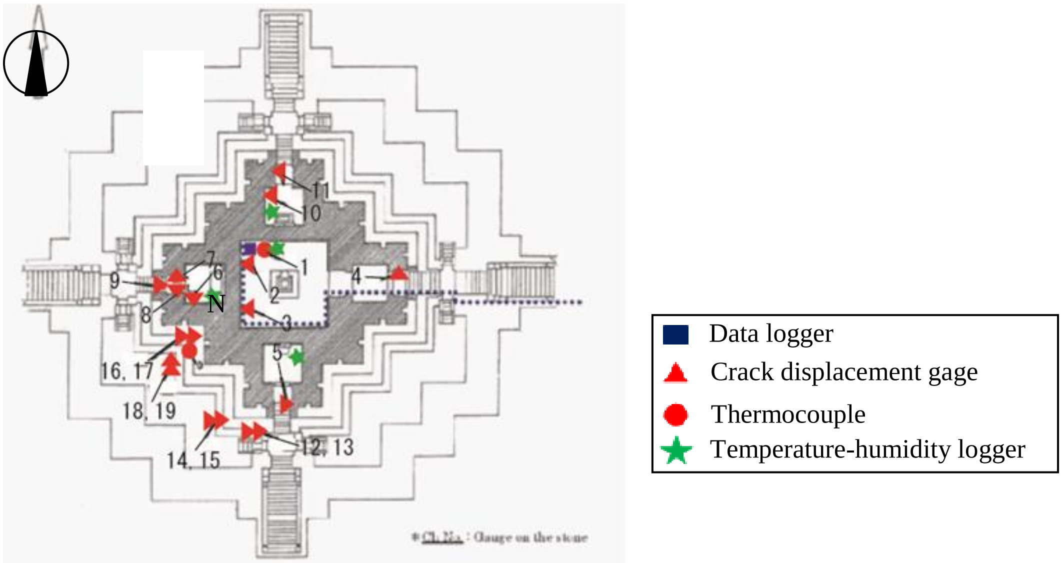


Figure 6 Location of equipment installed

3.3. Results of Monitoring and Discussion

Monitoring of crack displacement has been conducted since 29th October 2008. As well as, monitoring of temperature with humidity has been carried out. The present paper describes the movement of crack displacement and temperature before and after every earthquake. Since the crack displacement was affected by temperature, crack displacement records were corrected to account for the effect of the variation of the temperature. Correlation between movement of crack displacement and variation of temperature should be noticed (See Figs. 7 through 12). Furthermore, those figures show that the movement of crack displacement was not caused at the occurrence time of the three earthquakes. Those monitoring records indicated the structural stability of the Siva Temple at the present state.

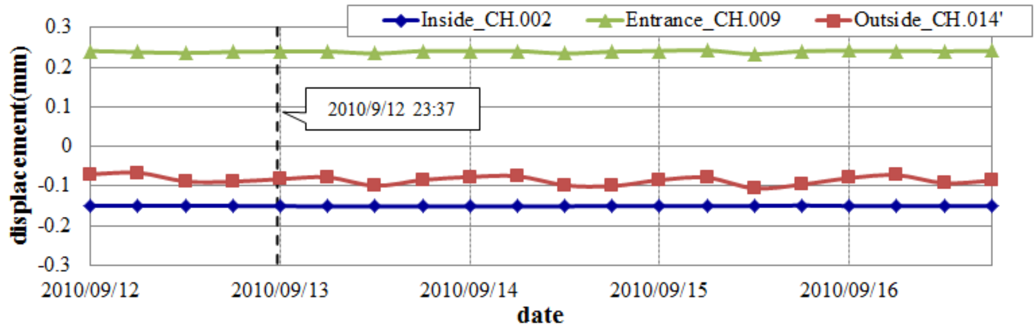


Figure 7 Crack displacement before and after earthquake(12th Sep. 2010)

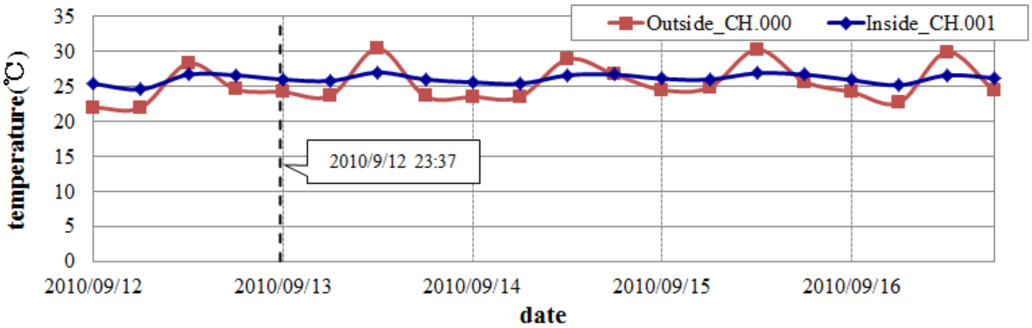


Figure 8 Temperature before and after earthquake(12th Sep. 2010)

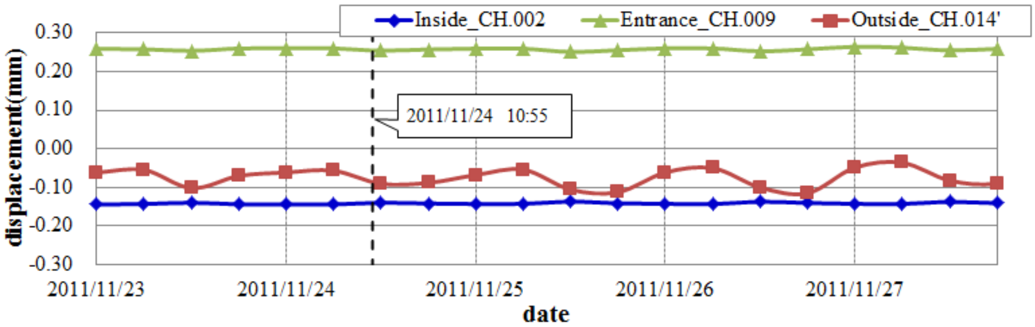


Figure 9 Crack displacement before and after earthquake(24th Nov. 2011)

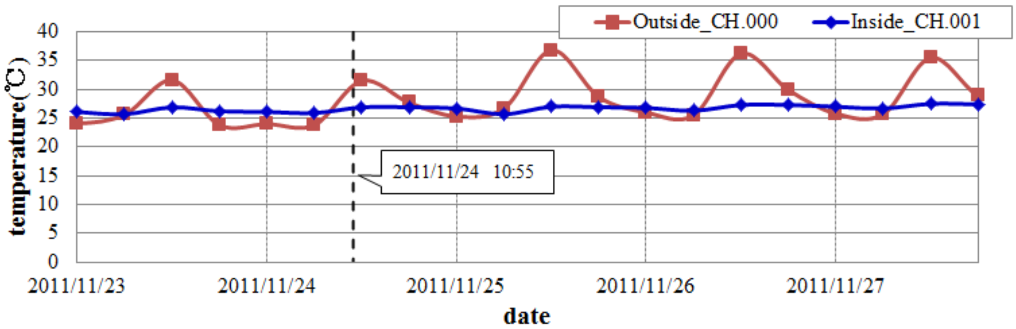


Figure 10 Temperature before and after earthquake(24th Nov. 2011)

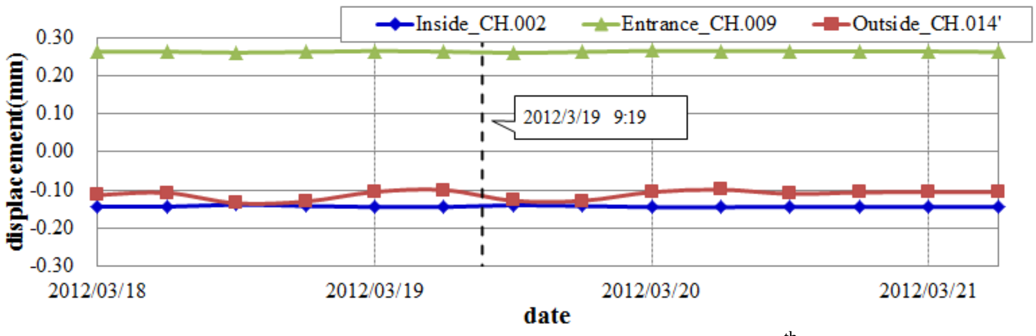


Figure 11 Crack displacement before and after earthquake(19th Mar. 2012)

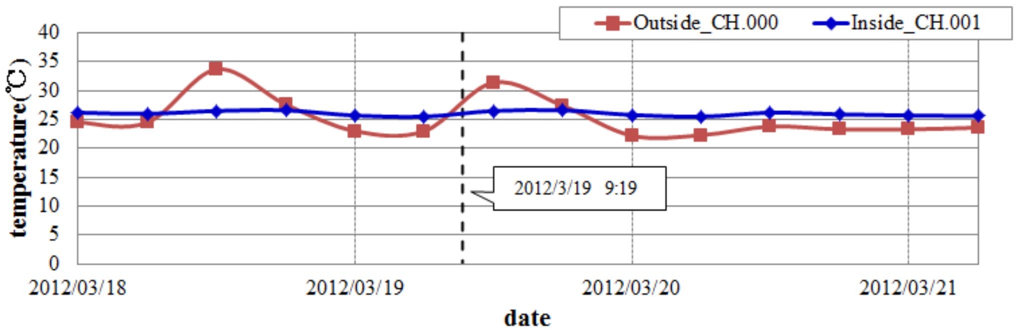


Figure 12 Temperature before and after earthquake(19th Mar. 2012)

4. EARTHQUAKE RESPONSE ANALYSIS USING SIMPLIFIED MODEL

4.1. Introduction

A simplified analysis model, lamped masses model, was introduced to simulate the seismic response of the Siva Temple. By employing the proposed model, the natural period and the vibration mode was evaluated and compared with the earthquake record. For proposing the structural restoration plan, such simulation analysis was so useful not only to understand the actual behaviours but also to evaluate the earthquake resistance of the structure.

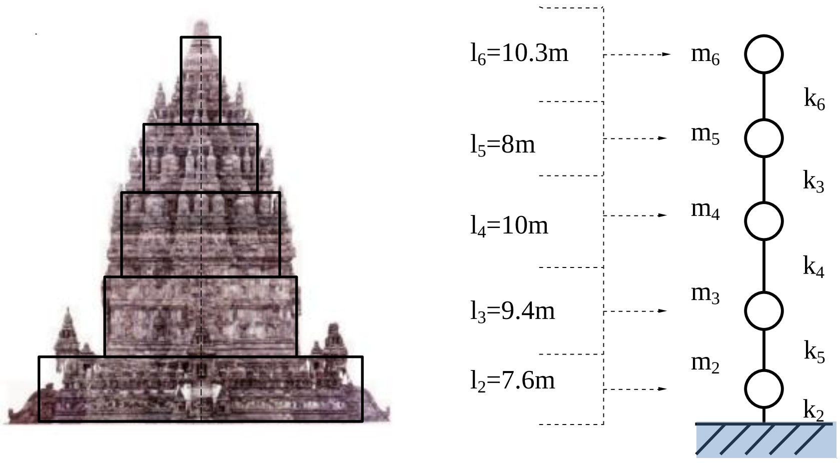
4.2. Analysis Model

In the present earthquake response analysis, it was employed the simplified model considering soil-structure interaction to simulate the seismic behaviours of the event on 12th September 2010. The structure of the Siva Temple was idealized by a stick model with 6 masses connected by beam elements. In addition, the 3 types of models were assumed as;

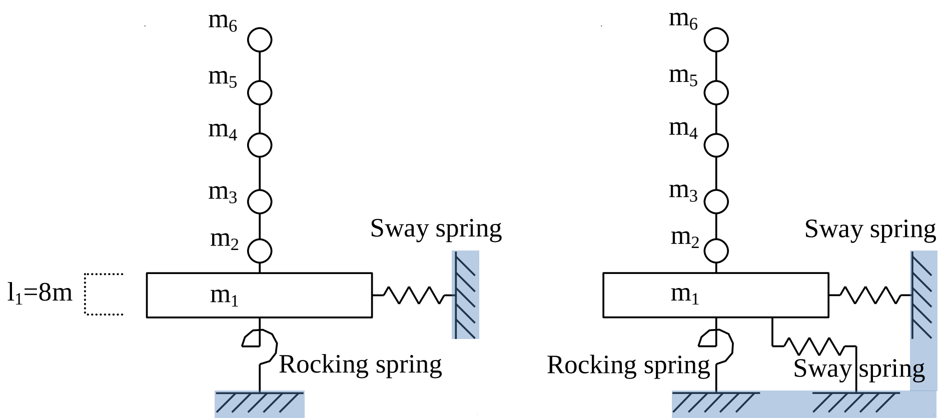
A) Rigid base model without dynamic soil-structure interaction
B) Sway-rocking (it is called SR) model, without base embedding effect
C) Embedding SR model, with base embedding effect,

Among the above models, dynamic soil-structure interaction was taken into account in both B)SR model and C)embedding SR model.

Not only eigenvalue analysis but also seismic response analysis was performed on the assumption that the structural material was concrete, as the inner structure was reconstructed by reinforced concrete technique. Such simplified model might be useful for the structure of which inner structural condition was not known. Here, it was ignored that the andesite stones were used for exterior walls. In this seismic response analysis, Rayleigh damping model for the whole system was assumed to be 4% (both for the primary and secondary modes). In addition, damping ratio of the soil spring was assumed to be 7%. Depth of embedding base was assumed to be 8m. The constant of the structural springs and the soil springs was evaluated, shown in Tables 4 and 5. In the present study, the soil spring stiffness was evaluated by employing the approximate solution of the vibration admittance theory [2]. The material properties of concrete (See Table 5) were evaluated from the experimental report of the laboratory material test of concrete samples, which was conducted in Gadjah Mada University [1].



A) Base fixed model



B) SR model

C) Embedding SR model

Figure 13 Three types of analysis models

Table 4 Element Specification of Each Mass

	Area(m^2)	Volume(m^3)	Equivalent Radius(m)	Second Moment of Area(m^4)
m_2	921	7000	17.1	6.75×10^4
m_3	408	3835	11.4	1.33×10^4
m_4	279	2790	9.4	6.20×10^3
m_5	144	1152	6.8	1.65×10^3
m_6	17	175	2.3	23

Table 5 Material Properties of Concrete and Soil

Concrete	ρ (kg/m^3)	2300
	ν	0.16
	E (N/m^2)	5.83E+09
Soil	ρ (kg/m^3)	240
	ν	280
	Vs (m/s)〈-8m～0m〉	1800
	Vs (m/s)〈under -8m〉	0.4

4.3. Results of Analysis and Discussion

Table 6 compares the natural frequency of the analysis models with that of the earthquake monitoring records. Fig. 14 also compares the analyses with the observation, where the peak acceleration of the seismic response is described. It should be noticed in Table 6 and Fig. 14 that the peak response acceleration of the embedding SR model is the closest to the actual record. Furthermore, the acceleration wave forms (Figs. 15 and 16) and the transfer function of the embedding SR model (Figs. 17 and 18) is well correlated with the record. Hence, for comparison of record and analysis, the observation points at Top (h=37.2m) and Middle (h=23.9m) correspond to the mass's number No.6 (h=40.2m) and No.4 (h=22.0m), respectively.

As results, Table 6 shows natural frequency of the analysis models in comparison with the observation. It can be noticed that the natural frequency of the base fixed model is rather higher than that of the SR models introduced to take into account soil-structure interaction. Furthermore, the natural frequency of SR model with embedment spring is in good agreement with the observation. Shown in Fig. 14, the maximum response acceleration of embedding SR model is in better agreement with that of the monitoring record, although the maximum response acceleration of the fixed base model at mass No.6 is much larger than the record. As results of the present study, it was made clear that dynamic soil-structure interaction greatly affects the seismic response of such massive and rigid structure on the soils. The acceleration response wave forms of the analysis and the monitoring (Figs. 15 and 16) show that the wave forms are in good agreement. Shown in Fig. 17, it can be recognized that every peak value of transfer function from the ground to the top is at 2.0Hz. Furthermore, it can be confirmed in Fig. 18 that every peak value of transfer function from the top to the base is at 3.0Hz. It demonstrated that the natural frequency of soil-structure system was lower than that of structure itself. These made it clear that soil-structure interaction greatly affects the seismic performance of this monument.

Table 6 Comparison of natural frequency

Analysis Model	Natural Frequency(Hz)
A) Base Fixed Model	4.6
B) SR Model	1.6
C) Embedding SR Model	2.0
Monitoring	2.0

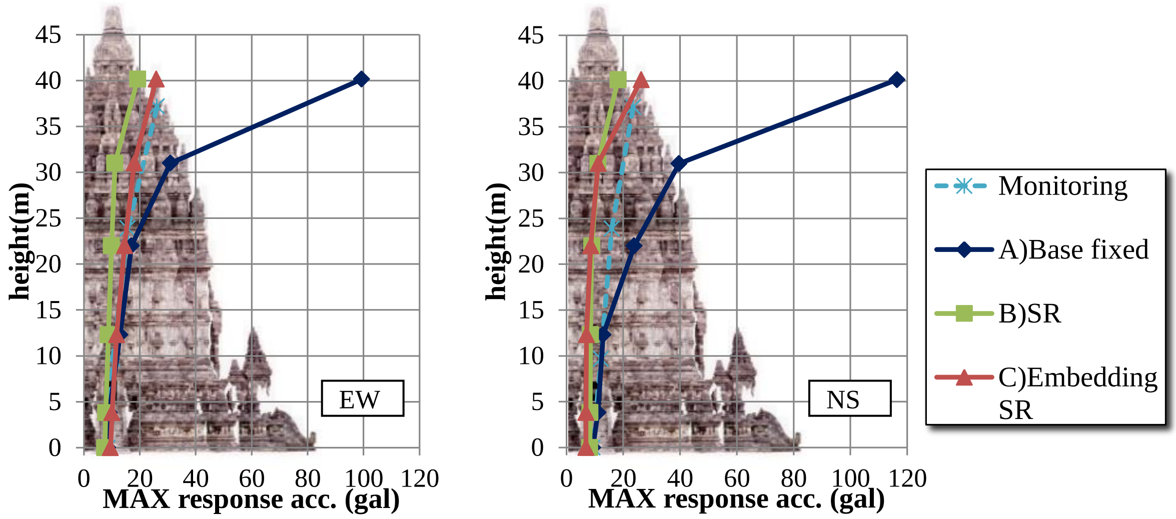


Figure 14 Comparison of peak response acc.

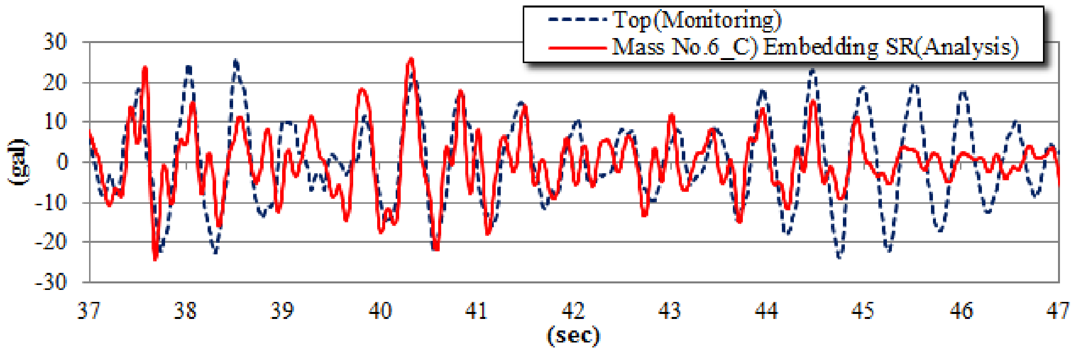


Figure 15 Comparison of acc. wave form (EW)

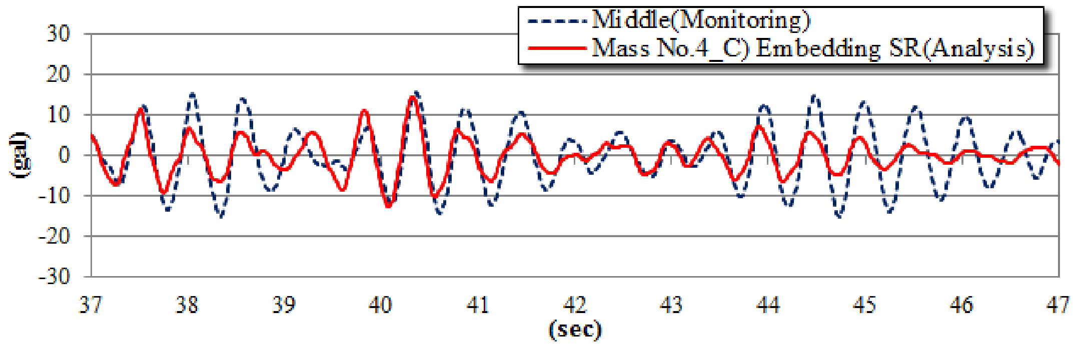


Figure 16 Comparison of acc. wave form (EW)

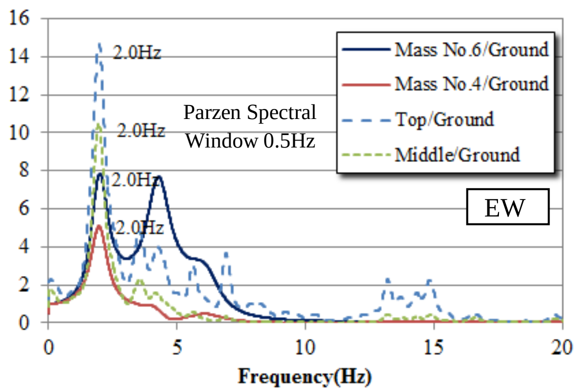


Figure 17 Comparison of transfer function to Ground

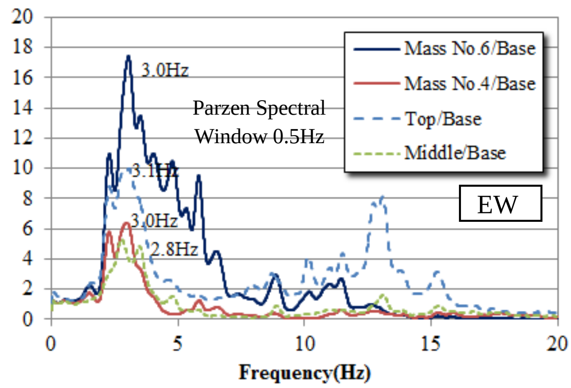


Figure 18 Comparison of transfer function to Base

5. CONCLUSIONS

The earthquake data of three events has been successfully recorded since the installation of the equipment at the Siva Temple. As the magnitude of the recorded three earthquakes was as small as 4.2 through 5.1, the ground motion level was less than 10cm/s^2 in PGA. However, those monitoring data revealed the fundamental dynamic characteristics of the earthquake response. Those date showed that the natural frequency observed during the earthquakes corresponded to that of the micro tremor measurement, because the ground motion level of those earthquakes was not high enough to cause non-linearity of both the structure and the soil-structure interaction. Utilizing the earthquake monitoring data, simulation analysis was successfully performed. The simulation analysis using the simplified model made it clear that soil-structure interaction greatly affected the seismic performance of the Siva Temple, indicating that the dynamic performance of such massive structure is rather affected by the soil-structure interaction. To assess the seismic safety, the simplified lumped masses model was useful for such heritage structures of which inner structural condition was unknown.

The monitoring records showed that the crack displacement was affected by temperature. However, the long term monitoring of crack displacement for about 4 years at the Siva Temple indicated that the structure has been stable, as there found no significant crack movement even when the earthquake occurred in September 2010, November 2011 and March 2012.

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