SEISMIC RESPONSE ANALYSIS OF NANJING YANGTSE RIVER BRIDGE

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SUMMARY

In this paper, dynamic behaviour studies of the Nanjing Yangtse River Bridge were carried out using the observational data, obtained from the earthquake excitation and the dynamic prototype tests of the bridge. The rusults of numerical analysis indicate that each pier moves individually under the earthquake excitation. Therefore, in the aseismic design of bridge structure the space action of the whole bridge need not be considered. The maximum acceleration at the bottom of bridge tower is 32% less than that at the nearby free ground surface. Finally, a method has been proposed for evaluating earthquake resistant behaviour of structure.

INTRODUCTION

The assumption that the piers of beam bridge vibrate together or individually under earthquake excitation is a decisive factor in determining correctly the earthquake load on piers. Although various mathematical models for computing the natural frequencies of bridge were proposed in consideration of these features, research workers hold still different points of view (Ref. 1, Ref. 2, Ref. 3). For better understanding of seismic response of beam bridges, a lot of dynamic prototype tests have been carried out in the seismological Bureau of Jiangsu Province. A set of strong motion acclerographs were located on the Nanjing bridge, and some strong motion seismograms have been acquired.

Based on these data, earthquake resistant behaviour of bridge was investigated.

INSTRUMENTATION AND STRONG MOTION STATIONS

Strong motion accelerograph

The main type of instruments used is the accelerograph, Type RDZ-1-12-66, which is a self-triggered, electro-magnetic type with 12 channels, wide frequency range of linearity (0.5-35 Hz), adjustable sensitivity (0.5-1.0 gal/mm) and 3 speeds of recording paper (2.1, 4.8, 11.3 cm/sec). Such intrument is especially suitable for measurement of ground motion with rather high frequency content and structural vibration where simultaneous

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recordings at a number of points are required (Ref. 4).

Vibration generator

The vibration generator used has a wide range of operating frequencies with high accuracy and high stability. It operates within frequency range from 0.5-30 Hz, in step of 0.001 Hz; The maximum force obtained 6000 Kg, at frequencies range 4.2-30 Hz; accuracy of the phase of each generator can be controlled to within 10° below 30 Hz during synchronized operation. The large excitation instrument is very useful for measurement of natural frequencies for the stiff and heavy structures such as bridge.

Strong motion stations

The positions of 10 strong motion stations are shown in Fig. 1. They are placed on the major bridge piers Nos. 1 to 4 and at various levels of bridge tower while one station (Sta. 1) is located about 50 m from the south bridge tower on free ground surface.

RESULTS OF ANALYSIS

An earthquake of magnitude 5.5 occurred on April 22, 1974, in Liyang County, Jiangsu Province, China, with epicentral intensity Vll. These stations on bridge were about 85 kilometers away from the epicenter. A set of strong motion accelerograms were acquired during the Liyang earthquake. After digitization and base-line adjustment of the records, maximum accelerations and durations of shaking were determined, Fourier and response spectra were computed. The dynamic characteristics of bridge structure were also obtained from the dynamic prototype test, using the vibration generator, train-induced vibration or ambient vibrations. The measured and computed results are shown in Fig. 2, Fig. 3, and given in Table 1, 2.

Seismic response characteristics of structure

By comparing the periods of peak value of Fourier spectra of piers shown in Fig. 2 to measured natural periods of piers, some characteristics can be found out. Natural periods of each pier are different during one earthquake excitation with max. peak horizontal acceleration of nearly 23 gal. at free ground surface, but the natural periods of piers of different height remain unchanged under random or train-induced vibration. There appeared a clear trend of a change of the behaviour, from spatial to separate, at levels of excitation by using vibration generator as a shaking source. It is very interesting to note that, for small vibration amplitude of the bridge by weak excitations such as random or train-induced excitations, the bridge reveals strong spatial vibration and measured natural periods are almost the same for piers Nos. 1-4. The spatial vibration decreases with increasing level of vibration up to max. peak horizontal acceleration 23 gal. at ground level. At the moment space action disappear, each pier moves individully under earthquake excitation. Therefore, the space action need not be considered in the aseismic design of bridge structure.

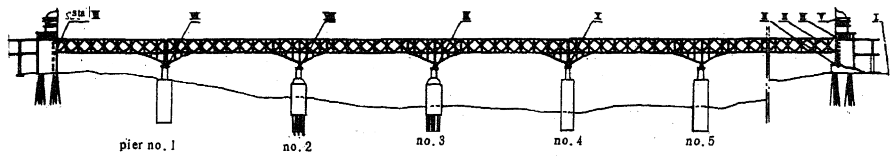

Fig.1. Locatin of strong-motion seismograph stations

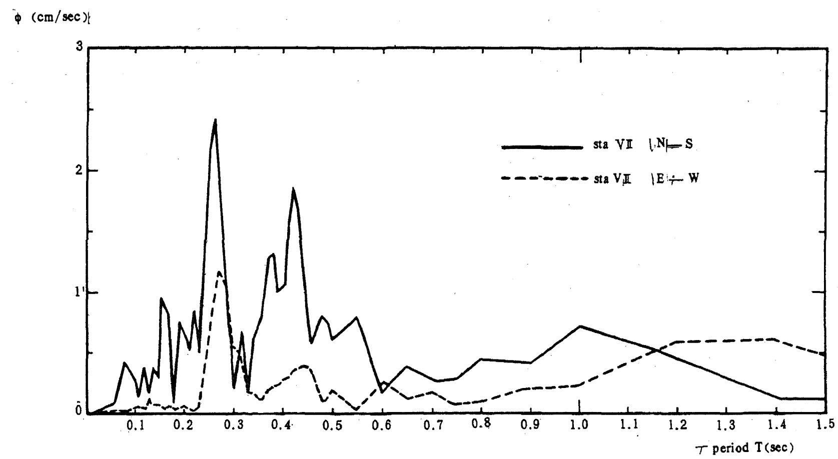

Fig.2.(a) Fourier acceleration spectra of the pier no. 1

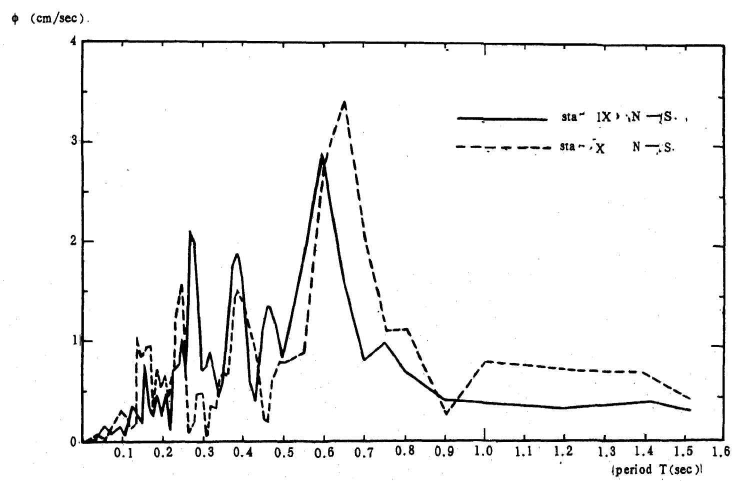

Fig.2.(b) Fourier spectra of the piers no. 3 and no. 4

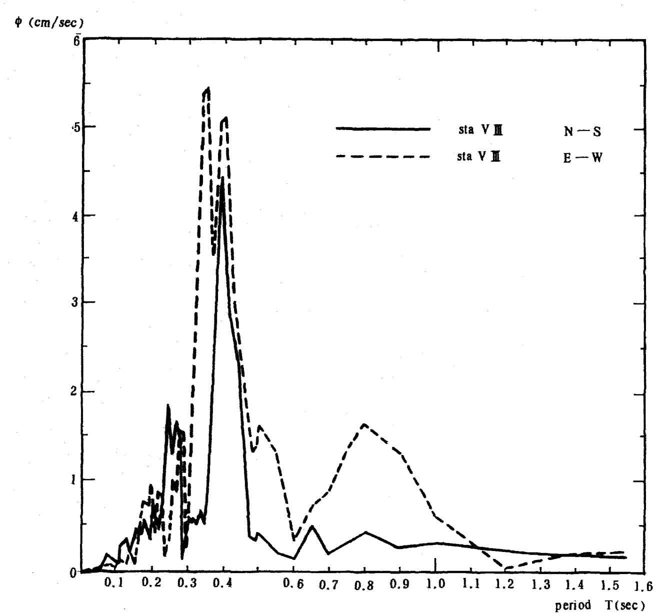

Fig.2.(c) Fourier acceleration spectra of the pier no. 2

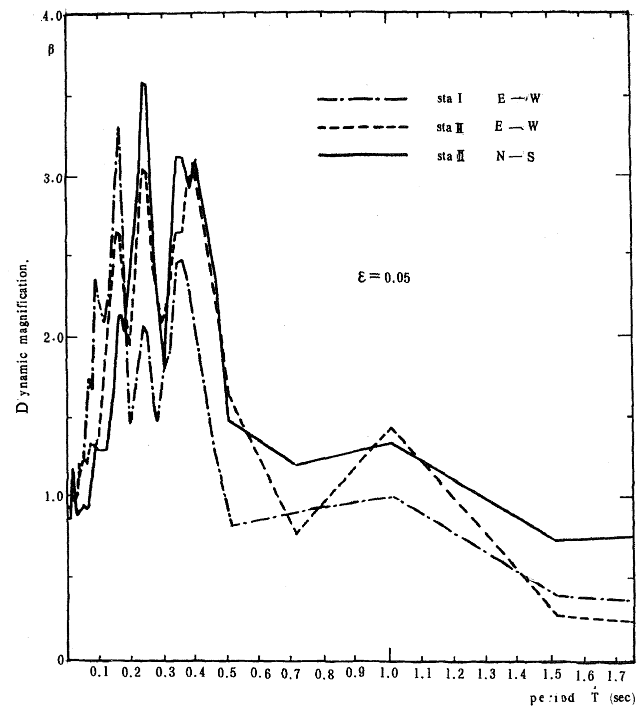

Fig.3. Horizontal acceleration response spectra

Measured Max. accelerations from earthquake Table 1

Position	Level	Station		Max. acceleration (gal)	Arriving time of Max. acceleration (sec)	Relative value
Free ground surface	2.00	I	N-S			
			E-W	22.92	3.88	1.47
South tower	3.75	II	N-S	-9.30	7.29	1
			E-W	15.51	3.27	1
South tower	27.4	III	N-S	14.23	5.95	1.53
			E-W			
South tower	42.89	IV	N-S			
			E-W	16.18	9.6	1.04
South tower	60.7	V	N-S	64.53	3.11	6.93
			E-W	-29.93	2.84	1.93
North tower	27.4	VI	N-S	31.2	1.63	3.35
			E-W			
Pier No.1	8.8	VII	N-S	-22.44	1.29	2.42
			E-W	7.0	2.24	0.45
Pier No.2	8.13	VIII	N-S	-18.31	1.07	1.97
			E-W	-38.22	1.05	2.46
Pier No.3	9.28	IX	N-S	13.21	2.83	1.42
			E-W			
Pier No.4	10.73	X	N-S	18.07	2.63	1.94
			E-W			

Remark: N-S is denoted longitudinal direction, E-W denoted transverse direction.

The relative Max. acceleration values at different positions of the bridge are shown in Table 1. The values of all the stations are more than 1, except Sta. VII, and increase with increasing height of bridge tower, the ratios at the top of each pier are nearly 2. The arriving times of Max.

Natural periods of tower, piers and predominate periods of free ground surface   Unit Sec.   Table 2

Structure position	South tower N-S	South tower E-W	Pier No.1 N-S	Pier No.1 E-W	Pier No.2 N-S	Pier No.2 E-W	Pier No.3 N-S	Pier No.3 E-W	Pier No.4 N-S	Pier No.4 E-W	Free ground surface N-S	Free ground surface E-W
Random or train-induced vibration	0.5	0.5	0.63	0.60	0.61	0.65	0.61	0.65	0.63	0.65	0.39	0.32
Vibration generator					0.51		0.64	0.52	0.69	0.54		
Actual earthquake	0.41	0.41	0.26	0.27	0.39	0.35	0.60		0.65			0.36

peak acceleration are not same at all the stations.

<u>Effect of the bridge tower mass on ground motion</u>

The existance of the bridge tower mass affects the characteristics of ground motion in the neighborhood. Based on the data obtained in actual earthquake, it can be seen that the Max. acceleration at the structure foundation is obviously less than that at nearby free ground surface (Ref.5, Ref.6, Ref.7). This phenomenon is also an evidence that the maximum acceleration at the bottom of huge structure, such as bridge tower, is always less than that at the nearby free field. It seems that this 32 percentage will increase with the increase of structure weight, but it is small in amount.

<u>Characteristics of ground motion</u>

The spectral shapes for a 5% damping are shown in Fig. 3. To compare the spectral shapes of the stations at the bottom of bridge tower and free field. There are quite similar. One predominant period of spectra of earthquake motion at the Sta. I is 0.36 sec close to predominant period 0.32 sec measured in ambient vibration for free field. There are main peak periods of spectra of earthquake motion at 0.36 and 0.4 sec at the bottom of bridge tower. The former approches the predominant period of the free field and the later approches the measured foundamental natural periods of the bridge tower. It indicates that the spectra of the bottom of structure not only reflected the characteristics of ground motion, but also dynamic behaviour of structure or soil-structure interacting system.

METHOD FOR EVALUATING EARTHQUAKE RESISTANT BEHAVIOUR OF STRUCTURE

The proposed method was applied to evaluate earthquake resistant behaviour of structure while earthquake records were acquired at structure. The method is as follows. At first, various possible cases of earthquake acceleration distribution at different times over all stations were studied and the cases giving Max. stresses of structural sections were then chosen from these possible cases, the stresses at various sections of the structure for a given earthquake intensity were finally proportional to the free-field acceleration corresponding to that intensity. For example, in evaluating earthquake resistant behaviour of any one of piers No. 1 to No. 4, the free-field record obtained near the bridge tower was used for all pier bases and the following four cases were considered.

Case I. Max. absolute accelerations both at top and at base of the pier.

Case II. Acceleration distribution at time of occurrence of Max. absolute acceleration at base of the pier.

Case III. Acceleration distribution at the time of occurrence of Max. absolute acceleration at top of the pier.

Case IV. Acceleration distribution at times when accelerations both at top and at base of the pier are in same direction.

It is very convenience to evaluate the earthquake resistant behaviour of piers in this way. The results computed for intensity VII are shown in Table 3.

Earthquake resistant calculation of piers Table 3

Piers	Position	Intensity VII K_H=0.1 section eccentricity e_o(m)		stress (Kg/cm^2)		Permissible value section eccentricity e_o(m)		stress (Kg/cm^2)	
		N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
No.1	bottom	0.396	0.195	-4.32	-8.17	2.24	5.2	17.2	17.2
No.2	bottom	2.08	1.98	13.1	-5.67	2.24	5.2	17.2	17.2
		(0.214)	(0.36)	(-7.26)	(-10.4)				
	foundation	2.48	3.04			4.05	6.25		
		(0.65)	(1.26)						
No.3	bottom	0.13		7.99		2.24	5.2	17.2	17.2
	foundation	0.612				4.05	6.25		
No.4	bottom	0.282		-5.47		2.24	5.2	17.2	17.2
	foundation	0.805				4.58	5.6		

Remark: The values of parenthesis is denoted not considering the superstructure earthquake force transfered to the piers.

COUCLUSIONS

1. In the aseismic design of bridge structure, the space action need not be considered.

2. The Max. acceleration at structure foundation is less than that at nearby free field. This decrease will be greater with the increasing structure weight, but it is small in amount.

3. By using the method proposed, earthquake resistant behaviour of structure at which the strong motion seismograms have been acquired can be evaluted with satisfaction.

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