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ANALYSIS OF FIXED REINFORCED CONCRETE(RC)ARCH BRIDGE WITH SMALL SECTION MEMBERS UNDER EARTHQUAKES

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ABSTRACT

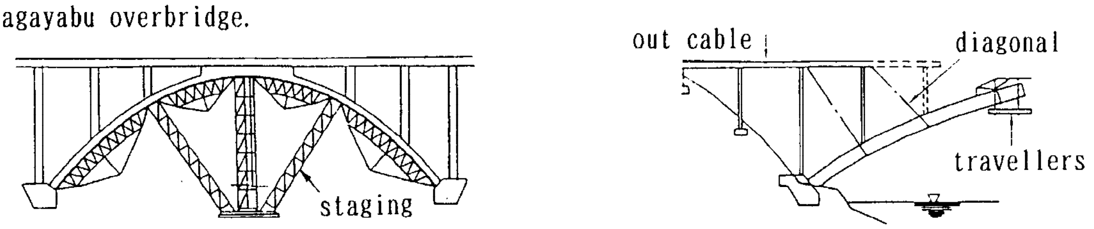
At present paper, the fixed reinforced concrete arch bridge with small section members, which is designed by the erection with staging, is investigated. In the two cases of excitation from earthquake waves, one is shown in *Specifications for Highway Bridges-part V Seismic Design* and another was measured in Japan in January of 1995, the dynamic analysis is made by the response spectrum methed.

KEY WORD

fixed RC arch bridge; erection with staging; dynamic analysis

INTRODUCTION

For reinfored concrete (RC) arch bridges, erections can be roughly divided into two types, one that is staging, the other being a cantilever. An erection with all staging can be used for an arch bridge with small spans on account of the possibilities of making use of the space below the girder. On the erection by cantilever, form travellers are placed at the front end of arch ribs. In order to supporting these travellers by diagonal members and the arch ribs, relatively large forces will be acted on the arch ribs in the course of the construction. In contrast, it is demonstrated as one of beneficial features of erection with all staging that under construction the forces acting on the arch ribs are small such that the cross sections of the members can be decreased as shown in Fig. 1, with the rigibity ratio is about 1:0.5. At present paper, in order to investigate the behavior of the fixed reinforced concrete (RC) arch bridge constructed by staging under earthqake, the dynamic analysis is made by the response spectrum methed using of the Nagayabu overbridge.



(erection with staging) (erection of cantilever)

Fig. 1 The erection of RC arch bridges

CALCULATION

Model

In present research, the Nagayabu overbridge is taken a mechanical model, which is the fixed RC arch bridge with 140m long, 83m arch span, 24m arch rise, 6m width, and hollow slab stiffeners, spand rel-filled arch, two-columns-section vertical members, end post and piers. The thickness of arch rib members is determined in value 1.2m, which is small in relativing to 83m arch span. The whole is shown in Fig. 2. In the calculation, it is assumed that some hinges are set on the top of vertical members, end posts and piers, and at the base of vertical members except the ones placed near to end post.

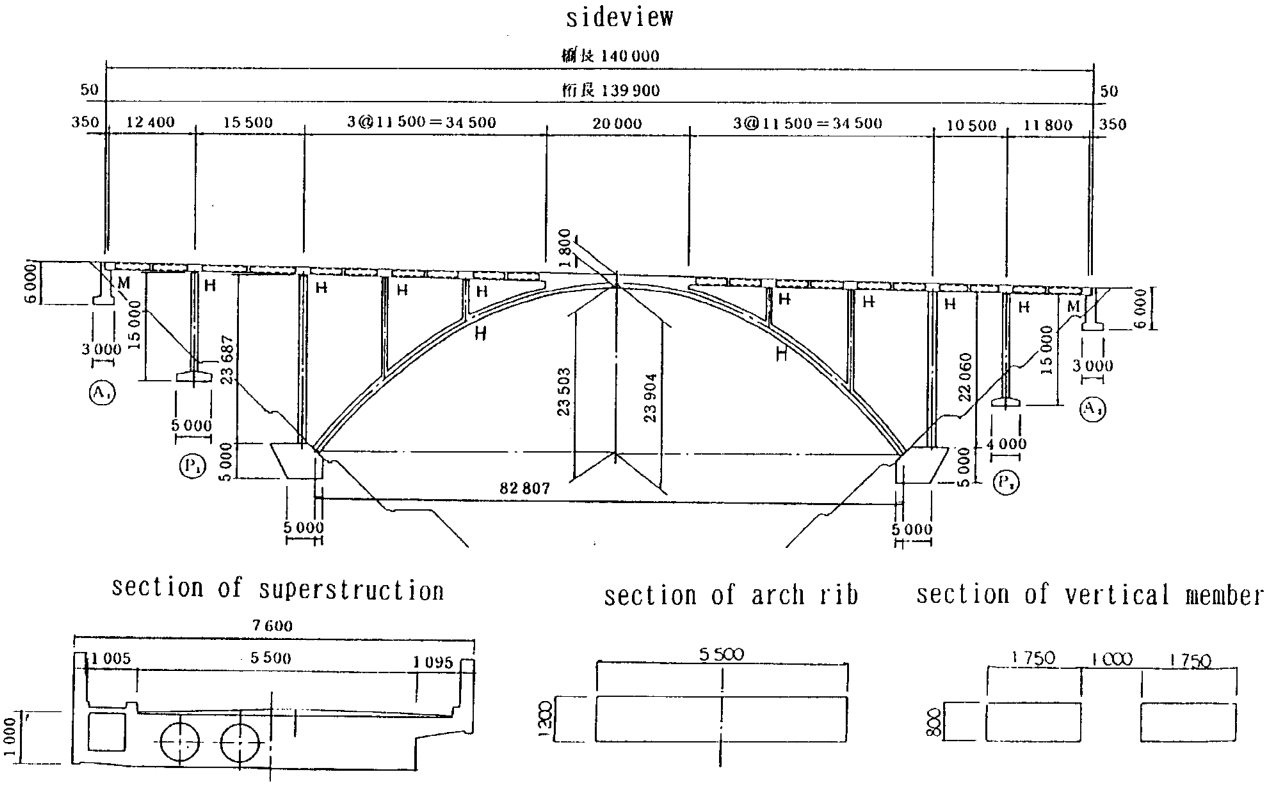


Fig. 2 Model of bridge

Accleration response spectrum

Based on *Specifications for Highway Bridges-Part V Seismic Design* , the acceleration response spectrum can be defined as:

$$S = C_Z \cdot C_I \cdot C_D \cdot S_O \qquad (1)$$

Where, S :accleration response spectrum used in the analysis made by the response spectrum method (unit:gal or $10^{-2}m/sec^2$)

C_Z :factor of region=0.85 (B)

C_I :factor of degree of road=1.0 (the first degree)

C_D :damping factor=1.0 (where: $C_D = \dfrac{1.5}{40h_i + 1} + 0.5$, h_i: damping coefficient=0.05)

S_O :standerd acceleration response spectrum=values corresponding to the 1st type ground.

Here, S_O is defined in the two cases, that is:

Case 1 :the acceleration response spectrum specified in *Specifications for Highway Bridges-Part V Seismic Design* seeing Fig. 3.

Case 2 :the acceleration response spectrum measured under earthquake happened in the south of Hyogo Province in last year, seeing Fig. 4.

The data shown in Fig. 3 are used in the design of the Nagayabu Bridge, which suppositively summed up on the earthquake caused by continental drifting in Japan from time immemorial. The data shown in Fig. 4 are measured by the Marine Meteorological Observatory of Kobe on the 17th of January in 1995 under the great fault earthquakes. In conventional practice, these data measured can not used in design of constructions, because they are so great that causing a loss of economy, but can be used in the further analysis later on. As viewed from the fact of that arch bridges are generally complexer in contruction than the girder bridges with other types, and these natural frequencies responding to different modes are very closed, the enough numbers of order of mode should be think on in the calculation. For comparing, the dynamic analysis is made by the response spectrum method by used of the data of Case2 in linear thedry rather than nonlinear in spite of obviously intensive exitation. The acceleration values in horizontal direction responding to Case2 are tending to 2000gal when cycle values are below 1.10sec. However the values for Case1 are just 200gal under same condition. Ten times difference between two cases is indicated.

In Table. 1, numerical results of calculation are shown. The mode shapes of the modes with great excitation factors shown in Table. 1 are plotted in Fig. 5 and Fig. 6 respectively the vibration occurred in the construction plane and out the plane.

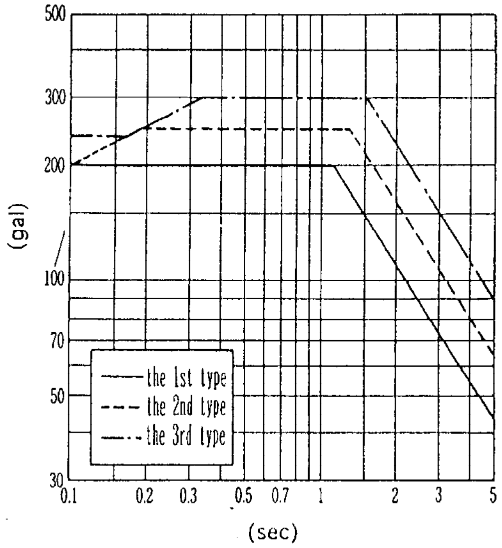


Fig. 3 The acceleration response spectrum specified in Specifications for Highway Bridges-PartV Seismic Design

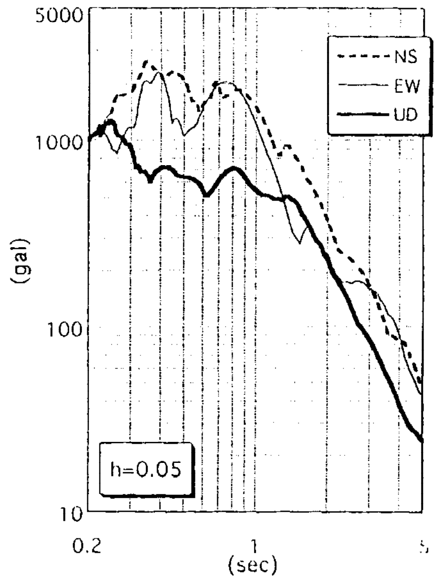
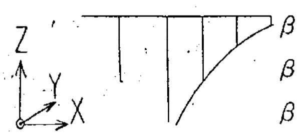


Fig. 4 The acceleration response spectrum measured by the Marine Meteorological Observatory of Kobe

Table. 1 The results of natural frequencies

mode NO.	normal modes (Hz)	main direction of vibration	exitation factor			acceleration response spectrum (10^{-2}m/sec^2)			
			βX	βY	βZ	CASE 1		CASE 2	
						lateral	vertical	lateral	vertical
1	0.933	in plane (1)	0.7620	0.0000	-0.0306	170	85	1275	425
2	1.438	out plane (1)	-0.0000	1.2860	0.0000	170	85	1700	510
3	2.243	in plane (2)	-0.0068	-0.0000	-0.0714	170	85	1700	595
4	3.028	out plane (2)	0.0000	0.1512	0.0000	170	85	1700	595
5	3.554	in plane (3)	0.2920	-0.0000	0.0175	170	85	1530	850
6	3.976	out plane (3)	-0.0000	0.8132	0.0000	170	85	1275	1105
7	4.531	in plane (4)	1.0210	0.0000	-0.0085	170	85	1020	935
8	5.162	in plane (5)	0.8939	-0.0000	0.2091	170	85	850	850
9	5.326	in plane (6)	-0.2161	-0.0000	0.1568	170	85	850	850
10	6.309	out plane (4)	0.0000	-0.0543	-0.0000	170	85	850	850
11	6.739	in plane (7)	1.1080	-0.0000	0.0265	170	85	850	850
12	6.985	out plane (5)	0.0000	0.5471	-0.0000	170	85	850	850
13	7.157	in plane (8)	-0.1023	-0.0000	-2.1090	170	85	850	850
14	7.534	in plane (9)	0.2192	0.0000	1.9760	170	85	850	850
15	8.004	out plane (6)	-0.0000	0.3371	0.0000	170	85	850	850
16	8.243	in plane (10)	0.3079	0.0000	0.2839	170	85	850	850
17	8.852	in plane (11)	0.6276	0.0000	-0.7480	170	85	850	850
18	9.080	out plane (7)	-0.0000	0.5433	-0.0000	170	85	850	850
19	9.734	in plane (12)	-0.2194	0.0000	-0.0420	170	85	850	850
20	9.830	in plane (13)	1.0040	0.0000	-0.0985	170	85	850	850



βX : The exitation factor in the longitudinal direction

βY : The exitation factor in the direction of perpendicalar to longitudinal

βZ : The exitation factor in the vertical direction

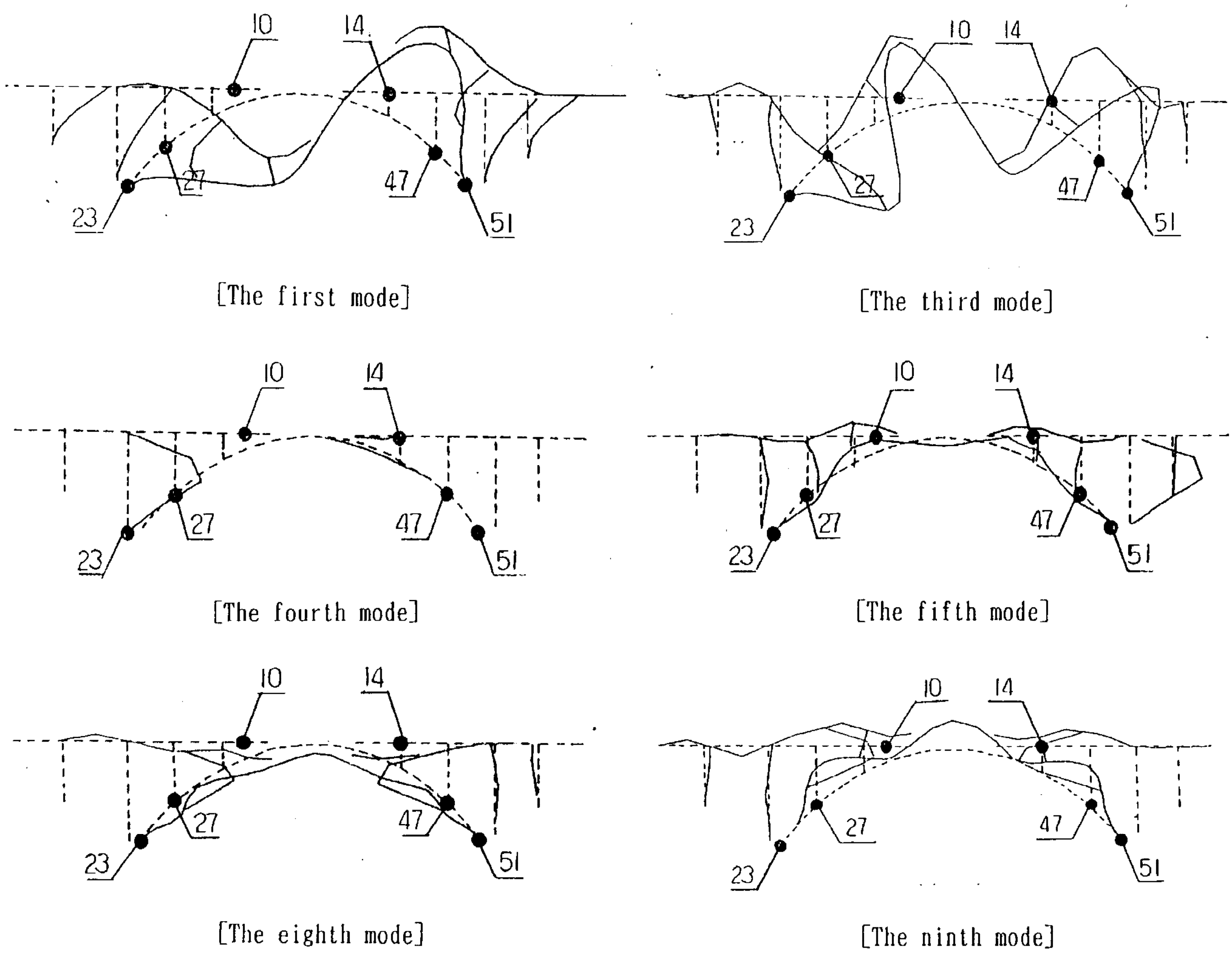


Fig.5 The shape of the modes in plane

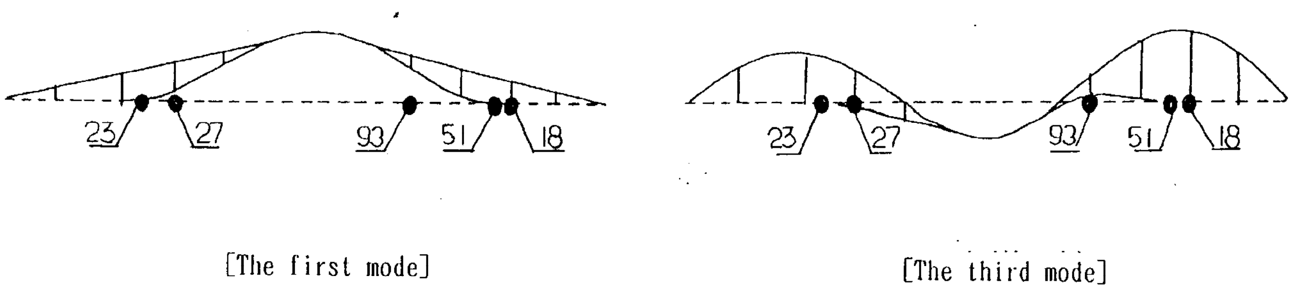


Fig.6 The shape of the modes out plane

Numerical results of Casel

For the Casel, the extreme responses and stresses are shown in Table. 2. For the shape of mode in plane, the maximum values occurred respeclively at the base of the arch ribs, close to the arch crown of the stiffening girders and the base of vertical members. Although the ones of the third mode in plane on arch ribs and stiffening girders are relatively great, they are fall below the allowed stresses, thus, safety. And, for vertical members, the maximums are responding the ninth mode in plane. For modes out plane, the maximal values of arch ribs and vertical members occurred at the places are same as the mode in plane, but the stiffening girders at the place near to end post. Despite the components of force responding to the modes out plane have great values than in plane, no notable stresses occur because of large stiffness.

Table. 2 The maximum components of forces and stresses for variour parts in casel

direction		in plane			out plane		
part		arch rib	stiffening girder	vertical member	arch rib	stiffening girder	vertical member
node		23	10	47	51	18	93
force response	M (10^3Nm) N (10^3N) S (10^3N)	12700 863 1180	3200 618 147	2540 618 196	35500 0 1770	4560 0 88	4520 0 941
force in static analysis	M (10^3Nm) N (10^3N) S (10^3N)	1610 20600 39	2030 59 88	510 2580 39	0 20800 0	0 88 0	0 2400 0
resultant of forces	M (10^3Nm) N (10^3N) S (10^3N)	14300 21500 1220	5230 677 235	3050 3200 235	35500 20800 1770	4560 88 88	4520 2400 941
stress	σc (N/mm^2) σs (N/mm^2) τ (N/mm^2)	7.5 72.0 0.14	6.1 204.9 0.10	12.4* 329.7* 0.10	4.2 2.9 0.18	1.9 124.7 0.02	2.7 79.0 0.18
allowance	σca (N/mm^2) σsa (N/mm^2) τa (N/mm^2)	11.8 294.2 0.58	11.8 294.2 0.58	11.8 294.2 0.58	11.8 294.2 0.58	11.8 294.2 0.58	11.8 294.2 0.58

Numerical results of Case2

In case2, the extreme responses of the different parts of the bridge are shown in Table. 3. For the shape of modes in plane and out plane, the places of the maximum components of force occurred are similar to the Case1, but the order of mode is first and third not only in plane but alse out plane. It is found that obvious vibrated shape appears for lower order of mode. This appearance can be explained by the quite large values of accelaration in response spectrums within the range of that the cycle is 0.3~1.0 sec.

For comparing, the components of force of two cases on the location at which maximum occur for the Case1 are shown in Table. 4. It is obvious that the all of components are much greater in Case2. And focusing attention on the moments which play an important part in the calculation of stresses, the values of arch ribs and stiffening girders for modes not only in plane but also out plane are approximately 10times greater than the Case1, and the vertical members are 1~3times. Consequently, it is understand that the responses of forces on arch ribs and stiffening girders are greater than the vertical members.

Table. 3 The maximum components of force for various parts in case2

directionn		in plane			out plane		
part		arch rib	stiffening girder	vertical member	arch rib	stiffening girder	vertical member
node		51	14	27	23	18	27
force response	M (10^3Nm)	137000	42800	10400	599000	45500	41400
	N (10^3N)	7890	4510	4590	0	0	0
	S (10^3N)	12700	1030	500	21100	2060	3540

Table. 4 The comparison of results between the two cases

directionn		in plane			out plane		
part		arch rib	stiffening girder	vertical member	arch rib	stiffening girder	vertical member
node		23	10	47	51	18	93
CASE 1 force response	M (10^3Nm)	12700	3200	2540	35500	4560	4520
	N (10^3N)	863	618	618	0	0	0
	S (10^3N)	1180	147	196	1770	88	941
CASE 2 force response	M (10^3Nm)	132000	38200	8830	585000	45500	3190
	N (10^3N)	7300	7020	7040	0	0	0
	S (10^3N)	11300	1760	441	21000	2060	2080

CONCLUTION

Based on the above, the results as follows can be obtained for the fixed reinforced concrete arch bridge with small section member.

1)By the acceleration response spectrum defined in *Shecifications for Highway Bridges-Part V Seismic Design* , on the modes vibrated in plane, the places of maximum force occur respectively at the base of the arch ribs, close to the arch crown of the stiffening girders and the base of the vertical members. For the arch ribs and stiffening girders, the extreme values are mainly dependent on the modes of the lower order, and the responding stresses are below the allowed stresses. By contrast, the ones of vertical members are dependent of the higher order. On the other hand, under the modes out plane, the places of maximum values happened for the arch ribs and the vertical members are same as the ones in plane but the stiffening girders are near by end post. In the case of the mode vibrated out of construction plane, as a result of large stiffness constants terrific stresses have not happened.

2)By the acceleration response spectrum measured by the Marine Meteorological Observatory of Kobe, the locations on which extreme response occur are same as the case of the standerd acceleration response spectrum, but the values are much greater than that. And, the maximum forces of various parts are all dependent on the modes of high order, then the top valves of the arch ribs and the stiffening girders are greater than the vertical members.

Based on the fact that the data of the acceleration response stectrum defined in *Shecifications for Highway Bridges-Part V Seismic Design* are specified at a distanse 100km from epicentre, it is reasonable that the dynamic analysis is made in the design of bridge used these data. Then, in view of the results that some abviously great components of force occur on the vertical members dependent on the modes of high order, it is necessary that not only static but also dynamic analysis are made in the design.

On other hand, it should be mentioned that the data of the Marine Meteorological Observatory of Kobe,, which is a fault one, are measured at a distance 20km from the epicentre. For this case it is found that very quite greater components of forces happen on the arch ribs and the stiffening girders than the vertical members, which values are much greater than the ones obtained by the standerd acceleration response spectrum, then, the variations of mechanic behaviors are dependant on the type of earthquakes. According to the fact that the arch ribs play an important part in the stability in the whole construction, it should be noticed that the enough stiffness is necessary to the some members of arch ribs under the similar fault earthquake. Then because the mechanical behaviors of arch bridges are sensitive to the modes vibrated not only in the construction plane but olso out of the plane, in the design of the similar arch bridges with small section members it should been pay attention to the regions located on active fault. And, nonlinear analysis is necessary to check for safety under this kinds of intensive exitation.