SEISMIC STRENGTHENING OF STEEL ARCH BRIDGE USING VISCOUS DAMPERS

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1. INTRODUCTION

The arch bridges in Japan were conventionally analyzed as its behavior under seismic loads is always in elastic range due to its construction in mountainous areas. However after 1995 Hyogo-ken Nanbu earthquake the updated specifications for highway bridges stipulated to reveal the inelastic behavior of all types of bridges. Plenty of researches represent three dimensional nonlinear dynamic response analyses are sufficient to understand dynamic response of bridges. In this study the steel arch bridge is investigated by means of the response analysis under seismic loads Level 2 Type I (plate boundary type earthquake with a large magnitude- Kanto earthquake) and Type II (an inland direct strike type earthquake- Hyogo-ken Nanbu earthquake). Also to increase energy absorption of bridge, the critical sections of bridge are determined and viscous dampers are supplemented to the system. Consequently the dynamic response analysis of the system with dampers is carried out.

2. OVERVIEW OF BRIDGE

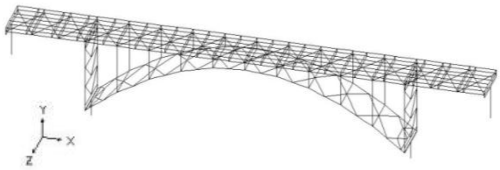


Fig 1 Three dimensional FE model

The arch bridge studied in this paper is an upper-deck type steel arch bridge with reinforced concrete deck slab. The total length of the deck and available width are 90.0m and 8.1m, respectively. RC deck slabs are supported by two main longitudinal girders attached transverse girders and diagonal members. Also the twin steel arch ribs consist of transverse and diagonal arch ribs have a span of 60.0m. The connection between arch ribs and main longitudinal girders are supported by 11 piers at the intersection joints between the main ribs and transverse bracings. Differently from others, piers at each end are strengthened by diagonal and transverse bracings. Both ends of the deck are supported by two end abutments. The three dimensional finite element model of the bridge is shown in **Fig 1**.

Table 1 Boundary conditions

	T_x	T_y	T_z	R_x	R_y	R_z
Abutment	F	R	R	R	F	F
Pier	R	R	R	R	F	F
Foundation	S	S	S	S	S	S

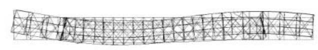
The deck slab, abutments and piers are modeled using linear beam element. Pile foundations are defined as a linear spring element and finally other steel elements are modeled as nonlinear fiber element. Boundary conditions are shown in **Table 1** (F: Free, R: Restraint, S: Spring).

3. DYNAMIC ANALYSIS OF BRIDGE

3.1. Eigen Value Analysis

Table 4 points out the frequencies, natural periods, effective masses and effective mass ratios of arch bridge for predominant eigenmodes. As seen from the table predominant eigenmodes of the bridge in the longitudinal direction (T_x) are the first, forth and thirteenth modes, in the transverse direction (T_z) are second and twenty-fifth modes.

Table 2 Predominant eigenmodes

Mode	Frequency (Hz)	Natural Period (sec)	Effective Mass (N)		Effective Mass Ratio (%)	Mode Shapes
			Tx	Tz		
1	1.521	0.657	439.5	0	15	
2	1.847	0.541	0	986.5	33	
4	2.645	0.378	747.2	0	24	
13	4.768	0.210	563.9	0	18	
25	6.620	0.151	0	707.9	23	

3.2. 3D Nonlinear Dynamic Response Analysis

First, the dynamic response analysis of the system is investigated without damper. The estimation of the maximum relative displacement at the both end of the abutments is essential to figure out the behavior of bridge under seismic loads. Thus, the analysis results of the maximum displacement at these points under seismic waves of Level 2 Type II described in Specifications for Highway Bridges Part V by Japan Road Association (Level 2-Type II, Ground Type I) is shown **Table 2**. The differential equations in finite element analysis are solved by Newmark-β method. The time interval of the integration 1/500sec and the damping coefficient for steel elements is set to 0.02. Rayleigh damping is used. Since two modes used in Rayleigh damping are dominant vibrations specified according to effective masses, for longitudinal axis the forth and thirteenth modes are used. From **Table 2**, as relative displacement is larger in Type II, investigation of capacity of damper was conducted by using Type II. Capacity of damper was estimated from the results of dynamic response analysis using single degree of freedom system. Based on target displacement, seismic coefficient which gives an indication of damper was determined. Capacity of damper was calculated from seismic coefficient and total weight of the bridge. Target displacements were varied from 0.02m to 0.10m.

Table 3 Relative displacements at the end of abutments

Location	Type I-1-1	Type II-1-1
	Relative Displacement (cm)	Relative Displacement (cm)
Left	5.08	14.70
Right	4.59	13.63

The attachments of four viscous dampers are located at the both abutments edge points as seen in the **Fig 2**. In total nine damper types (four dampers for Type I; five dampers for Type II earthquakes) are tested by means of maximum displacement and maximum acceleration at the centre section of steel deck. After comparison of them, damper type 3 is found out adequate for the bridge exposed to earthquakes Type I and Type II. Time history responses of abutment and center section of the bridge are given with the following figures. **Fig 3** and **Fig 4** are the system without damper, **Fig 5** and **Fig 6** are the system attached viscous dampers.

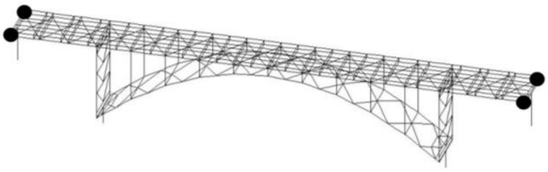


Fig 2 Locations of dampers

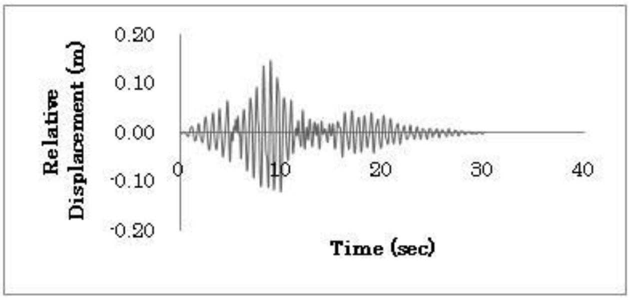


Fig 3 Displacements at the end of abutments

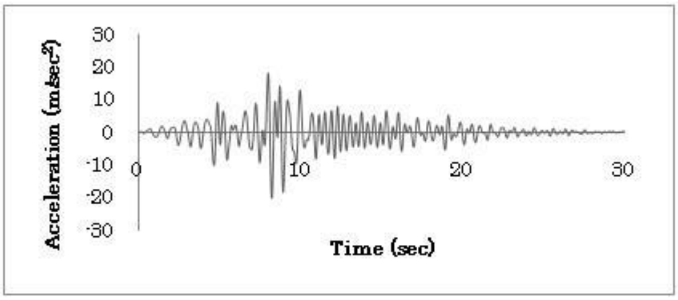


Fig 4 Accelerations at the center section of the bridge

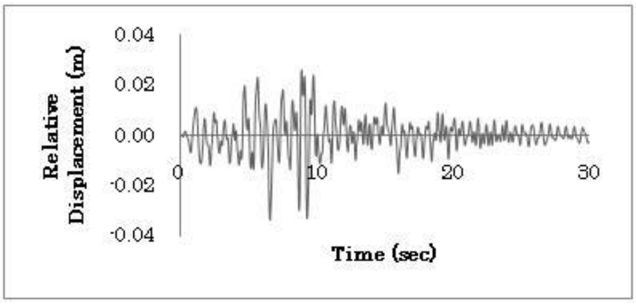


Fig 5 Displacements at the end of abutments

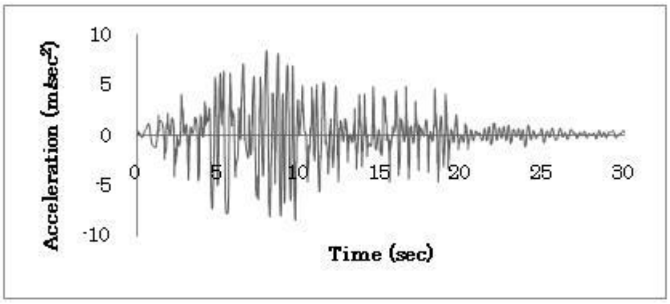


Fig 6 Accelerations at the center section of the bridge

4. CONCLUSIONS

Seismic strengthening of a steel arch bridge was conducted using viscous damper with velocity dependency. Applicability of evaluation of damper capacity using single degree of freedom system was confirmed.

REFERENCES

- Japan Road Association, Specifications For Highway Bridges, Part V Seismic Design, 2002.