eJSE International

EJSE Special Issue:
Selected Key Note papers from MDCMS 1 1st International Conference on Modern Design, Construction and Maintenance of Structures - Hanoi, Vietnam, December 2007

Research on seismic safety, retrofit, and design of a steel cable-stayed bridge

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ABSTRACT: After the 1995 Great Hanshin Earthquake, Japanese design specification of highway bridges was revised and many piers and bearing supports have been retrofitted. Large span bridges such as arch, truss, and cable-stayed bridges located in bay area of Kobe city also suffered damages from this earthquake. Therefore the investigation of seismic performance of long span bridges has been conducted for large earthquake. The Aratsu bridge constructed in 1987 is a steel cable-stayed bridge of 345m length, continuous three spans, and single plane of stay cables. The seismic performance of this bridge was examined by nonlinear time history analyses using tri-linear back bone and hysterecis loops. Furthermore fiber model analysis and FE analysis are successfully used to check the behaviors of this bridge. According to the analysis of the target bridge, weak members for large earthquake have been clarified. Install of damper and displacement restrainer, and strengthening of components of bearing supports are proposed retrofit countermeasures. The behavior of retrofitted bridge satisfies the criteria of design specification of highway bridges.

Keywords: seismic retrofit, cable-stayed bridge, nonlinear dynamic analysis, damper, performance design

1 INTRODUCTION

The 1995 Great Hanshin Earthquake attacked Kobe city and surrounding area, and caused death of more than 6000 people and many damages of houses, infrastructures such as lifeline systems, and urban highway bridges. After that Japanese specification of highway bridges design was revised and many bridges have been retrofitted according to the new specification. Over-viewing steel bridges damaged by this earthquake, a large number of failures of bearing supports and surrounding members were observed. Fig.1 shows the example in which pin type bearing support penetrated the bottom flange of the box girder. Set bolts of this support were broken by tensile axial forces as shown in Fig.2. Failure of set bolts in upper part of bearing caused the collision of girder and support. In other example shown in Fig.3 we observed the failure of set bolts, fracture of upper part of bearing and scattering of pin roller. Fig.4 shows relative displacement of girder in transverse direction (about 3.5m) which was also caused by the failure of bearing supports.

After the earthquake this highway bridge was not used any longer for a long time. In these damage examples failure of set bolts became the trigger for large damages of bearing supports, girders, and piers.

Furthermore large span bridges such as arch, truss, and cable-stayed bridges located in bay area of Kobe city suffered big damage from this earthquake. Therefore the investigation of seismic performance of long span bridges should be conducted for large earthquakes.

The Aratsu bridge constructed in 1987 and located in Hakata bay area is a steel cable-stayed bridge of 500m length, three spans continuous, and single plane of stay cables. The seismic performance of this bridge was examined by nonlinear seismic response analyses using the large-scale waves recorded in Great Hanshin Earthquake and so on, and was proposed some rational retrofit countermeasures.

Fig. 1 Penetration of a bearing support into bottom flange.

Fig. 2 Failure configuration of a set bolt.

Fig. 3 Failure example of pivot roller support.

Fig. 4 Relative displacement of girder in transverse direction (about 3.5 m).

2 FAILURE PATTERN LEARNED FROM THE DAMAGES OF STEEL BRIDGES DUE TO THE 1995 GREAT HANSHIN EARTHQUAKE

The damage examples mentioned above were summarized as follows by classification into acceleration, displacement, and direction of acting force.

(1) Acceleration for vertical direction and uplift

a) failure of set bolts, b) failure of girder and upper part of bearing support due to collision between girder and support , c) scatter of roller due to failure of bearing support, d) damage of girder and pier due to the collision of these two members

(2) Acceleration in longitudinal direction and displacement of movable bearing supports at ends of main girder

a) failure of displacement restriction device of bearing support, b) scatter of roller of bearing support, c) collision of adjacent girder and failure of girder connection devices, d) failure of expansion joint

(3) Acceleration in transverse direction

a) failure of bearing support, b) displacement of girder to transverse direction , c) damage of girder, pier, and bearing support due to the collision of these members.

Summarizing the abovementioned damage and failure of members, first failure of weakest member (mostly it is bearing supports) cause the following failure of other members (girders and piers), because structural system is changed by first failure. Therefore we should check the stress conditions of bearing supports for seismic retrofit strategy.

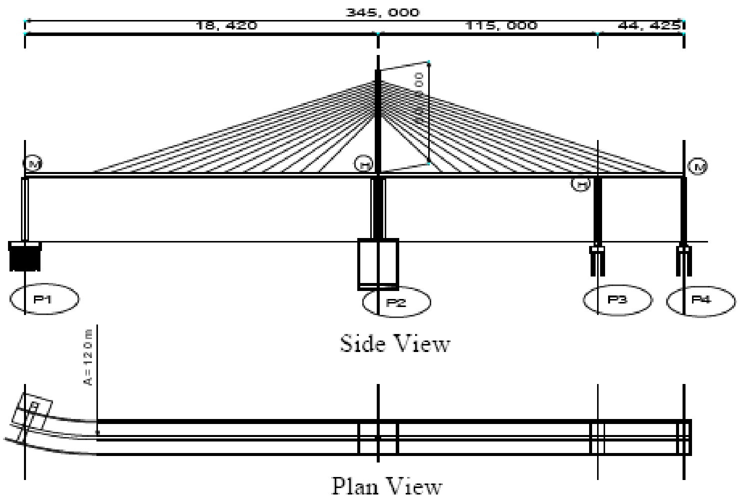

Fig. 5 Target cable-stayed bridge (named Aratsu bridge) (unit:mm).

3 TARGET BRIDGE FOR OUR SEISMIC RETROFIT PROJECT (FIG.5)

Cross section of the girder is tapered steel one box, and piers P1, P3 and P4 are frame type steel pier and P2 is hollow section reinforced concrete pier. The stay cable is multi fan type (13 stairs), and main tower is independent single tower. Connection of the

tower and girder is rigid type, and the main girder is supported by three pivot type bearing supports on the P2 pier. The main girder is supported by roller (movable) type bearing on P1 and P4. Restrainers for longitudinal horizontal displacement are settled on the top of these two piers. Shear type pin support are settled on P3 to resist upper force. P2 has already been retrofitted after the 1995 earthquake.
As the characteristics of this bridge two items are pointed out, which are as follows. The steel tower was constructed in vertical direction by so-called metal touch, i.e. tension forces can not be transmitted each other in this type of steel connection. The low-yield-point steel is used for over hang part (both sides of upper flange) of main girder, instead of high quality material for box part. Therefore the main girder is supposed to yield easily in transverse direction.

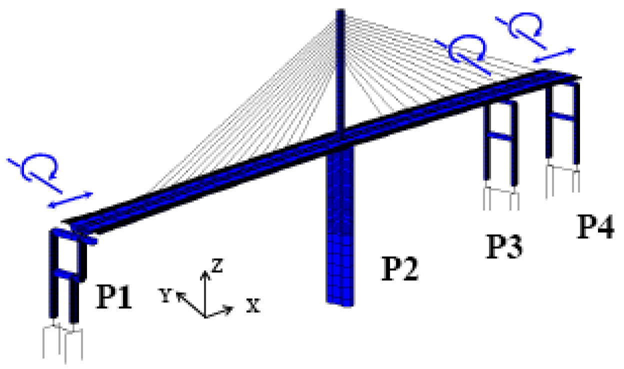

Fig. 6 Image of analytical model (Frame Structure).

4 PROBLEMS OF THIS BRIDGE CLARIFIED BY NONLINEAR DYNAMIC RESPONSE ANALYSIS

4.1 *Model for analysis*

Fig.6 shows analytical models. This bridge is important for network of urban highway, therefore after the large earthquake this bridge should exist as sound structure with slight damage. Some particular members are slightly allowed to enter the plastic region. For this reason all members of bridge except cables and soil spring are modeled as nonlinear structural members. Each stay cable is divided into eight members and degree of freedom of whole structure is 6000.

4.2 *Ground motion and damping ratio*

Ground motions used for our analysis are three acceleration waves recorded on ground surface at JR Takatori (Great Hanshin Earthquake), Itazimabashi (Hyuganada Earthquake), and Fukuoka city (Fukuokaken Seihouoki Earthquake). A region coefficient specified highway design specification is 0.7 for Fukuoka, so this value is multiplied former two waves, but is not used for third wave because the wave was recorded in Fukuoka. Rayleigh damping and strain energy proportion damping are used. Damping ratio of each member is 0.02

4.3 *Problem of this bridge clarified by dynamic analysis and design are as follows*

(1) Displacement restrainer for roller support on piers 1 and 4is failed by seismic force in longitudinal direction. According to this event relative displacement (700 mm) over allowable displacement of the support (110mm) occurs, and collision for adjacent girder and failure of bearing support happen.
(2) Set bolts become in plastic region due to vertical seismic force.
(3) Bearing support on pier 1 and 4 fails due to seismic force in transverse direction. This event introduces failure of girder support because of the horizontal forces acting on the support.
(4) In girder and tower supports at pier 2 some component of bearing support yield.
(5) Web plate in edge rib of steel deck of main girder buckle.
(6) Metal touch connection in the main tower does not sustain over turning moment in transverse direction.

5 RETROFIT POLICY AND COUNTERMEASURES

5.1 *Basic policy of seismic retrofit*

In our study following items are adopted as basic policy, and effective countermeasures satisfying these items are adopted, after that retrofitted structures are checked by nonlinear dynamic analysis, and ensuring that the problems pointed out for existing bridge is solved, finally countermeasures are decided.
(1) Retrofit cost is cheap.
(2) Traffic regulation is few during retrofit construction.
(3)Appearance is excellent after the retrofit.
(4) Minimizing the removal members.
(5) Retrofit is possible at the construction site.

5.2 *Adopted retrofit countermeasures*

Main retrofit countermeasures exist in both girder ends and main tower.

(1) to equip dampers at main girder ends to increase the damping force , reduce the longitudinal displacement, and avoid failure the bearing supports(image of install of damper is shown in Fig. 7)
(2) to install the auxiliary horizontal bearing supports at main girder ends to avoid the excessive transverse displacement of main girder
(3) to install the auxiliary vertical bearing support at intermediate bearing support line (pier 2) to avoid the over turning of the tower in transverse direction
(4) to exchange and enlarge the size of some parts of existing bearing supports to reduce the stress within the allowable stress
(5) to increase the throat thickness of welding of sole plate in existing bearing supports
(6) to add horizontal stiffener in end longitudinal girder in main girders to protect the buckling the web plate
(7) to add the plate by welding in metal touch connection of the main tower to increase the tensile strength caused by seismic forces.

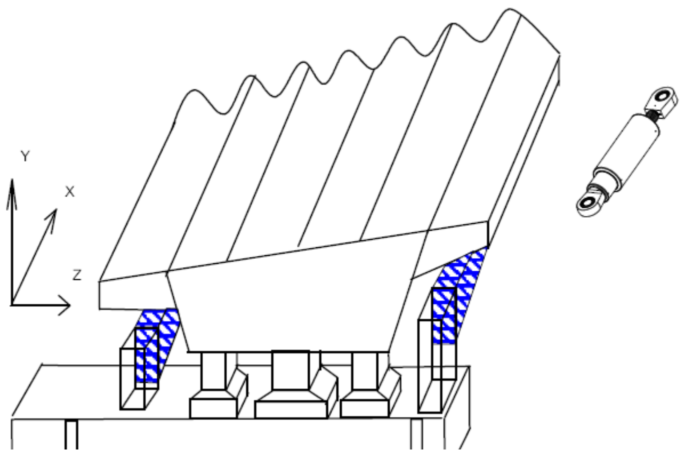

Fig. 7 Image of damper installed at girder end and sketch of velocity proportion damper.

6 VERIFICATION OF RETROFIT EFFECT BY DYNAMIC ANALYSIS

The retrofit effect is verified using nonlinear analysis for bending moments, displacements, stresses, and liquefaction as follows.

(1) Bending moment of main girder
Longitudinal bending moment become under the yield moment for all ground motion, but some part of cross section of main girder enter the plastic region due to the transverse bending moments. The maximum strain in outer edge of main girder becomes twice of yield strain, but as mentioned afterward this value does not give severe effect for car driver and repair during or after the large earthquake judging from maximum and residual deformation.

(2) Bending moment of RC pier
Stress of pier bottom exceeds the yield strength but is below the ultimate strength.

(3) Bending moment of steel piers and tower
Bending moments of steel pier was retrofitted several years ago. Therefore stress of the pier bottom is below the allowable stress. Bending moment of tower in both longitudinal and transverse directions is below the yield moment.

(4) Relative displacement of both end girders
Relative displacement between main girder and piers (pier 1 and 4) in both main girder ends reduce to the value shown in Table 1 due to the effect of damper, and these values are under the capacity of bearing supports regarding the horizontal movement. Therefore worry about the collision of girders clear up.

(5) Stresses of members in bearing supports
Stresses of members in bearing supports are al most reduced below the yield stress. Members of which stresses are beyond yield stresses (these are set bolts, side blocks, and caps) are changed to high strength materials.

(6) Residual displacement of piers and tower
Residual displacements of top of piers and tower become below one tenth of allowable values.

(7) Effect of liquefaction
Soil around pier 1 at the level from -10.6m to -15.65m is pointed out to have a possibility to liquefy during large earthquake. After the analysis considering the effect of liquefaction of the soil, it becomes clear that section forces and displacement at major point have only 0.5% difference. These differences have no meaning in retrofit design.

7 ELASTO PLASTIC FE ANALYSIS FOR DECK PLATE

The deck plate of main girder produce twice the strain of yield strain according to the nonlinear analysis of bridges as above mentioned. To clarify the effect of this strain on car traffic, we conducted the elasto-plastic finite element analysis for deck plate consist of three panels in longitudinal direction and over hang flange only in transverse direction as shown in Fig.9. Fig.10 shows the results of analysis which says maximum residual deformation is below 10mm, and the shape is smooth. Therefore the degree of plasticity is judged not interfere the car traffic after the earthquake.

8 CONCLUSIONS

According to nonlinear analysis of target cable stayed bridge, we found weak members for large earthquake. Considering retrofit countermeasure for this bridge, the strengthened bridge was analyzed again. The behavior of retrofitted bridge satisfies the criteria of Japanese specification for highway bridge. Install of damper and displacement restrainer, and strengthening of component of bearing support are useful countermeasure. Nonlinear time history analysis using tri-linear back bone and hysterecis loops successfully used. Furthermore fiber analysis and FE analysis are successfully used to check the behaviors of this bridge for large earthquake.

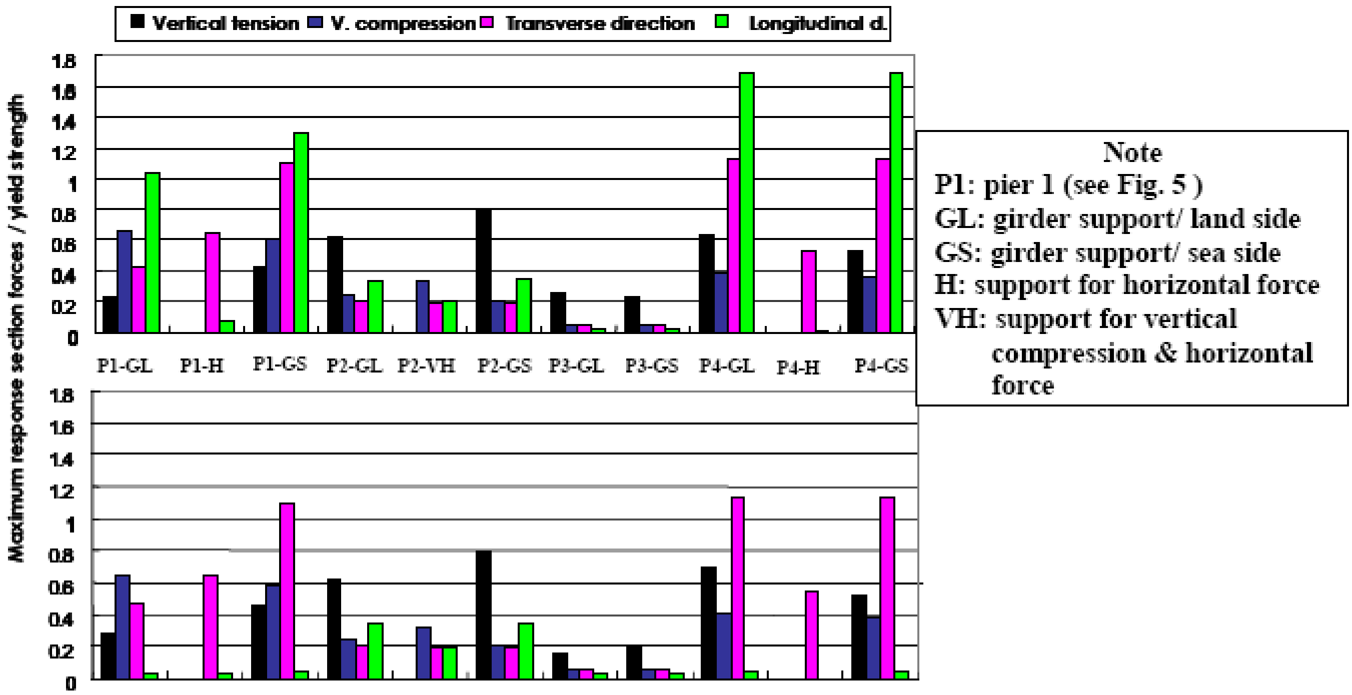

Fig. 8 Comparison of response section forces before (upper) and after (below) retrofit.

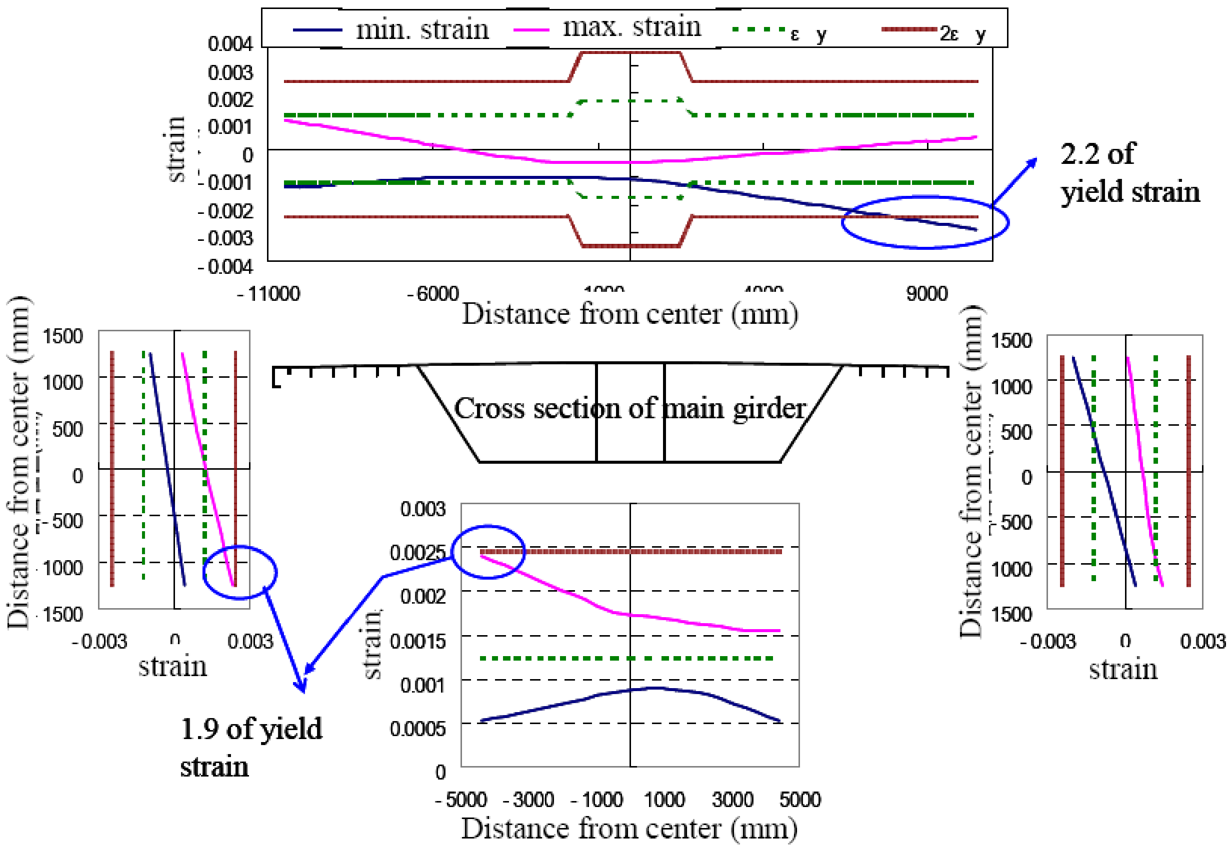

Fig. 9 Strain distribution on main girder.

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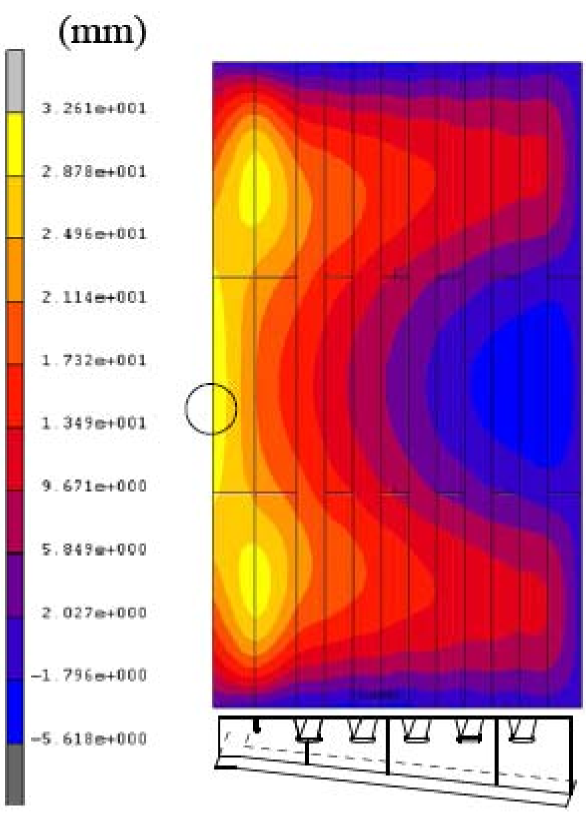

Fig. 10 Distribution of maximum vertical displacement of deck plate when 2 yield-strain is induced at edge of mid span plate (circle point).

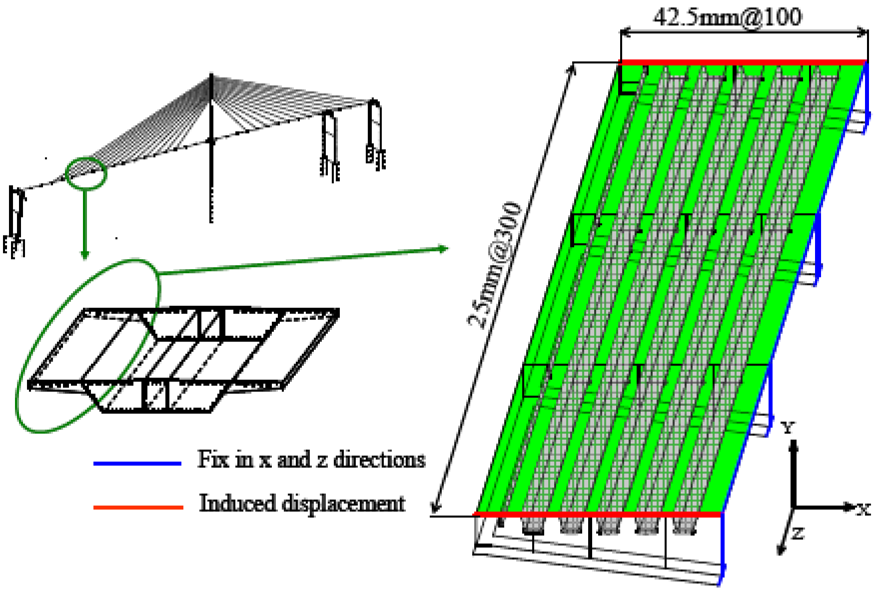

Fig. 10 Analytical model of steel deck system for Finite Element Analysis.