SEISMIC RETROFIT DESIGN OF TEMPOZAN CABLE-STAYED BRIDGE

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Abstract

This paper describes the seismic retrofit for Tempozan Bridge on Hanshin Expressway. Tempozan Bridge is continuous three-span of 640m (120+350+170m) cable-stayed bridge. For evaluating the seismic performance of this bridge, huge possible earthquakes at the bridge site are considered as input motions of 3-D dynamic analysis. As the result of the analysis, the scenario of seismic damage and policy of retrofits are determined. It is evaluated that adopting the shear panel dampers at diagonal braces of the towers, is particularly effective for absorbing the seismic energy. The seismic performance of shear damper is verified experimentally.

Introduction

After 1995 Kobe Earthquake, seismic retrofits for existing bridges have been implemented steadily in Japan. Hanshin expressway had already completed seismic countermeasures such as reinforcing the piers and adopting the bridge restrainer systems for most of general elevated bridges. However, in case of long-span bridges or special type bridges, the seismic countermeasures are making little headway because these bridges require advanced analysis techniques and it is not rational to apply the same methods for general bridges. When these bridges were damaged heavily, the influence such as cost and time for recovery could be greater than those of general bridges. Therefore, early seismic retrofits for long-span bridges or special-type bridges are recommended.

As a case example for seismic damage of cable-stayed bridge, Higashi Kobe Bridge (Bridge length; 885m, center span length; 485m) on Hanshin expressway at 1995 Kobe Earthquake is known. This bridge has a long natural period of 4.4 second for longitudinal direction, it caused little damage in longitudinal direction, but in transverse direction, wind shoes which support the transverse force were destroyed and secondary eye-bar pendulum supports which resist constant uplift force were broken. As the result, side span of the bridge was lifted up and the difference in level of 1m occurred in the road surface. In the case of the other cable-stayed bridges on Hanshin expressway, the seismic damage from 1995 Kobe Earthquake was minor. However, there is a risk which suffers from a huge possible earthquake in the future, it is necessary to evaluate the seismic performance of present structural condition and to determine the optimal retrofitting. In this paper, the

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study for seismic retrofit on Tempozan Bridge is described. Tempozan Bridge is continuous three-span of 640m (120+350+170m) cable-stayed bridge as shown in ***Fig.1*** and ***Fig.2***.

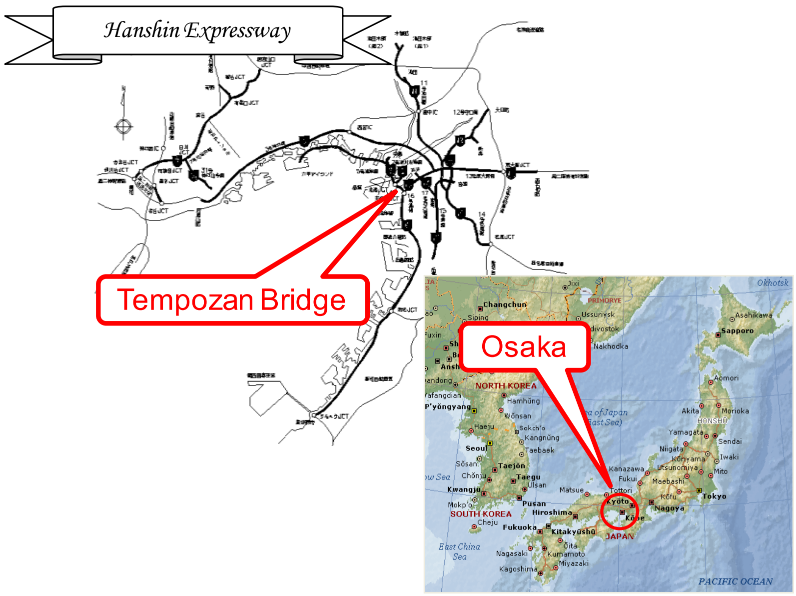

Fig. 1 Location of Tempozan Bridge

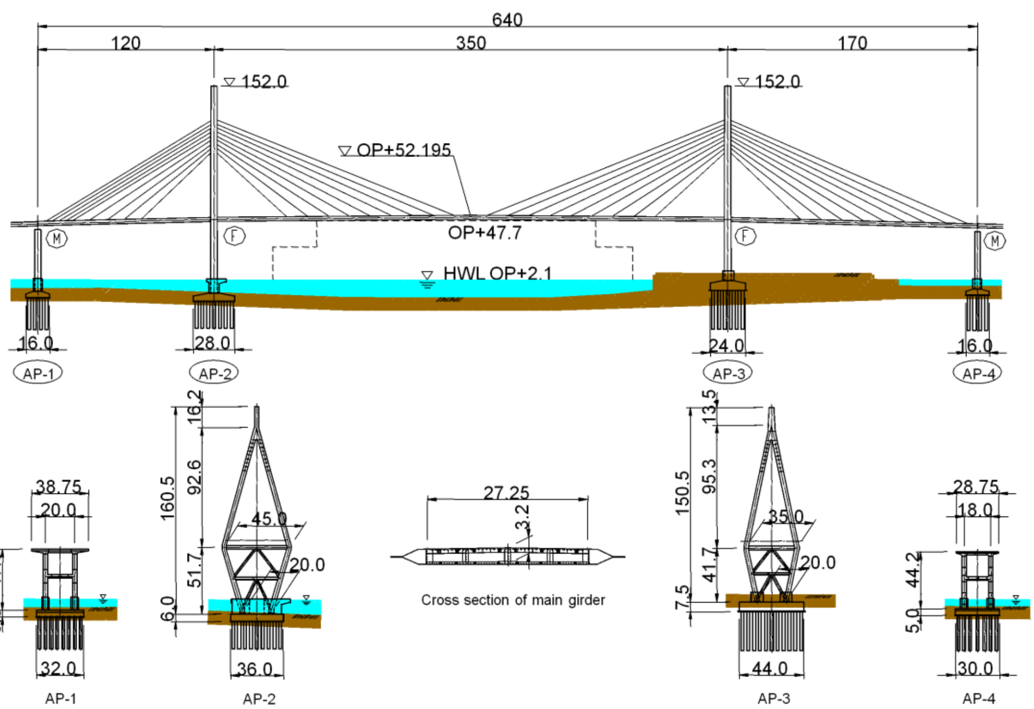

Fig. 2 General view of Tempozan Bridge

Introduction of Tempozan Cable-stayed Bridge

General view of Tempozan Bridge is shown in ***Fig.2***. It has continuous steel box-girder which is supported by stay cables from two steel towers, and two steel rigid-frame piers at both ends. For constant uplift force caused by imbalanced side span distribution, pendulum supports are adopted at both ends.

Because the main girder is 50 meters above water level, this bridge is longitudinally fixed at two flexible towers AP2 and AP3 in consideration of seismic design. It results in longer natural period of 3.7 seconds in the longitudinal whole structure oscillation mode and this long period alleviates the seismic inertia force on the superstructure.

For the transverse seismic force, horizontal beam and diagonal bracing are installed at lower section from the main girder in the towers and at both ends on the rigid-frame piers, the wind shoes are installed to prevent too much displacement.

Evaluation of seismic performance

(1) Input earthquake motion

As for input earthquake motion, natural period characteristic of Osaka Bay area is considered. For this study, the scenario earthquake at the bridge site, which was made by hybrid method on the basis of three-dimensional subsurface structure of Osaka basin, is used. To put it concretely, at first basic design, spectral envelope (hereinafter referred to as spectral envelope) is made from six varieties of acceleration response spectrums which are selected from preliminary examination; next adjusted input wave for final study is adopted. The comparison of the spectral envelope and the acceleration response spectrums of Japanese specifications for highway bridges are shown in ***Fig.3***.

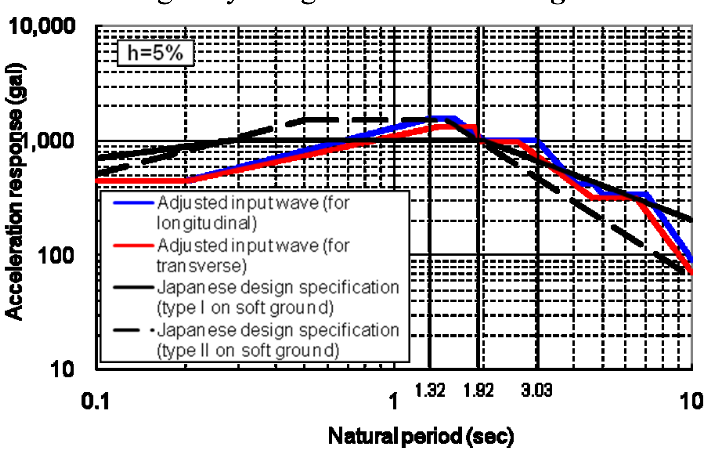

Fig. 3 Comparison of spectral envelope and acceleration response spectrums of Japanese specifications for highway bridges

(2) Dynamic analysis model

Cable stayed bridge is very flexible structure, especially in Tempozan Bridge, large horizontal displacement is caused at the earthquake. Therefore, the following influences to the sectional force on the member with geometric nonlinearity were studied by limited displacement analysis.

a) The horizontal force on pendulum support caused by the relative displacement on the end of girder.
b) The additional bending moment at the member on the tower caused by longitudinal earthquake input.
c) The additional bending moment at the member on the tower caused by transverse earthquake input.

As the results of the analyses, it became clear that the influences with geometric nonlinearity were sufficiently small except to the horizontal force on pendulum support. The pendulum support was modeled as illustrated in ***Fig.4***. Whole bridge model for dynamic analysis is shown in ***Fig.4*** and each member models which were considered as non-linear elements are shown in ***Table 1***. The results of eigenvalue analysis are shown in ***Table 2***.

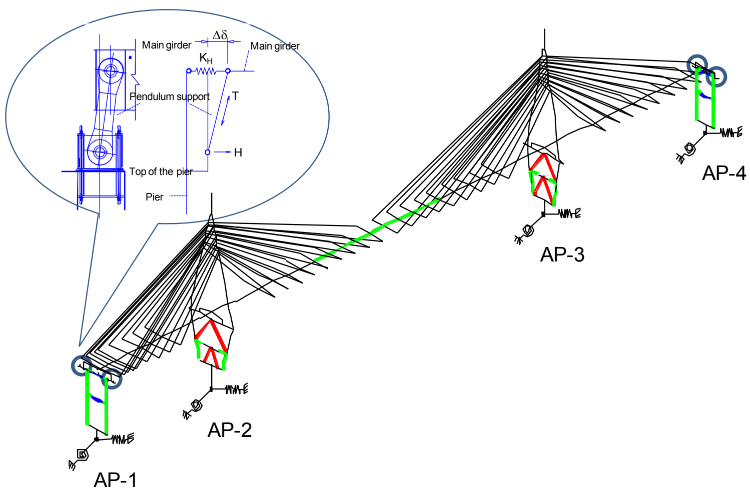

Fig. 4 Dynamic analysis model for whole bridge

(3) Result of analysis and policy of retrofit

Non-linear time history response analysis for dynamic analysis model shown in ***Fig.4*** was conducted making use of the input earthquake motion which described above.

Table 1 Non-linear member models

Bending members	Axial members (Diagonal bracings of the towers)	Shear members (Horizontal beams of the piers)	Pendulum supports
M-ϕ curve (Bilinear model)	N-ε curve (Elastic-perfectly plastic solid model)	S-γ curve (Bilinear model)	H-$\Delta\delta$ curve (Nonlinear spring)

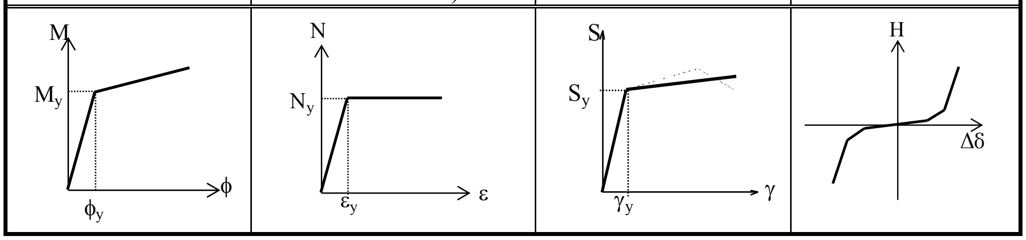

Table 2 The result of eigenvalue analysis

Dominant Vibration Mode	Longitudinal	Transverse	
		Main span	Side span
Natural Period	3.03 [sec]	1.92 [sec]	1.32 [sec]
Effective Mass Ratio	M_x =18.8%	M_x =9.8%	M_x =16.3%

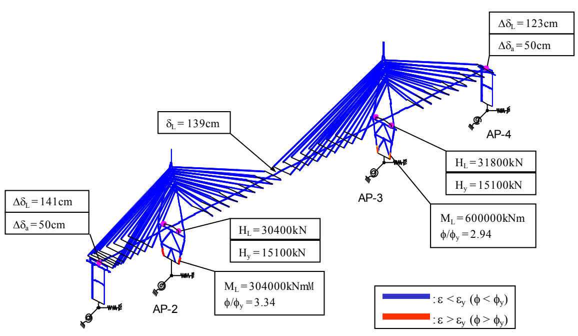

Fig. 5 The analytical results for longitudinal response

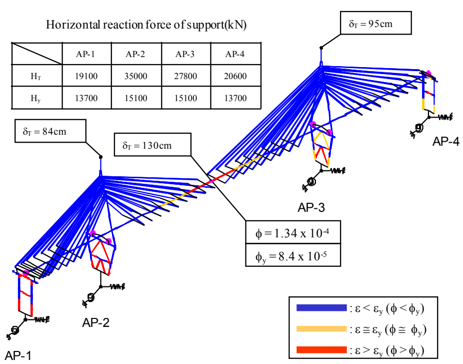

Fig. 6 The analytical results for transverse response

a) Responses of longitudinal direction

The analytical result for longitudinal response is shown in ***Fig.5***. This bridge is fixed at two towers and so seismic inertia forces are transmitted to the tower members through the supports at the towers. Therefore, the horizontal reaction forces of the supports at the towers and response curvatures at the base of towers enlarge, so each response value exceeded the yield value largely. At the same time, relative displacement responses between main girder and the top of the end piers are as large as approximately three times allowable displacement of the pendulum supports.

b) Responses of transverse direction

The analytical result for transverse response is shown in ***Fig.6***. The horizontal reaction forces exceeded the allowable horizontal reaction forces at every supports. The response sectional forces exceeded the yield resistances on the diagonal braces, horizontal beams and column members at the towers. The response strains at the diagonal braces were 1.4 to 2.4 times as large as yield point, and the response curvatures on horizontal beams were approximately ten times as large as yield point at the section near both ends of the beam. The response curvatures at the basis of the towers were approximately 1.5 times as large as yield point. The end piers are rigid double-deck frame pier which have large aspect ratio, large shearing force acting on the horizontal beams and the response shearing force at the central part of the beams were approximately 10 to 20 times of the yield point. The responses of curvature at the basis and the corner zones of the end piers exceeded the yield point at most parts; especially it was more than four times of the yield point at the base of AP-4 pier.

c) The Scenario of seismic damage and policy of retrofits

Fatal damage situations that would happen in Tempozan Bridge by huge earthquake are evaluated as follows:

i) The horizontal reaction force for transverse direction or excessive horizontal displacement for longitudinal direction cause damage of the wind shoe which is linked to damage of the pendulum supports. When the pendulum supports are destroyed, excessive displacement and stress as shown in ***Fig.7*** would act, and there could be a possibility of the collapse of the bridge.

ii) When the pivot supports which fix this bridge is destroyed, the main girder would move and hit to the tower as shown in ***Fig.8***. Then there is possibility of the damage or collapse of the tower, which would directly lead to collapse the bridge.

iii) Although it would not directly lead to collapse of the bridge, there is a possibility that buckling or damage of the member which caused from excessive stress, would lead to unacceptable residual deformation.

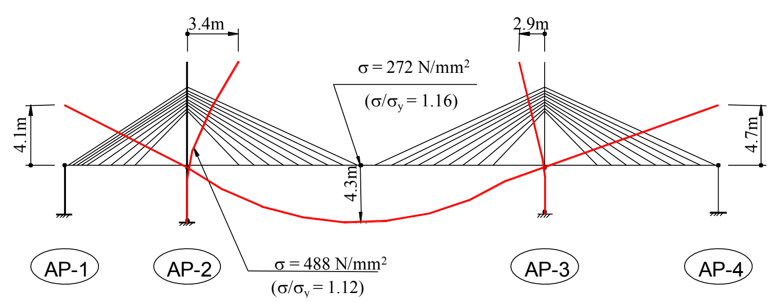

Fig. 7 Response displacement and stress after destroying the pendulum support

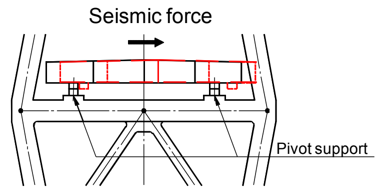

Fig. 8 Collision between tower and main girder after destroying the pivot support

Among these damage situations, i) and ii) are really fatal damage situation and these must be evaded. However, in the case of iii), the required performances (e.g. allowable deformation of the member) need to be determined concretely with consideration for level of damage, effects on other member, repair difficulty level, and so on. The policies of retrofits for Tempozan Bridge based on the scenario of seismic damage are summarized as ***Table 3***.

Table 3 The policy of retrofits for Tempozan Bridge

Members			Input direction	Policy for retrofit
Supports	Pendulum support		Longitudinal	Adopting a device which limits relative displacement between main girder and end pier less than 50 cm
	Wind support		Transverse	Strengthen the supports or adopting displacement limit devices
	Pivot support		Longitudinal Transverse	
Towers	Column member at near the base		Longitudinal Transverse	Guarantee required ductility by obtaining enough energy absorption
	Diagonal brace		Transverse	
	Horizontal beam		Transverse	Guarantee required ductility by reinforcing for local buckling
End piers	Horizontal beam		Transverse	Strengthen for shear buckling and guarantee shear ductility
	Column member	Corner zone	Transverse	Guarantee required ductility by reinforcing for local buckling
		Near the base	Transverse	
Main girder	Center of main span		Transverse	Prevention of buckling of outer web
Whole bridge			Longitudinal Transverse	Decrease of seismic response by installation of seismic isolation device or damping device and guarantee required ductility of each members

Contents of retrofitting

Based on the policies summarized as ***Table 3***, seismic retrofitting for Tempozan Bridge will be executed. The methods and the effect of each object members are described as follows.

(1) Towers

a) Methods of retrofitting

At lower part of the towers, concrete will be filled as shown in ***Fig.9*** for the purpose of guaranteeing the ductility for longitudinal and transverse direction. Additionally, shear damper that use low yield point steel panels will be installed at diagonal braces of the towers which resist a horizontal force during earthquake as shown in ***Fig.9*** for the purpose of decreasing the acting horizontal force less than shear yield point and absorbing the seismic energy.

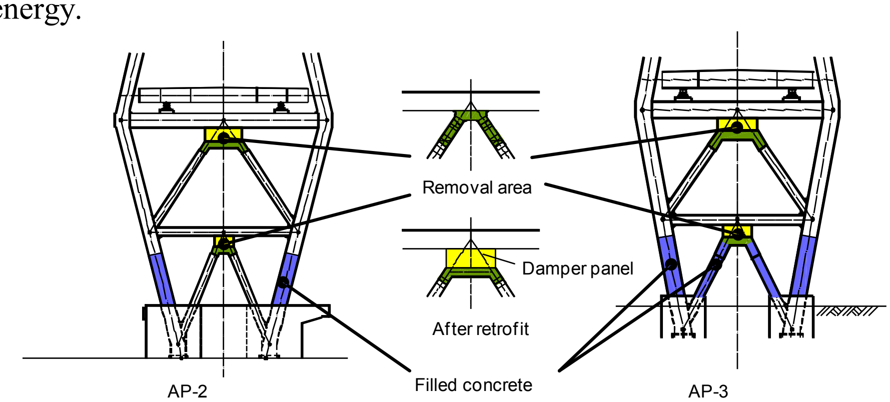

Fig. 9 The retrofit method for the towers

b) Verification of effect

The shear damper was modeled as shown as ***Fig.10*** and ***Fig.11***. H_y in ***Fig.11*** is determined as 90% as horizontal component of buckling capacity at the diagonal braces. Initial size of shear damper was decided that permissible strain of the shear panels are 3%.

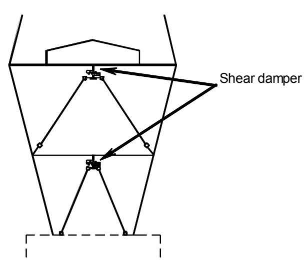

Fig. 10 The spring modeling of shear dampers

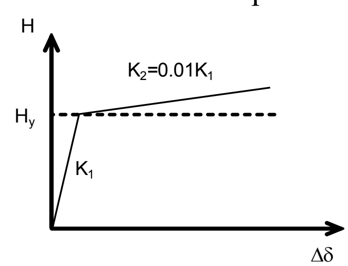

Fig. 11 Relationship between horizontal force and displacement of shear damper

Comparison of seismic response displacements at focused points and response curvature of main girder between before and after installation of shear damper are shown in ***Fig.12***. Response displacement in transverse direction at the top of AP-2 was increased slightly caused by plastic deformation of the shear damper. However, the other responses including horizontal reaction force of the supports were decreased by adoption of the shear dampers. Maximum responses on the tower members were all fitted into elastic range as shown in ***Fig.13*** except the shear dampers.

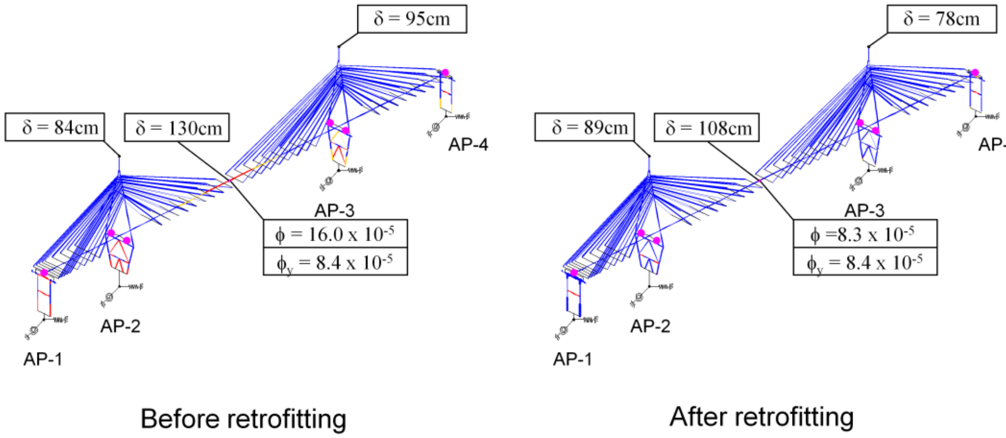

Fig. 12 The effect of the shear dampers in seismic response

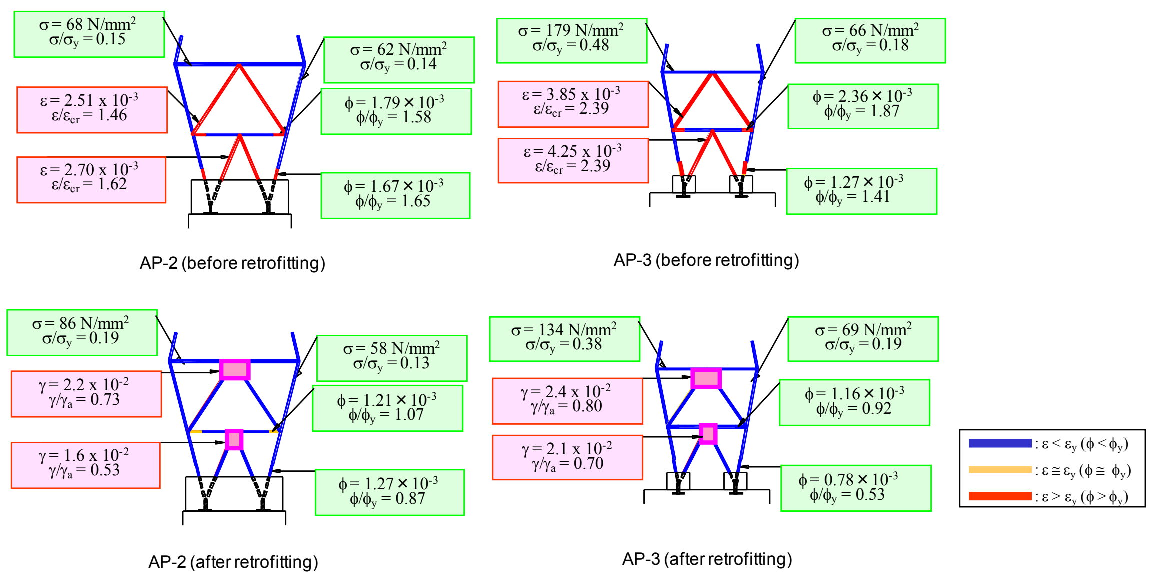

Fig. 13 Maximum responses on the tower members

Focused on seismic responses on shear dampers, it is confirmed that maximum response of shear strain did not reach to their allowable value of 3% but the number of times which exceed elastic range was quite a lot. Therefore, decrease of allowable shear strain on the shear damper that caused by repeated plastic deformation was concerned, cumulative plastic shear strain which is calculated as shown in ***Fig.14*** was determined as performance indicator and verification test was executed.

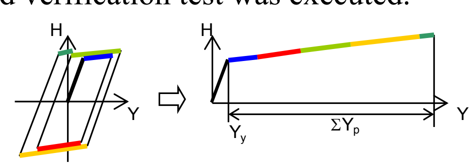

Fig. 14 Calculation of cumulative plastic shear strain

(2)End piers

To achieve the performance described in ***Table 3***, required ductility for transverse direction at end piers must be guaranteed. Concrete filling in near the base of the column members and adopting horizontal stiffeners in horizontal beams will be implemented for increasing their ductility.

(3)Bearing supports

To achieve the performance described in ***Table 3***, the pivot supports (fixed support) on the towers will be strengthened in longitudinal direction, and displacement-limiting devices and high-damping rubber bearings will be installed on the end piers for limiting the relative displacement between main girder and end pier. In transverse direction, displacement-limiting devices will be installed for preventing the destruction of wind shoes on the end piers. In vertical direction, uplift-preventive cable will be installed between main girder and the end piers as a fail-safe device on the assumption that the pendulum support would be destroyed.

<u>Verification test for shear damper</u>

(1) Experiment objective

Verification test for shear damper which are installed in gusset plate at diagonal brace of the towers was conducted in order to confirm or verify following items:

i) Hysteresis curve (H-γ relationship)
In order to verify the validity of hysteresis model for shear damper in dynamic analysis (shown in ***Fig.11***), the relationship between horizontal load and shear strain would be confirmed by cyclic loading test.

ii) Deformation performance (Low-cycle fatigue)
Allowable shear strain γ_a for deformation performance check and allowable cumulative plastic shear strain $\Sigma\gamma_p$ (shown in ***Fig.14***) would be configured from the result of these tests.

iii) Influence of axial force that acts on the shear damper
Influence of axial force to the performance of the shear damper would be confirmed because they are installed in gusset plate at diagonal brace of the towers where vertical axial force acts due to dead load.

(2)Specimen and setup

The specimen and the setup of these tests are shown in ***Fig.15*** and ***Fig.16***. Three same type specimens which were designed as allowable shear strain of 6% were made and they were named C1, C2, and C3. Each specimen was loaded horizontally under axial force of 0kN, 240kN (for dead load situation) and 360kN (for live load situation). Horizontal loading was conducted with displacement (strain) control and increased step by step of 1% strain in both directions.

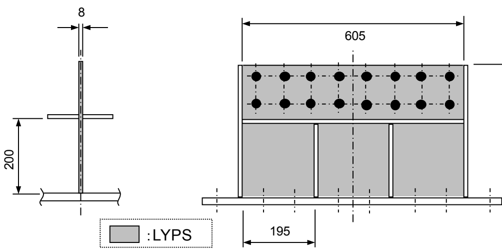

Fig. 15 The specimen of the verification test

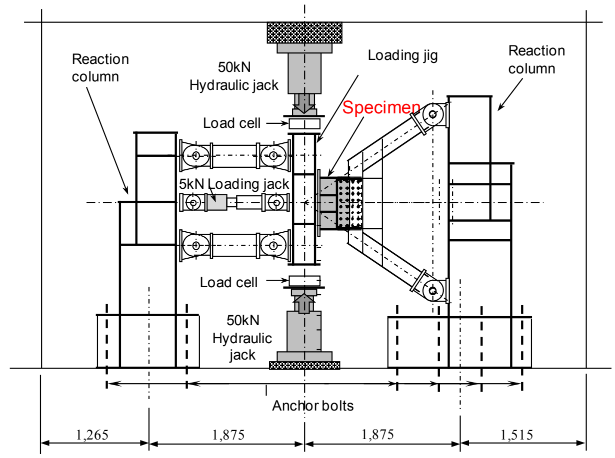

Fig. 16 The setup of the verification test

(3)Results of the test

All tests were stopped because large gap between specimen and supporting jig occurred. Every specimen were loaded cyclically in both direction to 9% of shear strain, but C1 specimen under no axial force continued to be loaded monotonically until 18% of shear strain. Decreases of load were not observed clearly in all specimens during the tests.

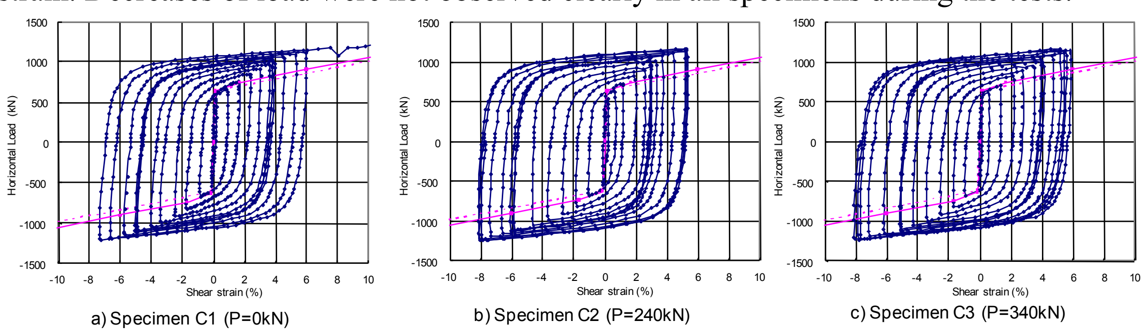

Fig. 17 H-γ relationship f each specimen

a) Horizontal load-shear strain relationship

Relationships between horizontal load and shear strain (H-γ relationship) of each specimen are shown in ***Fig.17***. The bi-linear relationship shown in ***Fig.11*** and tri-linear relationships which consider the effect of the members to restrain the shear panel are also shown in the figures. According to the graph, H-γ relationship of shear damper has the following characters:

i) Yield load H_y is increased during the cyclic load in the plastic region.
ii) Stiffness (shown as slope of H-γ curve) in plastic region is decreased as frequency of repetition loading increases.
iii) H- γ relationship is affected by the stiffness of not only the shear panel but

the members that restrain the panel.

iv) The difference of the hysteresis with the axial force is seldom seen.

b) Cumulative plastic shear strain and deformation performance

As the results of dynamic analysis shown in ***Fig.12***, cumulative plastic shear strains of four shear damper panels which would be adopted in Tempozan Bridge are 0.211 to 0.377. It shows that the shear dampers of this bridge will suffer about 0.4 of cumulative plastic shear strain during a huge earthquake, therein stable performance will be required. The cumulative plastic shear strains of the test specimen are estimated from H-γ relationship shown in ***Fig.17*** and they are 1.745 in C1 and 2.064 in both C2 and C3. It is confirmed that the shear dampers are able to expect sufficient performance during a huge earthquake from the fact that the decrease of loading capacity is not observed during the test.

Conclusions

This paper describes the study on seismic retrofit of Tempozan Bridge in Hanshin Expressway and confirmatory test for shear damper. Contents are collected below:

- The input earthquake motions for this study were developed from the scenario earthquakes which would be estimated to occur at the bridge site.
- The influences with geometric nonlinearity of the bridge were studied before conducting 3-D dynamic analysis. As the result of the study, they are sufficiently-small except to the horizontal force on pendulum support.
- As the results of the dynamic analysis, the following scenario is assumed; when the pivot supports which fix this bridge is destroyed, the main girder moves and hits to the tower. Then there is possibility of the damage or collapse of the tower, which would directly lead to collapse of the bridge.
- Although it would not directly lead to collapse of the bridge, there is a possibility that buckling or damage of the member which caused from excessive stress, would lead to unacceptable residual deformation.
- Responding to the assumed scenario, concrete will be filled at lower part of the towers for increasing the ductility, and shear damper will be set at diagonal braces of the towers for the purpose of absorbing the seismic energy.
- Verification tests for shear damper were carried out. As the result, their seismic performances were verified and the difference of the hysteresis with the axial force was seldom seen.

References

1. Ishibashi T, Hamada N, Kanaji H, Noguchi J and Kobayashi Y. “Seismic Evaluation on existing 3-span continuous steel cable-stayed bridge which is 640m long”, Proc. of the 8th Symposium on earthquake-resistant design of bridges based on the post-earthquake retained bearing strength method, Feb 2005 (In Japanese)