# Seismic Retrofit Strategy using Damage Control Design Concept and the Response Reduction Effect for a Long-span Truss Bridge

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**Abstract**

A long-span cantilever truss bridge with 980m long is located in the Hanshin Expressway in the Port of Osaka and has seismic vulnerability which causes severe damages to not a few main members due to "Level 2 Earthquake", corresponding to the Safety Evaluation Earthquake. In this project, the damage control design concept is employed to achieve rational seismic retrofit. The concept differentiates main members, which support vertical load, from sub members against lateral forces such as seismic force or wind force. According to the concept, main members should be designed to be elastic and sub members are allowed to behave in plastic deformation in order to provide adequate damping to the entire bridge. In this paper, floor seismic isolation system and hysteretic damper bracing are proposed based on the damage control design concept and their effects of response reduction from a series of dynamic analyses are also described.

**Introduction**

The Hyogo-ken Nanbu earthquake hit Kobe, Japan at dawn on January 17, 1995 and the structures on the Kobe route and the Wangan route of the Hanshin expressway suffered damages. After this devastating earthquake, seismic retrofit projects have been accelerated in Japan. The first step is to increase shear strength and ductility of columns using steel jacketing, RC jacketing or CFRT jacketing. The second step is for superstructures such as installing of cable restrainers in terms of existing steel plate restrainers, replacement of vulnerable bearings, extension of cap-beams against girder dropping and reinforcement of end diaphragms. However the retrofit for long span bridges is lagging behind that for ordinary bridges in Japan. To our knowledge, the seismic retrofit project for the Minato Bridge must be the first extensive challenge as the retrofit of a long span bridge in Japan.

The Minato Bridge was completed in 1974 and is the third longest truss bridge in the

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world. In the assessment of the seismic safety of this bridge, design seismic load was reviewed considering extremely large earthquakes whose return period exceeds 1000 years. Using these earthquake motions, a great number of dynamic analyses are performed and it is found that a lot of members can't remain in elastic condition. There are two alternatives for retrofit such as reinforcement of members or seismic response modification for the retrofit. We selected the latter method, because reinforcement of many members should be uneconomical and unreasonable. In addition, the damage control design concept which keeps main members elastic and expects hysteretic damping of sub members such as lateral bracings is employed. It is of considerable practical concern because the residual displacement of the entire bridge should be very little and traffic can be open immediately after an earthquake. This implies that the restoration cost and the economic loss will be dramatically reduced.

## Bridge Description

The Minato Bridge is located in the Osaka City and the long span bridge on the Wangan route of the Hanshin expressway. This cantilever truss bridge consists of two 235m side-spans and a 510m main span and there are two-layer of floor system with four lanes each (**Figure-1**). Floor system in one-layer consists of 14 floor structure systems. The bearings of intermediate-portions are fixed against horizontal movements and their design capacity is 131.4MN for dead and live load. The foundation type at intermediate-portion is caisson and that at edge piers is steel pile.

The as-built seismic design was performed using modified seismic coefficient design method which considered the dynamic characteristics based on dynamic analysis and the original design acceleration response is 250 gal.

## Assessment of the As-built Structure

### *Analysis Model*

A 3-D global model as shown in **Figure-2** is used to analyze the current seismic performance as well as the retrofitted structure. The superstructure is composed of elastic members for main members and the damping ratio is assumed to be 2%. Ground springs at P-2 and P-3 consist of non-linear horizontal springs and rotation springs and those at P-1 and P-4 are elastic springs. The non-linear springs are developed using static pushover analysis for the caisson-ground interactive model and the Hardin-Drnevich model is applied for restoring characteristics of the caisson.

### *Modal Analysis*

In order to make sure of vibration characteristics for the as-built structure, modal analyses were performed using the global bridge model. **Figure-3** provides the dominant mode for the longitudinal and transverse directions. The first mode for the longitudinal direction shows jackknife-shape at hinges and the natural period is 2.8sec. The second mode with 1.4sec natural period expresses that the suspended span goes up and down. On the other hand, the first mode for the transverse direction presents that suspended span sways from side to side and the period is 4.4sec. The second mode with 1.9sec period expresses that only side span bends.

The relationship between design acceleration response spectra for this bridge and their natural periods are shown in **Figure-4**. Only the acceleration of the first mode for the transverse direction is relatively low. However all other accelerations are very large: the acceleration for the longitudinal is around 800 gal and that for the transverse is over 1G.

### *Current Seismic Performance*

Response spectrum analysis and time history analysis using the site-specific earthquake motions are performed in order to evaluate the current seismic performance of this bridge.

According to the response spectrum analysis, the effect of each mode to the force response in the members or structural components can be clear. The first mode for the longitudinal direction affects the lower chord members in the side spans, the upper chord members in the center span near the tower, and main bearings at towers. The first mode for the transverse direction accounts for almost all member force of upper chord members and lower chord members in the center span. The second mode for the transverse direction makes up the great portion of member forces in the side spans.

The time history analysis reveals the member stress for the ground motions considering the site-specific characteristics. Giving that the stress ratio should be the quotient of maximum stress and the allowable stress, inelastic members can be expressed by the stress ratio over 1.0. Inelastic members due to the longitudinal motions locate on chord members near the edges of the bridge and hinges between the cantilever portion and the suspended span. On the other hand, those for the transverse motions locate on chord members near the edges of the bridge and diagonal members, vertical members and lower chord members around towers. This implies that the first mode for the longitudinal direction and the second mode for the transceivers are crucial for the bridge.

## Retrofit Strategy

### *Damage Control Design*

As this retrofit strategy, the damage control design concept in which damages are allowed in only sub members which support lateral forces is applied. These members are expected to behave in elasto-plastic condition and generate adequate hysteretic damping. However main members which support vertical force such as dead load and live load should be almost elastic. This concept has been already employed in the field of high-rise buildings in Japan. Using this concept, the structure would have only small residual deformation and can open traffic just after even an extreme large earthquake. Moreover restoration cost after such earthquake can be significantly reduced, because the main members remain almost elastic.

***Seismic Response Modification***

There are seismic isolation devices and energy absorption devices as seismic response modification devices. These devices are key technique for the damage control design. As mentioned above, it needs stable elasto-plastic behavior and damping for sub members. These devices, which are mounted in sub members, are very useful to reduce the seismic response. In applying them, the acceleration response spectra provide important fodder for making a decision. For this bridge, the first mode for the longitudinal direction with 2.8sec period and the second mode for the transverse with 1.9sec are critical modes judging from their overstress ratios. Period elongation is very useful for the former mode, however it will make acceleration increase for the latter by the look of the design spectra. Thus, it is decided that isolation technique is employed for the longitudinal and damping addition is used for transverse.

## Seismic Response Modification Devices for the Longitudinal Direction

***Floor Deck Isolation***

This bridge has two-layer floor decks with 6 plate girders, which are placed on the cross beams of the main truss. The weight of floor systems amounts to approximately 200MN, which accounts for 40% of the total superstructure weight, and they are supported by conventional steel bearings. According to the dynamic analysis, these existing bearings, which suffered damages due to even the Hyogoken-Nanbu earthquake whose epicenter is approximately 40km away from the bridge site, will be severely damaged. Under these circumstances, isolation bearings are adopted in terms of existing bearings in order to reduce the seismic response and avoid their damages.

There are several types of isolation bearings such as lead-cored rubber bearings, a high-damping rubber bearings or sliding bearings. In this bridge, sliding isolation system which consists of sliding bearings and lateral rubber springs is selected as illustrated in **Figure-5**. The reason is that lead-cored rubber bearings and high-damping rubber bearings which must provide a long natural period around 3 seconds require high-shaped bearings which can’t be installed in actual space at the site.

***Response***

Seismic isolation is based on period elongation and consequently the natural period of rubber springs are crucial for the force reduction of main truss members and the displacement increase of floor decks. Moreover, the friction coefficient of sliding bearings also affects these responses. In order to select optimal design values, the time history analyses considering several rubber stiffness and friction coefficients of sliding bearings are performed.

**Figure-6** in which natural periods of rubber springs and friction coefficients of sliding bearings are varied shows the relationship between the bearing displacement and the reduction ratio of chord member force. It was found that longer periods such as T=3.0 and 3.5 sec than the natural period of the first mode for the longitudinal direction reduce the member force and enlarge the bearing displacement by degrees with a decrease in friction coefficient. On the other hand, decreasing in friction coefficient in the case of shorter natural period than T=2.75 sec increases both member force and bearing displacement. The displacement demand for the clearance at the edge of girders is approximately 50cm. This design condition gives the optimal value which is the combination of T=3.0sec and $\mu$=0.05 and this provides the allowable member forces.

**Seismic Response Modification Devices for the Transverse Direction**

***Hysteretic Damper Bracing***

A steel hysteretic damper is composed of core steel member by low-yield point steel (LYP235) and the restraint hollow against the bucking of core steel. Under normal conditions, bucking strength of a steel bracing should be lower than its yield strength and hysteretic damping due to the cyclic loading can't be expected consequently. On the contrary, hysteretic damper bracings provide quite large damping without bucking.

The sway bracings of towers and lateral bracings on the lower side near towers will buckle under the compression force due to an extreme large earthquake in the transverse direction. In order to give adequate damping to the entire bridge and avoid the buckling or yield of main truss members, the design method in which these existing bracings are replaced with hysteretic damper bracings is selected (**Figure-7**).

***Layout of Hysteretic Damper Bracings***

The damping of bracings is required against the second mode for the traverse direction which is the critical vibration mode as mentioned above. According to the mode analysis, strain energy for each member can be obtained and the members which have large strain energy are expected to provide effective damping from the viewpoint of mode damping. Under these aspects, the sway bracings at the tower and the lower lateral bracings near towers which have large strain energy are determined to be replaced with hysteretic damper bracings as shown in **Figure-8**.

***Response***

**Figure-9 and 10** show the change in strain energy of the upper and the lower chord members by lateral damper bracings and sway damper bracings at towers respectively. It was found that the former are extremely effective to reduce the strain energy of lower chord members near towers, but the effect for the upper is almost never. On the other hand, the latter can reduce that of the upper chord in the center portion of the side span. **Figure-11** shows the change in member force of the upper and the lower chord members using both groups of damper bracings. The maximum reduction ratios for the upper and the lower chord members are 85% and 42% respectively.

## Effect of the Retrofit Strategy

The retrofit strategy using seismic isolation devices and hysteretic damper bracings is been practicing for the bridge. Now the overstress ratio R is defined as below.

$$R_i = \sigma max_i / \sigma a_i$$

where $\sigma max_i$ : maximum stress by the dynamic analysis (i=1,2,・・・)
$\sigma a_i$ : allowable stress (i=1,2,・・・)

The retrofit effect can be observed in **Figure-12**. It was found that the number of inelastic members is dramatically reduced in both the longitudinal direction and the transverse direction by this retrofit strategy. However some members remain inelastic and have to be replaced or strengthened in the final stage.

## Concluding Remarks

The following concluding remarks were derived from the results.

- The damage control design concept for the seismic retrofit of a long span bridge is very attractive and makes it possible to open traffic just after an earthquake and dramatically reduce the restoration cost and the economic loss.
- Seismic isolation devices and hysteretic damper bracings are very powerful tools for the damage control design. However it is necessary to examine earthquake motions and their response spectra, and judge the effect by applying these devices in the range of relatively long period.
- It was found that longer periods such as T=3.0 and 3.5 sec than that of first mode for the longitudinal direction reduce the member force and enlarge the bearing displacement by degrees with a decrease in friction coefficient in this sliding isolation sysytem. The optimal combination of the natural period of a rubber spring and the friction coefficient is T=3.0sec and $\mu$=0.05.
- Hysteretic damper bracings installed in the lower lateral bracings are extremely effective to reduce the strain energy of lower chord members near the tower, but the effect for the upper is almost never. On the other hand, sway damper bracings at towers can reduce that of the upper chord in the center portion of the side span.

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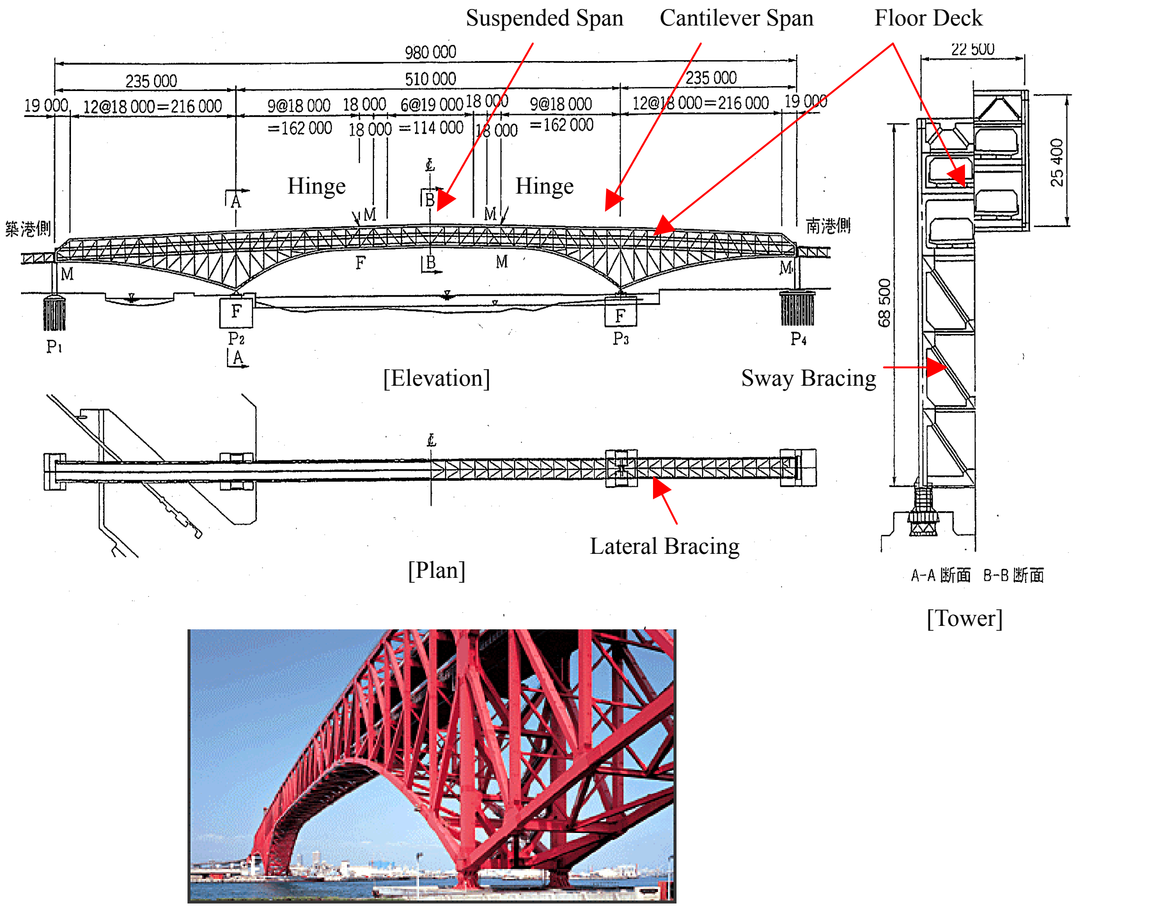



**Figure-1**　The Minato Bridge

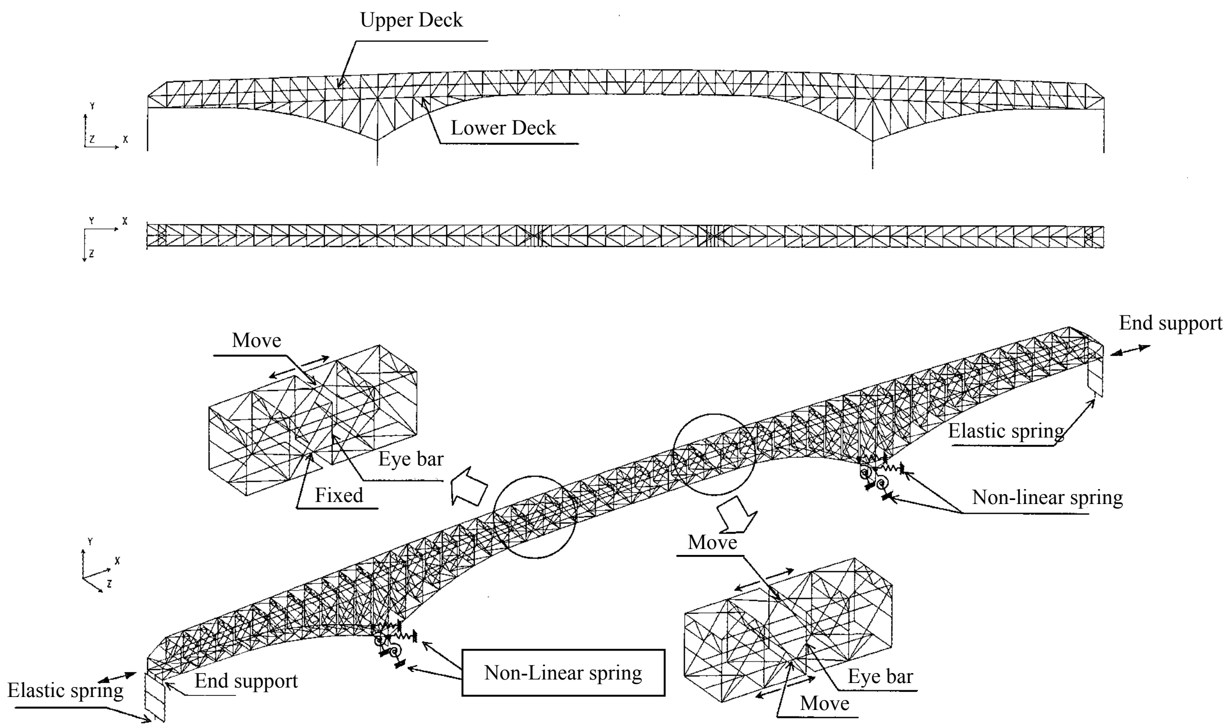
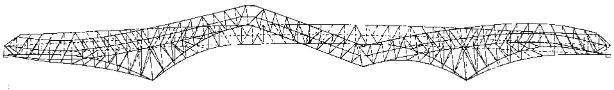
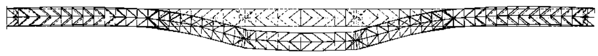


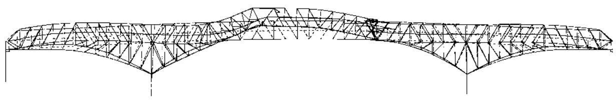

**Figure-2**　Analytical Model



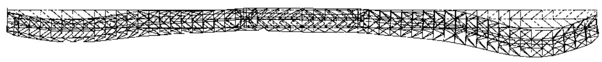
First Mode for Longitudinal Dir. (T=2.8sec)



First Mode for Transverse Dir. (T=4.4sec): Plan

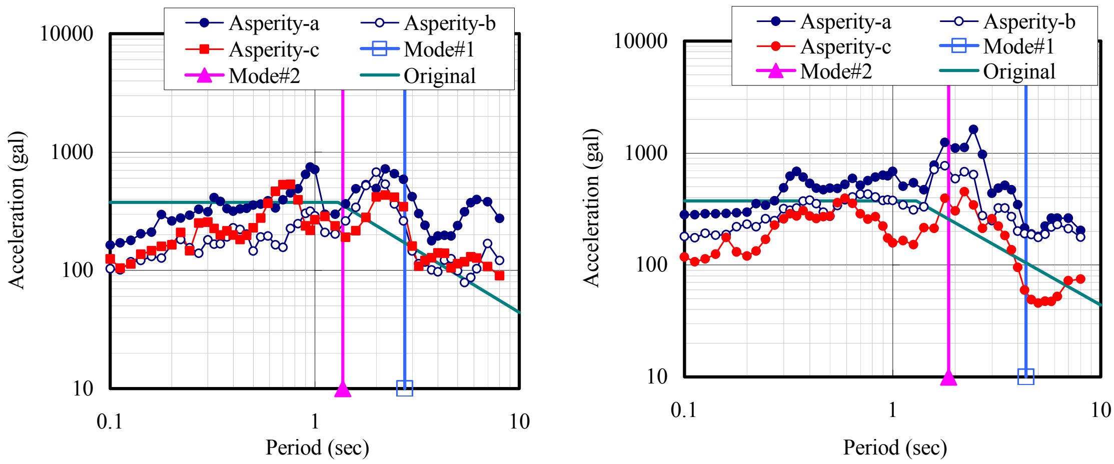


Second Mode for Transverse Dir. (T=1.4sec)



Second Mode for Transverse Dir. (T=1.9sec): Plan

**Figure-3** Vibration Mode




a) Longitudinal Dir.

b) Transverse Dir.

**Figure-4** Acceleration Response Spectra

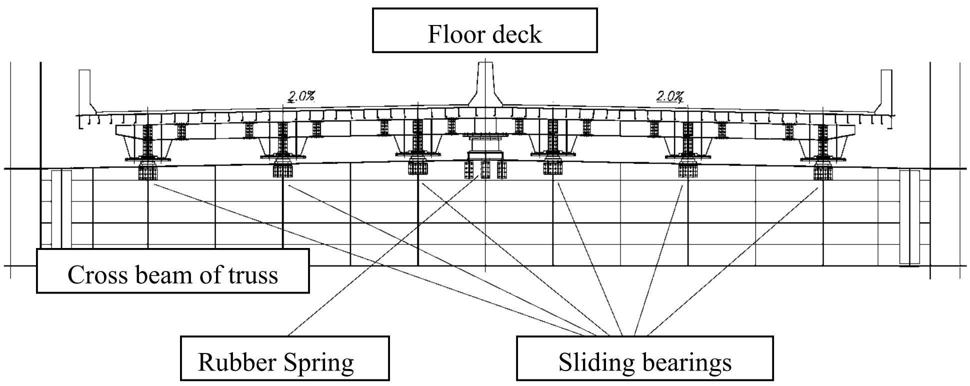



**Figure-5** Floor Deck Isolation System

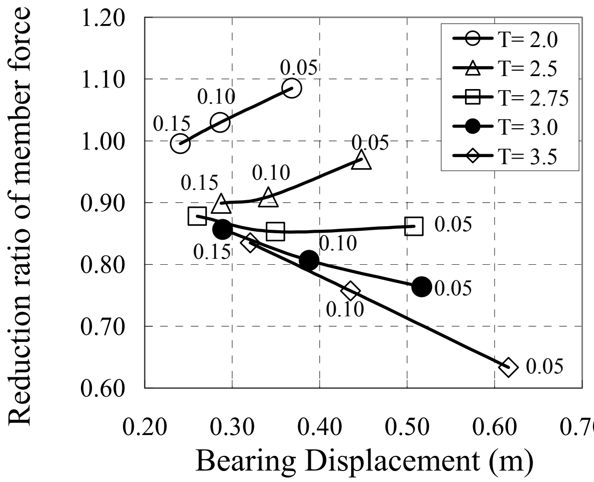



**Figure-6** Change in Responses (μ=0.05, 0.10 and 0.15)

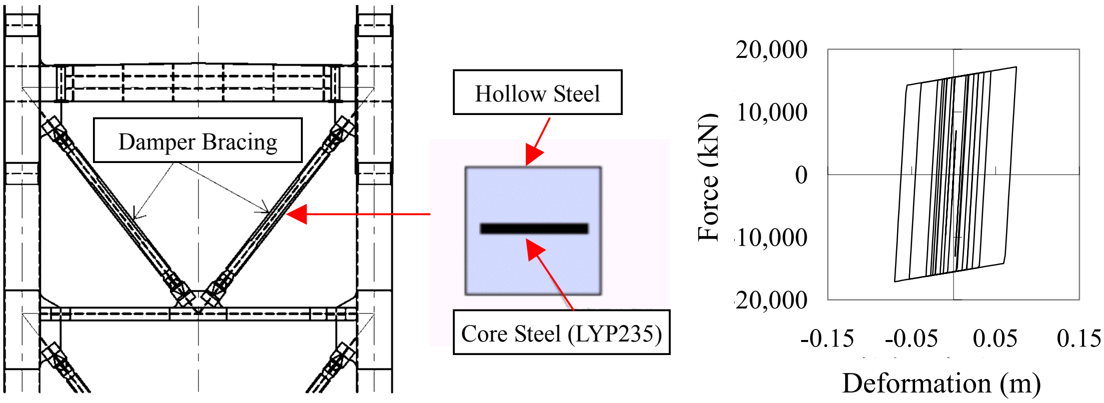



**Figure-7** Steel Damper bracing and hysteretic loops

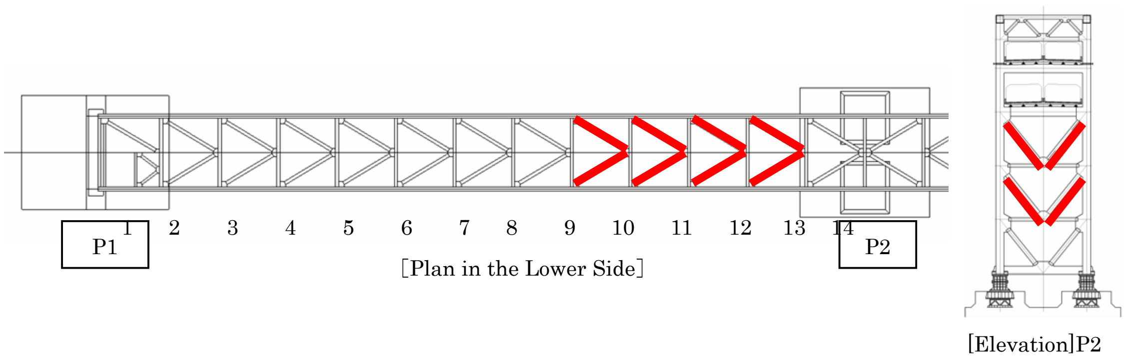



**Figure-8** Optimal Layout of damper bracings

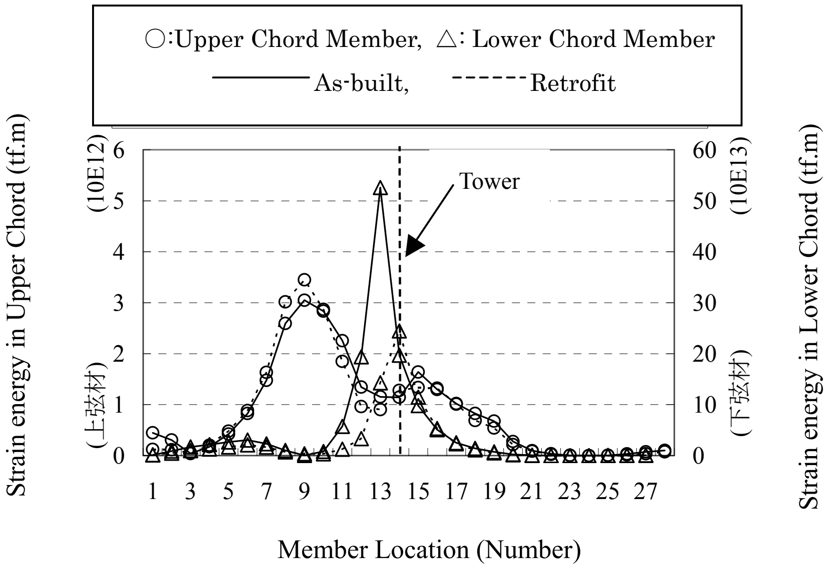



**Figure-9** Effect of Lateral Damper Bracings on the Lower

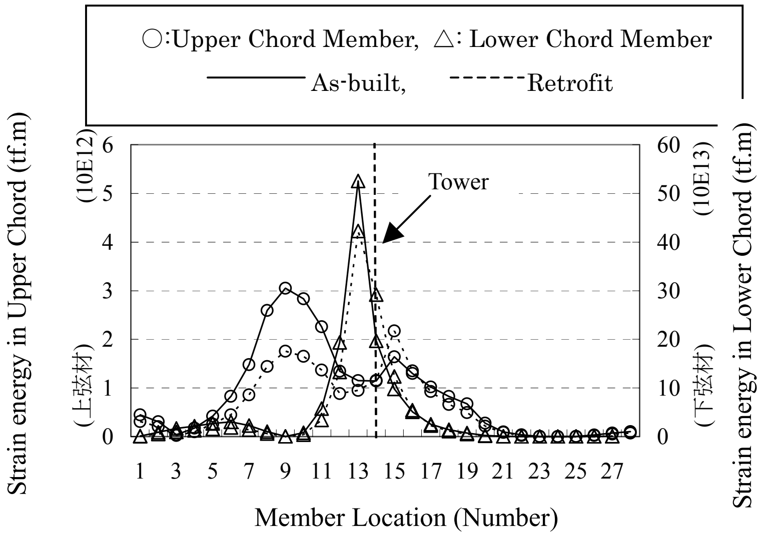



**Figure-10** Effect of Sway Damper Bracings at the Tower

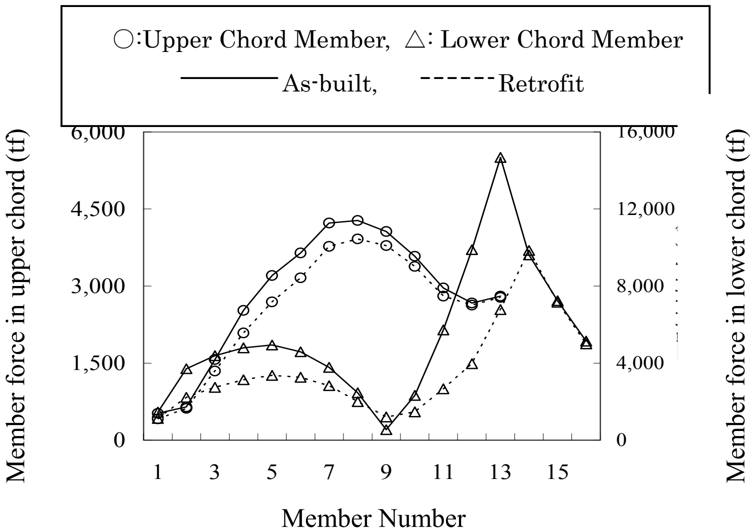
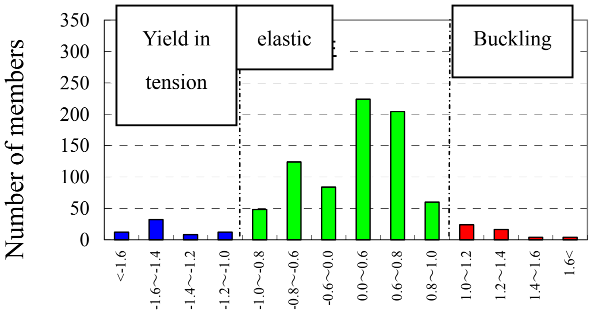
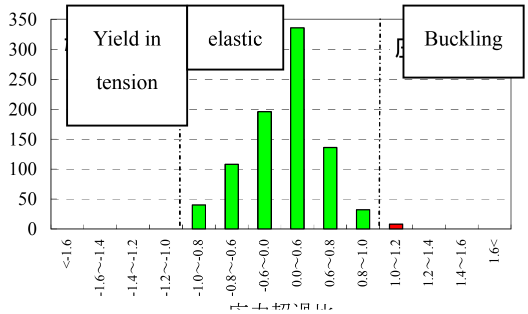



**Figure-11** Effect of the Combined Damper Bracings



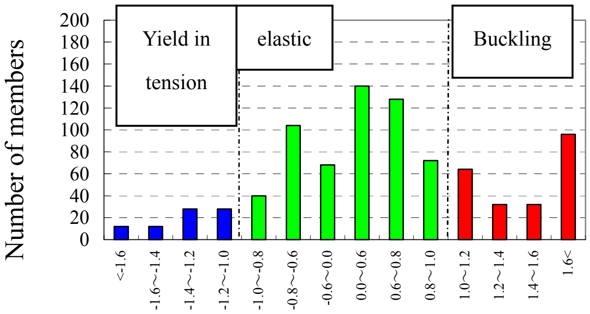

Overstress ratio: R

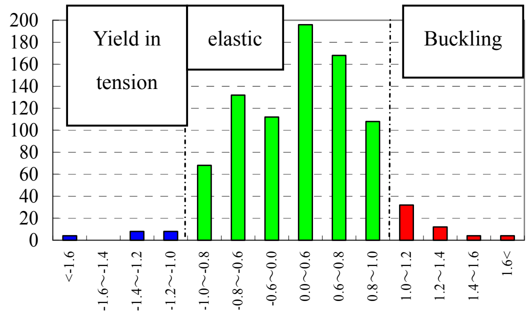
Overstress ratio: R

a) As-built

b)Retrofit

(1) Longitudinal Direction




Overstress ratio: R

Overstress ratio: R

a) As-built

b)Retrofit

(2) Transverse Direction

**Figure-12** Effect in Overstress Ratio