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Seismic reinforcement design and construction of steel strutted beam bridge

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ABSTRACT: The Taura Daini Viaduct was constructed as a steel strutted beam bridge. It has been about 35 years since it opened for use. The seismic performance of this viaduct was subjected to verification according to the Specifications for Highway Bridges (2012). The results indicated that bending and axial force of the strutted beam pier and the superstructure exceeded the bearing and axial capacity, and that the bearing capacity and movement and girder gap space were insufficient. Therefore, it was decided that seismic reinforcement would be conducted for this viaduct. For the seismic reinforcement design of the viaduct, measures to elongate the natural period of the viaduct were reviewed, including the use of buckling restraint braces or replacement of conventional bearings such as pin bearings and pin roller bearings with base-isolated rubber bearings. This paper reports the reviews made about design and construction of the viaduct.

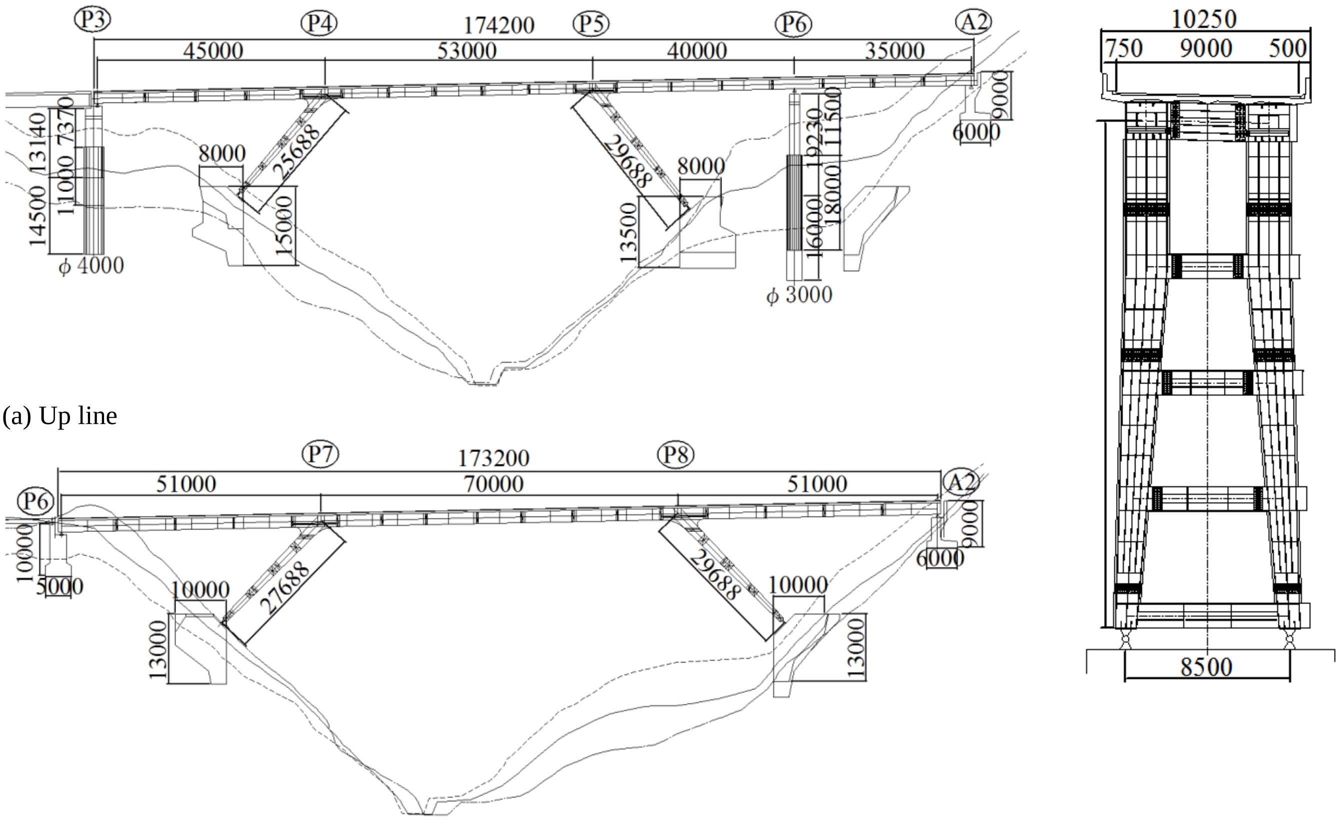
1 INTRODUCTION

The Yokohama–Yokosuka Road is a local toll road with a total length of about 37 km and four lanes. It starts in Yokohama city and crosses the Miura Peninsula north to south. The Taura Daini Viaduct is located in a mountainous area between the Zushi Interchange and the Yokosuka Interchange of the Yokohama–Yokosuka Road. The bridge spans a valley from one ridge to another ridge. The valley is very steep, and the height from the road surface to the valley bottom is about 60 m. When a bridge is to be built over a valley, a steel strutted beam bridge is generally used as the structural type of bridge because of the special conditions required for this type of bridge, such as high bridge piers and the need for a very long span when piers are positioned avoiding the valley. This viaduct is 174.2 m long for the up line and 173.2 m long for the down line (Figure 1). The pier part of this steel strutted beam bridge has a steel double main box girder structure 26 m to 30 m in length and lateral struts at intervals of 5 to 7 m.

Expressways serve as emergency transport routes at the time of a major earthquake, and thus need to allow emergency vehicles to pass at an early stage. Therefore, expressway bridges are required to ensure safety against bridge unseating and maintain seismic performance sufficient to allow swift restoration of their service with only temporary repairs in the event of a major earthquake. For bridges with an ordinary structure type such as girder bridges, reinforcement of the piers and replacement of bearings can often provide the bridge with sufficient seismic performance. However, special types of bridges such as cable-stayed bridges, suspension bridges, arch bridges, truss bridges, or strutted beam bridges require review with regard to the necessity for seismic reinforcement of the superstructure in addition to piers and bearings as they show complicated behavior at the time of an earthquake.

The Taura Daini Viaduct was designed and constructed according to the 1973 version of the Specifications for Highway Bridges for its substructure and the 1980 version for its superstructure. Since the construction of this viaduct, the seismic code in the Specifications for Highway Bridges has been revised with every major earthquake, including the 1995 Great Hanshin Earthquake, according to lessons learned from the damage that occurred. The viaduct was verified based on the updated seismic code, and it was revealed that its substructure and superstructure failed to satisfy the required seismic performance.

Therefore, the necessary seismic retrofitting of the viaduct was conducted, including seismic reinforcement design according to the latest seismic code, replacement of bearings with base-isolated bearings and installation of buckling restraint braces (BRBs) to elongate the natural period of the bridge, provision of a structural measure against negative reaction at the strutted beam (SB) pier, and installation of a bridge unseating prevention device.



(a) Up line

(b) Down line

(1) Side view

(2) Front view of the strutted beam pier (up line P5)

Figure 1. General view of the Taura Daini Viaduct.

Dynamic analysis was conducted with material non-linearity and geometric non-linearity taken into consideration to review the reinforcement policy prior to designing these seismic reinforcement measures.

This paper reports the details of the reviews on the seismic reinforcement design of the strutted beam bridge of the Taura Daini Viaduct and its construction work.

2 SEISMIC REINFORCEMENT DESIGN

2.1 *Target Seismic Performance and Seismic Performance Verification Method*

According to the updated version of *the Specifications for Highway Bridges* (Japan Road Association 2012) at the time of seismic reinforcement design of the Taura Daini Viaduct, the seismic performance required of a highway bridge was as specified in Table 1 and Table 2. Since the viaduct is required to satisfy the level of seismic performance for an expressway bridge, it is stipulated that the limit state of members be determined so as to ensure that plastic deformation would only occur to members for which plasticization is considered in the event of Level 2 seismic motion, and that damage to these members would be to a degree that would allow them to be easily repaired. Level 2 seismic motion indicates two types of seismic motion: one from a large-scale plate-type earthquake, and the other from an inland earthquake such as the Great Hanshin Earthquake. Plasticization in an existing pier should only be allowed in RC piers, for which restoration is relatively easy. For steel strutted beam piers, for which restoration is expected to be difficult, the aim should be to keep the seismic response within the elastic range. The state of limit where chemical characteristics will not exceed the elastic range should be determined for existing steel bearings, abutments, footings, foundations, and superstructure members, such as main girders, cross beams, strutted beam piers, and lateral struts. The state of limit that can ensure energy absorption should be set for base-isolated bearings and damping devices to be newly installed in seismic reinforcement design.

To verify the current seismic performance for the entire bridge system in the event of a major earthquake, nonlinear dynamic analysis using the entire system model for the strutted beam rigid-frame part was conducted. The analysis model used was a solid framework model of a multi-mass system, which is the entire bridge system modeled with beam elements and spring elements. Members where damage is expected to occur were modeled by considering the material nonlinearity. Since this is a large-scale steel strutted beam bridge, analysis was conducted taking into account geometric nonlinearity, with the impacts of deformation due to an earthquake taken into consideration.

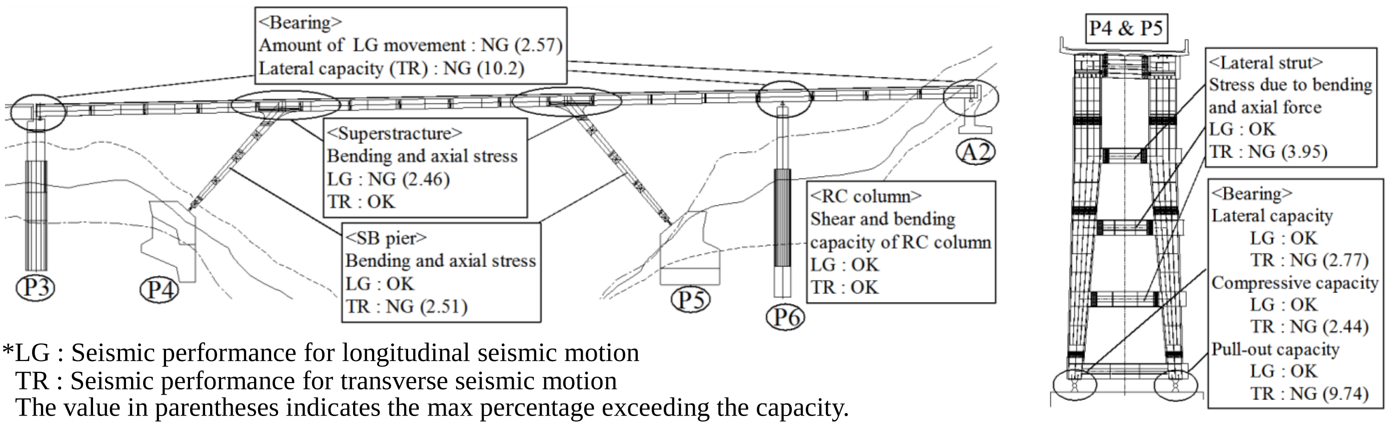
Table 1. Viewpoints of seismic performance.

Seismic performance of bridge	Safety in terms of seismic design	Serviceability in terms of seismic design	Repairability in terms of seismic design	
			Short-term repairability	Long-term repairability
Seismic performance 1: Performance that can prevent loss of bridge soundness due to an earthquake	Ensure safety against bridge unseating	Ensure the functions of the bridge as before the occurrence of an earthquake	No repair necessary to restore functions	Only minor repair required
Seismic performance 2: Performance that can limit the damage due to an earthquake to a limited area and allows the intended functions of the bridge to be swiftly restored	Ensure safety against bridge unseating	Functions as a bridge can be swiftly restored after the occurrence of an earthquake	Functional restoration can be handled with temporary repairs	Permanent restoration can be relatively easily realized
Seismic performance 3: Performance that prevents damage due to an earthquake from causing fatal damage to the bridge	Ensure safety against bridge unseating	-	-	-

Table 2. Design seismic motion and target bridge seismic performance.

Design seismic motion		Type A bridge	Type B bridge
Level 1 seismic motion		Performance that can prevent loss of bridge soundness due to an earthquake (seismic performance 1)	
Level 2 seismic motion	Type I seismic motion (plate boundary type major earthquake) Type II seismic motion (inland earthquake like the Great Hanshin Earthquake)	Performance that prevents damage due to an earthquake from causing fatal damage to the bridge (seismic performance 3)	Performance that can limit the damage due to an earthquake to a limited area and allows the intended functions of the bridge to be swiftly restored (seismic performance 2)

* Type A bridges are defined as bridges other than Type B bridges. Type B bridges are the following kinds of bridges:
• Bridges of national expressways, urban expressways, designated city expressways, Honshu–Shikoku connecting roads, and ordinary national highways
• Of prefectural and municipal roads, multi-formation roads, bridges over railroads, bridges over roadways, or other bridges that are especially important from the viewpoint of their positioning in local disaster management plans or their use conditions



*LG : Seismic performance for longitudinal seismic motion
TR : Seismic performance for transverse seismic motion
The value in parentheses indicates the max percentage exceeding the capacity.

Figure 2. Outline of the seismic performance verification results (Up line).

2.2 *Results of Seismic Performance Verification*

Figure 2 shows the outline of the results of seismic performance verification conducted for the entire bridge system of the Taura Daini Viaduct, including its substructure, against Level 2 seismic motion as specified by the Specifications for Highway Bridges in conjunction with seismic reinforcement design for the viaduct.

Verification of seismic performance of the viaduct against seismic motion in the bridge axial direction revealed that the major girders were slightly plasticized, the movement of the bearings exceeded the design level, and the girder gap space was insufficient. For seismic motion in the direction normal to the bridge axis, it was revealed that SB piers were plasticized and that the pushing or pulling force that occurs to bearings at the base of the SB piers exceeded the bearing capacity. In addition, the results showed that the horizontal force acting on the RC abutment and pier base exceeded the bearing capacity and that the major girders of the up line and down line collided.

2.3 *Seismic Reinforcement Policy*

Two design policy proposals are described below.

2.3.1 *Proposal of reinforcement by base isolation and improvement of the pier's bearing capacity* (Plan A)

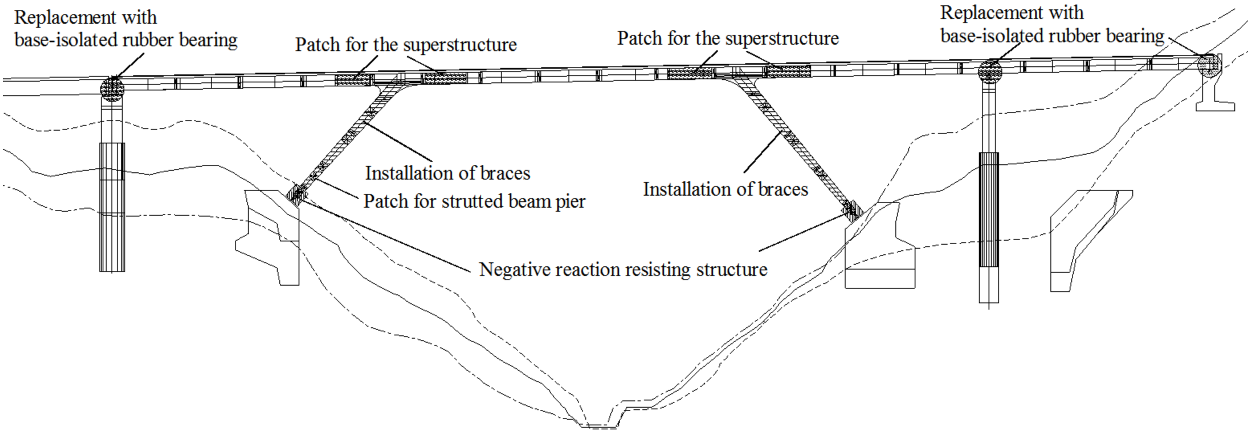
When a large special bridge such as the Taura Daini Viaduct is seismically reinforced, conventional reinforcement measures generally include installation of base isolators or dampers to control seismic responses and reinforcement of members to improve member bearing capacity. The basic reinforcement plan initially developed for this viaduct considered replacement of existing bearings with base-isolated bearings as a specific measure to control seismic responses in the bridge axial direction, and the installation of damping devices to control displacement. However, since the existing bearings did not comply with the current standards, and the horizontal force in the direction normal to the bridge axis would exceed the bearing capacity in the event of a major earthquake, replacement with base-isolated bearings was ultimately adopted as the basic policy. As shown in Figure 3, the policy adopted here was to replace the existing steel bearings used in the RC piers and abutments with base-isolated bearings to elongate the natural period of the viaduct and enhance the attenuation effect of the bearings so as to ultimately reduce the response of the entire bridge. Furthermore, it was decided to enhance the bearing capacity of members by conducting reinforcement of patches for strutted beam piers, installation of braces between studs, and reinforcement of patches for the superstructure.

The natural period of the bridge was relatively small in the direction normal to the bridge axis, and that increased the response acceleration of the entire bridge, which was a primary factor in not satisfying the verification. For the natural period, it was presumed that the influencing factor was the high stiffness of the entire pier because of rigid connection of the lateral struts, each having a box-shaped section, against two strutted beam piers. In addition, high stiffness of the pier was presumed to make the rocking behavior of the SB piers predominant and eventually increase the bearing response at the base of the SB piers. For pulling in particular, the pulling force about7 times the dead load(Up line P4) works on the bearings and conspicuously exceeds the bearing at the cap fixing the upper and lower bearings and anchor part.

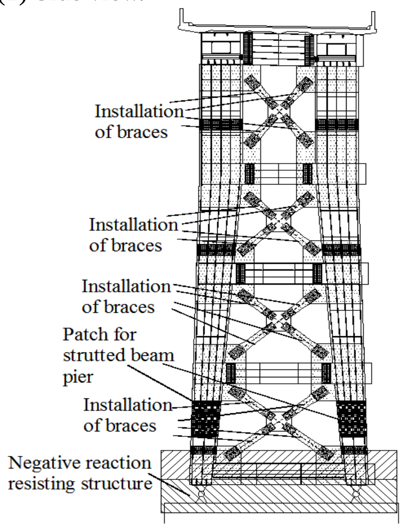
For reinforcement against the negative reaction occurring at pivot bearings at the base of the SB piers, the method for such reinforcement had to be one that does not affect the present structural system, and it was necessary to review an appropriate method that can support large pulling force and follow the rotational movement in the bridge axial direction. Furthermore, it was decided that a method that requires no bearing replacement would be reviewed, because a bearing on which large axial force constantly acts, such as one at an SB pier base of a strutted beam bridge, requires a large temporary structure for bearing replacement, as well as the facts that safe execution of bearing replacement while keeping the road in service presents many difficulties and bearings that should replace those at the base need to be very large.

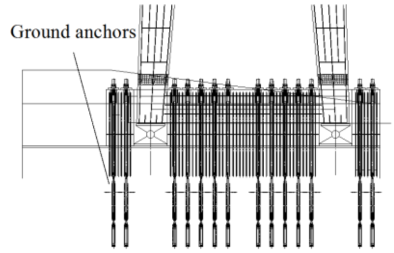
Therefore, as shown in Figure 3 (3), we studied a structural solution that would provide reinforced concrete lining to the SB pier base, improve the bearing capacity of bearings by replacing bearings with Mesnager hinges, and resist negative reaction by installing ground anchors that penetrate the abutment from the RC lining to the ground. However, if the pier base became rigid in the rotational direction, it could further increase the response of the SB pier base including the pier itself. In addition, it was revealed that it was necessary to secure some degree of space in the interval between ground anchors from the impact of borehole bending due to drilling error, or loosening of the ground or stress release by adjacent boreholes. Conversely, in a case in which the ground anchor interval needs to be decreased, the borehole drilling length in the ground needs to be very long to allow installation of anchorages with greater displacement in the depth direction. Furthermore, the drilling depth in the abutment is about 4 m, and reinforcing bars are densely arranged there. Therefore, it was considered difficult to drill boreholes while avoiding the reinforcing bars. In addition, when 10% of the pulling force acting on the bearings is introduced to ground anchors as the initial prestressing force, it will work like a force that pushes the bearings with a load as great as 75% of the dead load reaction, which could therefore negatively affect the ordinary rotational behavior.

From the above, it was found to be difficult to realize reinforcement by providing Mesnager hinges to the base of SB piers with RC lining and by installing ground anchors. It was considered necessary to reduce the stiffness of the bridge piers to ensure reasonable measures for the direction normal to the bridge axis. On the other hand, since it is necessary to ensure the existing performance in normal situations while reducing stiffness, it was necessary to review a structural solution that could satisfy both requirements.

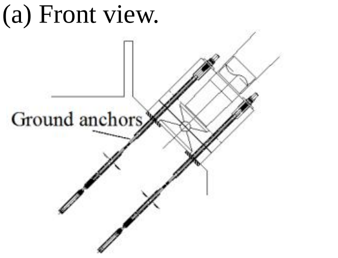


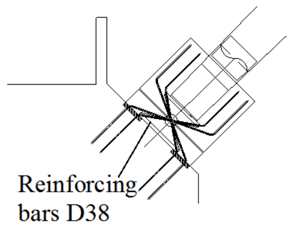
(1) Side view.





(a) Front view.

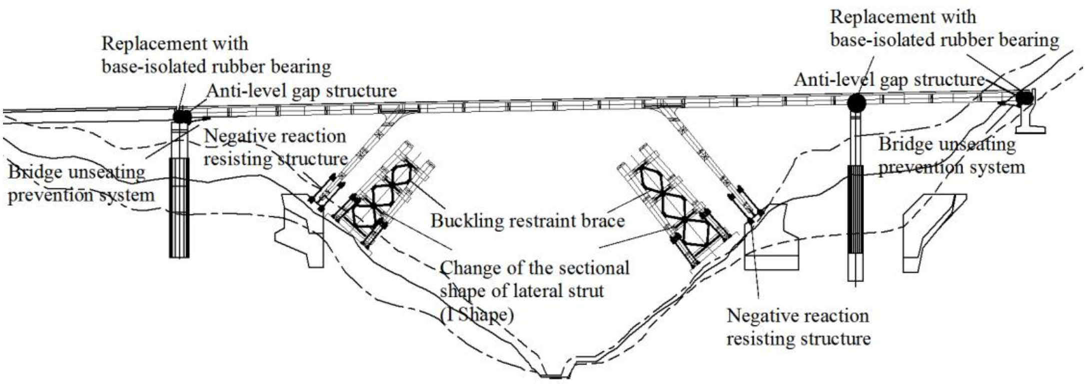




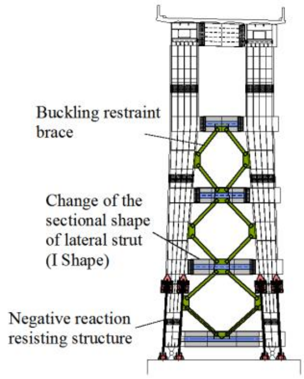
(b) Side view (Ground anchor). (c) Side view (Mesnager Hinge).

(2) Frond view of the strutted beam pier. (3) Negative reaction at the strutted beam pier base.

Figure 3. General view of reinforcement by base isolation and improvement of pier bearing capacity.



(1) Side view



(2) Frond view of the strutted beam pier

Figure 4. Proposed policy for reinforcement for the purpose of elongating the natural period by base isolation and reduction in pier stiffness

2.3.2 *Proposed policy for reinforcement for the purpose of elongating the natural period by base isolation and reduction in pier stiffness (Plan B)*

Based on the above reinforcement policy, it was decided that the top priority would be put on reducing reinforcement of the SB pier base by reducing the negative reaction working on the base.

Therefore, it was decided to change the sectional shape of the lateral struts from the existing box shape to an I shape, to install BRBs among the struts, and to reduce the response to below (1) under Level 2 seismic motion by elongating the natural period of the viaduct and making use of the attenuation effect of the braces (Figure5). It was revealed that use of this reinforcement policy would reduce the rocking behavior of the SB piers and reduce the bearing reaction at the base of the piers, and ultimately could reduce the negative reaction resisting structure compared with the initial plan (Figure 4).

2.4 *Results of the Seismic Reinforcement Review*

According to the review based on the seismic reinforcement policy discussed in 2-3, it was decided to use the following major seismic reinforcement work: application of carbon fiber lining to RC piers and replacement of pin bearings and pin roller bearings with base-isolated rubber bearings against seismic motion in the bridge axial direction and the direction normal to the bridge axis; and replacement of the sectional shape of the lateral struts from a box shape to an I shape, installation of BRBs, and installation of a negative reaction resistant structure at the SB pier base against seismic motion in the direction normal to the bridge axis.

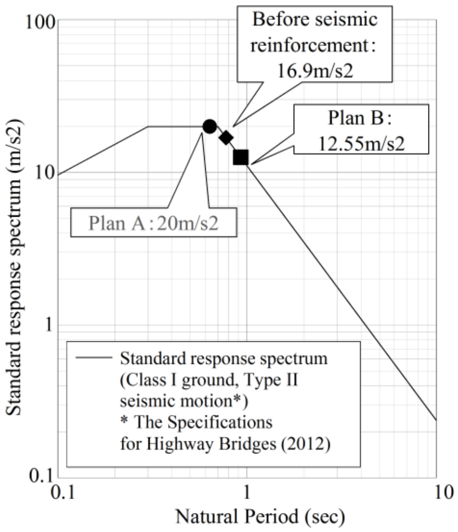


Figure 5. Standard response spectrum (Up line).

The above work was adopted as reinforcement work for its greater excellence in structural performance, constructability, and economic efficiency compared to a case in which the methods mentioned in 2-3(1) are used. Rough estimation indicated that the construction cost would be about 20 to 30% less than the other plan.

2.4.1 *Change of the sectional shape of lateral struts and installation of BRBs*

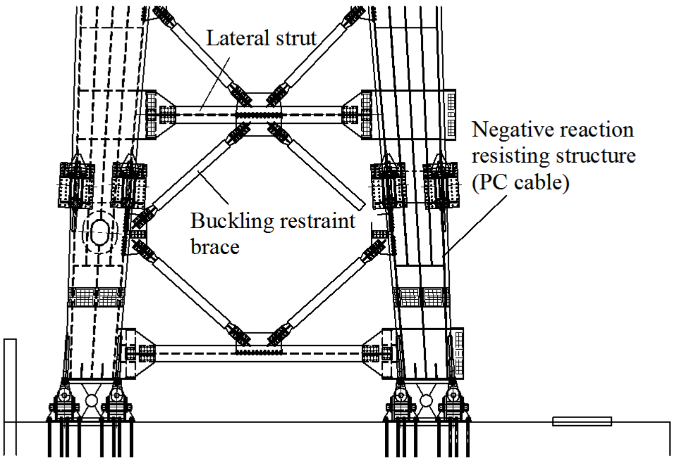
For the direction normal to the bridge axis, replacement of the sectional shape of lateral struts from a rigid box shape to an I shape was conducted to reduce the stiffness of the entire bridge pier. However, stiffness would become too small if this measure alone were taken, which would not satisfy verification for normal times and Level 1 seismic motion. Therefore, it was decided that BRBs would be added as diagonals to compensate for the lack of stiffness. BRBs should be designed to have an elastic response to Level 1 seismic motion and cause plasticization to reduce the stiffness of the bridge piers and absorb energy against Level 2 seismic motion. In other words, to change the resistance characteristics of bridge piers according to the loading case using BRBs, lateral struts would be replaced with less rigid members. The idea of installing BRBs while maintaining the original sectional shape, or box shape, of lateral struts was considered, but this measure would result in smaller relative deformation inside the pier and only show limited benefits.

For the arrangement of BRBs, which will be described in detail in 4-3, safety was reviewed against the type of seismic motion that can frequently occur during construction, and a lozenge arrangement was determined as a result. In addition, BRBs are required not to yield under axial force that occurs in normal times or at the time of a Level 1 earthquake, and plasticize to sufficiently absorb energy against Level 2 seismic motion. Furthermore, in designing the BRBs, the target was to maintain the performance of the braces even in the face of three occurrences of Level 2 seismic motion while considering the impact of aftershocks. In other words, the braces were designed so that the maximum strain that would occur to the braces under Level 2 seismic motion would be no more than the limit strain, and that three times the cumulative plastic strain would be no more than the limit cumulative plastic strain. In light of the above results, it was decided that double steel pipe dampers would be installed in the up line and down line as BRBs of 1100 kN and 1500 kN, respectively.

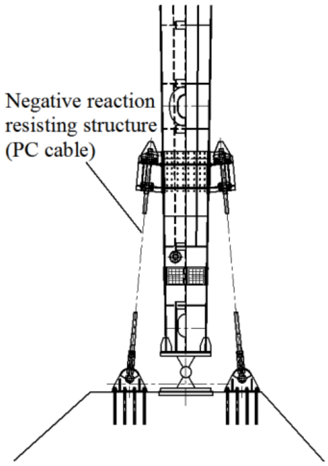
The existing lateral struts were connected with high-strength bolts to joints integrated with bridge piers. In installing lateral struts after replacement, since it was necessary to avoid changes to the existing piers, such as drilling, as much as possible, it was decided that the existing lateral struts would be removed with the bolt connection as the boundary. The remaining existing joints and bolt holes would then be effectively used for newly installed lateral struts. While the newly installed lateral struts would have an I shape, the existing joints were box-shaped, and how to appropriately connect them was a problem. As a solution, cross-shaped gusset plates would be attached to the inside of an existing joint, and new lateral struts would be installed via the plates. The arrangement of BRBs was determined from the viewpoint of safety against earthquakes during construction as described later.

2.4.2 *Negative reaction resisting structure at the SB pier base*

Taking the above measure in (1) greatly reduced the bearing response at the SB base pier during an earthquake, and successfully satisfied verification of the bearing against the pushing force. However, with regard to the pulling force, the design load of the pivot bearings, which were existing bearings, was small, and it was impossible to avoid exceeding the bearing capacity of the bearings.



(1) Front view.



(2) Plan view.

Figure 6. Negative reaction resisting structure.

It was thus decided to set PC cables at the base of each pier as a structure that complements the bearing fixing function against pulling force. When measures other than this were adopted, it was impossible to greatly reduce the pulling force, and the cable structure could not allow the necessary number of cables. As an alternative, it was conceivable to arrange ground anchors through the abutment, but this would require an increased number of anchors, and it would be very difficult to place such anchors on the abutment.

In light of the rocking behavior mentioned above, PC cables meant to resist the pulling force acting on the SB pier base were designed using the maximum value of the pulling force acting on each pier, instead of the total value of one support line. The design did not intend to have PC cables resist pulling force jointly with the bearing proper, but aimed to have the cables resist the force alone. Since four cables were required per pier, they were arranged symmetrically at a column web position that easily allows transmission of stress to the pier column as shown in Figure 5. Redundancy against earthquakes of unexpected scale was allowed for by designing anchor parts using the fracture strength of the PC cables, and consideration was made so that anchor parts would not cause brittle failure due to cone-shaped failure or other causes.

2.4.3 *Bridge unseating prevention system*

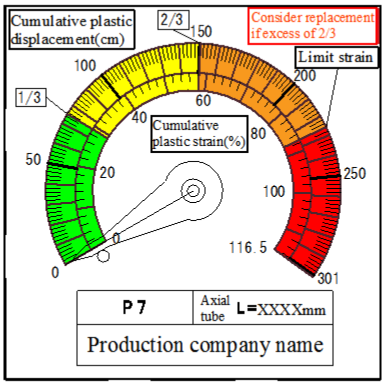
For the bridge unseating prevention system, the necessity of each structure was judged according to the Specifications for Highway Bridges (2012). The judgment result indicated that the girder seating length and bridge unseating prevention structure would be necessary for the viaduct. According to the Specifications, a bearing structure can be omitted for a "superstructure to which two or more substructures are rigidly connected." However, it was judged that omission was not possible for this viaduct as its SB pier base is a bearing structure. A PC cable structure was used for the bridge unseating prevention structure, and steel protrusions were used as a gap prevention structure. It was necessary to install various brackets associated with these members, but the reuse of existing brackets would minimize the reinforcement structure, and the bracket arrangement was designed so as not to obstruct future maintenance.

Expressways serve as emergency transport routes in the event of a major earthquake and thus need to allow emergency vehicles to pass at an early stage. If a bearing with a great bearing height is damaged by an earthquake and becomes deviated, it will create a large gap at the expansion device, and this can make it difficult for the road to be quickly passable to emergency vehicles. Therefore, it is effective to install a structure that prevents the creation of road level gaps. Considering that this viaduct is an expressway bridge and its bearing height is about 600 mm, it was decided that an anti-level gap structure would be installed at the end support.

3 CONSIDERATIONS FOR MAINTENANCE

It is necessary to appropriately understand the damage condition of BRBs during post-earthquake inspections to judge if the road is available for emergency vehicles or if BRBs need to be replaced. Therefore, the inspection method needs to be easily made for a quick response after an earthquake. For BRBs, it is specified that the ultimate cumulative plastic strain be three times the cumulative plastic strain that occurs at the time of Level 2 seismic motion. Therefore, it is necessary to check the residual performance after an earthquake with maximum displacement and cumulative plastic displacement. In this respect, devices to measure cumulative plastic deformation were installed at representative locations of the Taura Daini Viaduct, as shown in Figure 7, to allow easy judgment on the need for replacement after an earthquake. The cumulative plastic displacement gauges that were installed require no electric power and are designed to compare plastic deformation assumed in the dynamic analysis of the seismic reinforcement design with plastic deformation that is actually caused by an earthquake.

(1)Installation situation.



(2) Scale plate.

Figure 7. Cumulative plastic strain gauge.

Figure 8. Status of inspection route before seismic reinforcement work.

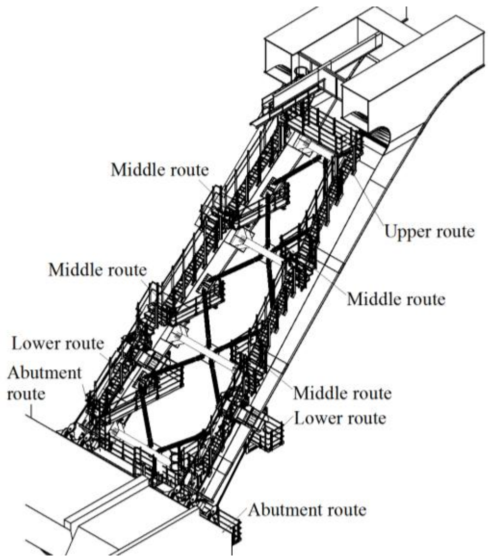


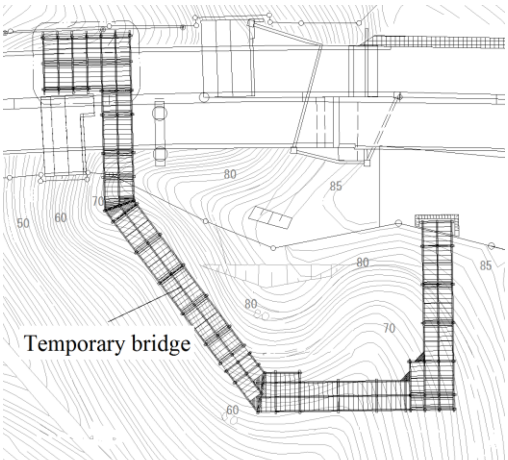
Figure 9. Planning of new inspection route.

An inspection gallery between main girders of the superstructure and steps for inspection of the exterior and interior surfaces of the SB piers were provided in the Taura Daini Viaduct (Figure 8). Since it was necessary to reach the top of each SB pier, with an inclination from 40° to 45°, by ascending and descending steps only, to inspect a length of about 25 m, a safer and more efficient inspection route was studied. Requirements for the review of the inspection route included the availability of inspection traffic between the SB piers and the superstructure and accessibility to the negative reaction resisting structure, which needs to be inspected in the event of an earthquake, the gusset part of BSBs, and the cumulative plastic displacement gauges. An appropriate inspection route layout plan that satisfies these requirements was developed. Only steps were originally provided to access the top surface of the SB piers, but an inspection route structure that ensures safe inspection with handrails, stairs, and walkways was developed (Figure 9).

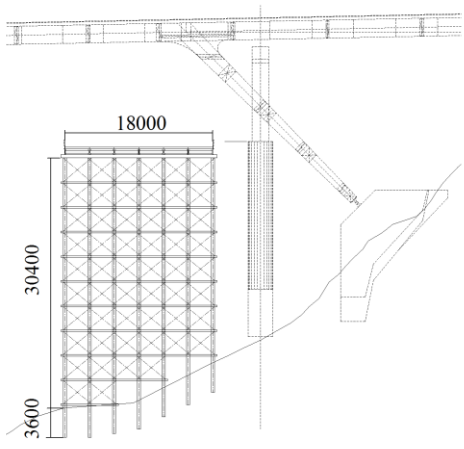
4 SEISMIC REINFORCEMENT WORK

4.1 *Overall Work Plan*

From the viewpoint of ensuring the convenience of expressway users, execution of the seismic reinforcement work was required to allow loading in and out of materials and equipment and access by workers to the work site with only minimal application of traffic control to the expressway for the purpose of work. However, the Taura Daini Viaduct is located over a steep valley, and there was no road under the viaduct. Therefore, it was an urgent issue to construct a construction road for access by work vehicles transporting materials and equipment and for workers entering and leaving the site.



(1) Plan view.



(2) Side view.

Figure 10. Temporary bridge for work.

The solution was to build a construction road accessible from the main line of the expressway. To set the starting point, an access road (about 230 m in length) constructed by cutting the ground during construction of the viaduct was repaired and reused. A temporary abutment, about 30 m in maximum ground height, was constructed on the slope (Figure 10). Construction of this road enabled traveling of work vehicles to the work yard with only simple traffic control for the road shoulder, not with lane traffic control.

4.2 *Bearing Replacement Work*

To remove the existing bearings, preparatory work was conducted including girder reinforcement for jacking up, installation of temporary receiving brackets, and installation of temporary fixing brackets against earthquake situations.

When anchor bolts are newly installed as bearings, the general procedure is to reuse some existing bearings or existing bearing anchor bolts, and drill holes for fixation with anchor bolts after removal of the existing bearings. However, regarding the former, it was found that the soundness of anchor bolts could be damaged after the removal of existing bearings. Regarding the latter, holes for anchoring could not be drilled at predetermined locations because of interference with existing reinforcing bars, which would require correction of hole positions on the plates attached to the substructure and elongate the period of jacking up. In this work, as it was to be conducted under a road in service, it was decided that holes for anchor bolts for new bearings would be drilled before removal of existing bearings in order to shorten the jacking-up period during the work.

This bearing replacement was work to be conducted under a road in service. Therefore, the upper limit of jacking up was set to 3 mm to minimize the gap on the road at the expansion device part, and bearings were cut and removed after jacking up. When bearings are installed during construction of a bridge, a crane is generally used to install the bearings. In this work, bearings were carefully installed using a jack device with a claw or chain block, as the replacement work was to be conducted under the superstructure girders and the work space was small.

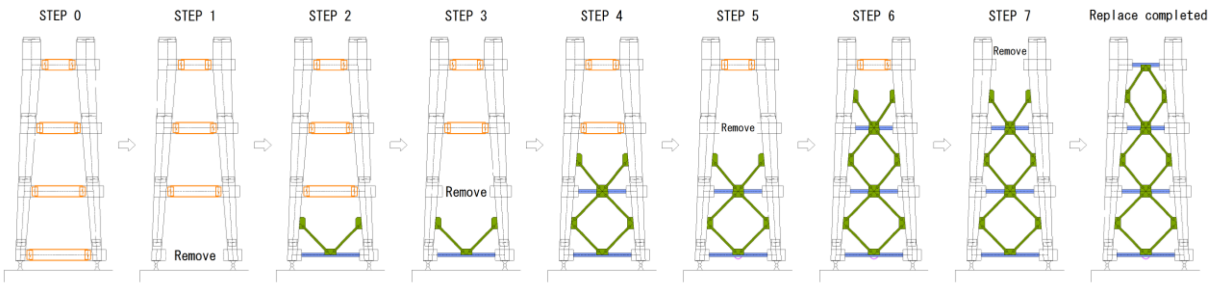
4.3 *Replacement of Lateral Struts*

Temporary removal of lateral struts was necessary to conduct work under the road in service, so the structural safety against normal times and Level 1 seismic motion was verified, and a procedure that would reduce the impact on the bridge under Level 2 seismic motion was studied. The two work procedures reviewed were to replace bearings from down to up (Plan 1) and from up to down (Plan 2) (Figure 11).

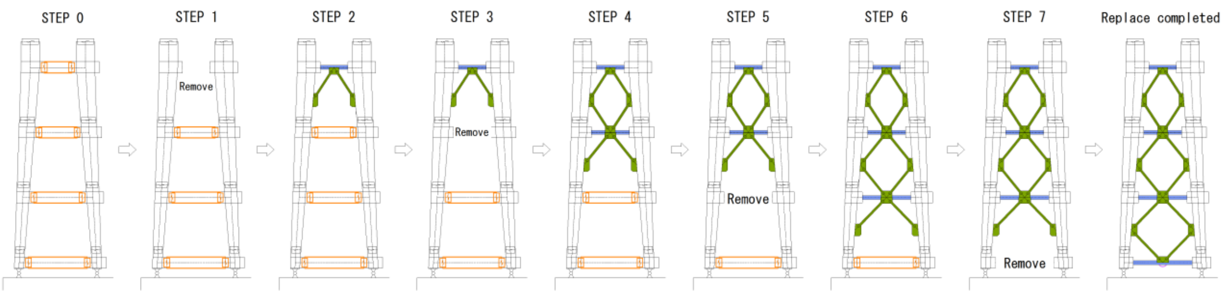
For normal times and Level 1 seismic motion, static framework analysis was conducted in each step of the procedure, as in the previous section. It was confirmed that both plans satisfied the verification. The allowable value increment during work was not considered, as the work was to be done with the road kept in service.

For Level 2 seismic motion, time series response analysis was conducted for steps in which the bending moment was predominant in the analysis for normal times and Level 1 seismic motion. As a result, the response increase ratio increased at the lower end side of the pier for Plan 1 and at the upper end side of the pier for Plan 2. The response at the lower end of the pier exceeded the allowable value for Plan 1, while it was within the allowable value for Plan 2. Based on this result, it was decided that lateral struts would be replaced from up to down.

The four arrangements of BRBs, or ◇pattern, × pattern, V pattern, and inverted V pattern, were reviewed (Figure 12). For × and inverted V patterns, brace panel points were set at the SB pier base.



(1) Replace down to up (Plan1).



(1) Replace up to down (Plan2).

Figure 11. Step diagram of replacement of lateral struts.

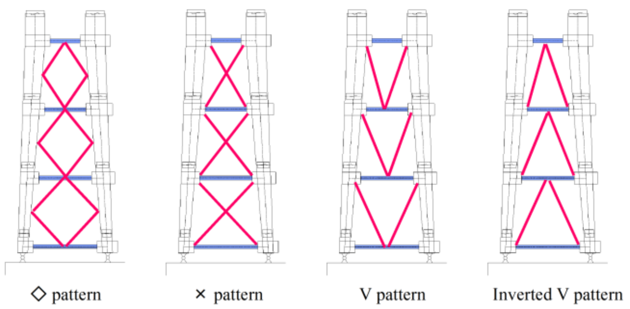


Figure 12. Arrangement of BRBs.

However, since reinforcing members are densely arranged above the bearings within a sectional width of about 1 m at the pier base and there is only a small space there, it was judged that reinforcement would be difficult. Thus, these two patterns were not adopted. For ◇and V patterns, the response properties against normal times and Level 1 seismic motion were compared for the structural system expected to occur during work. For the V pattern, the fixed interval of pier columns increased due to the lateral framing. As a result, verification upon completion could not be satisfied, so this pattern was not adopted. Based on the above, the ◇pattern was adopted for the arrangement of BRBs.

5 CONCLUSIONS

This paper describes reviews made with regard to seismic reinforcement design for the Taura Daini Viaduct, which is a steel strutted beam bridge, as well as considerations about future maintainability and measures taken for reinforcement work. When an existing bridge like this viaduct is provided with seismic reinforcement, the design should be developed by fully considering safety for work being conducted while the bridge is kept in service and open to traffic. Conditions to be considered for repair and reinforcement work vary from bridge to bridge. The contents discussed in this paper may only be applicable to a limited number of cases, but the authors will be pleased if it serves as a reference for future seismic reinforcement of special bridges.

REFERENCES

Japan Road Association. 2012. The Specifications for Highway Bridges.