SEISMIC PERFORMANCE IMPROVEMENT OF TOYOSATO BRIDGE

Miyuki HIRANO [1)]

ABSTRACT

Public Works Bureau of Osaka City has been conducting examination of seismic pier reinforcement for bridges over the Yodo River that flows northern part of Osaka. However, it revealed problem that pushes up the cost for temporal works. In this relation, our examination was implemented by setting target on Toyosato Bridge to improve the total seismic performance of whole bridge system aiming at reduction of seismic force by lengthening of the natural period. By applying the selected seismic performance improvement, reinforcement of almost piers becomes not necessary and minimizing of reinforcing scale was enabled.

1. BACKGROUND OF THE EXAMINATION

Public Works Bureau of Osaka City Government (**Fig.1**) is currently performing maintenance for approximately 760 bridges within the authority area. After the Hyogo-ken Nanbu Earthquake occurred on January 17th 1995, the bureau aggressively promoted installation of unseating prevention structure systems and pier reinforcement to these existing bridges as seismic measures, giving priority to the emergency traffic routes that

Fig.1 Osaka City

1) Staff of Bridges Department, Public Works Division, Public Works Bureau of Osaka City Government

are specified in the disaster prevention plan of Osaka City to secure the traffic routes in the event of earthquake. Especially, the highest priority has been given to elevated bridges crossing over other roads where potential secondary disaster may happen. However, in this moment when the very urgent countermeasures on the spot were completed, it is now planed to expand improvement measures to such places as piers of the existing bridges stand inside river area in the future.

Many of the river bridges on the emergency traffic route are the large sized ones among the objected bridges to be controlled, especially the bridges crossing over the Yodo River that flows northern part of Osaka city area have main span of 50m to 240m (**Fig.2**). The constructed period of these bridges are distributed in variety from 1931 to 1989. However, many among them were constructed in the period before 1965 when bridge seismic designing technology did not indicated sufficient development. Moreover, most of these bridges have such substructures as wall-shaped piers or rigid-frame piers that are wider in transverse direction to the bridge axis and narrower in bridge axis direction,

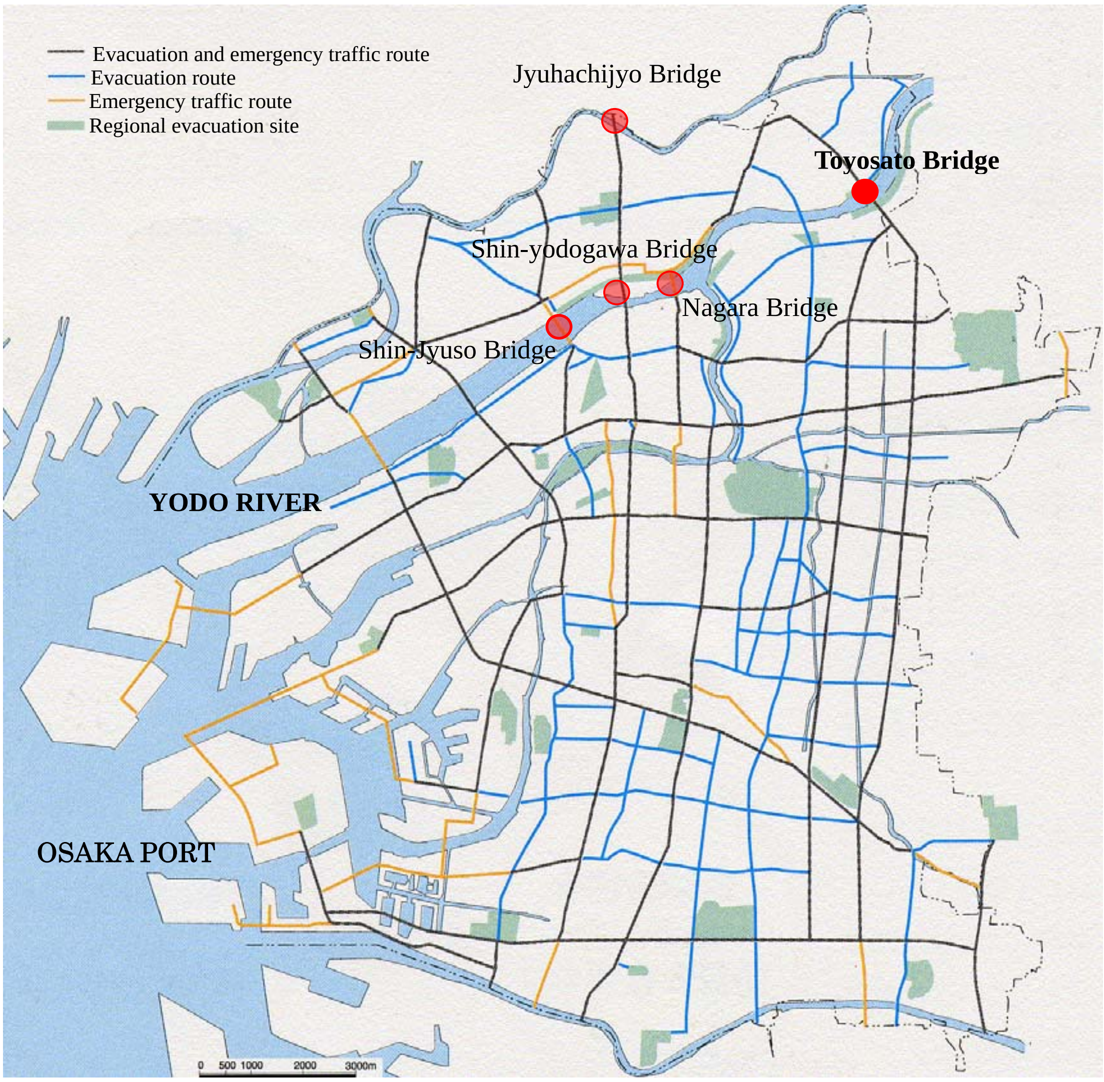


Fig.2 Large scaled river bridges on the emergency traffic routes

though the superstructure of these bridges are indicating varieties of styles such as arch bridges, cable-stayed bridges and girder bridges.

Based on the static analysis (ductility design method), we examined on the reinforcement to such existing river piers by conventional reinforcement such as steel plate jacketing reinforcement or reinforced concrete jacketing reinforcement. This revealed some problems with such reinforcement that makes work harder because restrictions to river conditions and river environment are to be considered and needs additional cost for provisional bridge constructions and temporal flow closure works.

Under such circumstance, the necessity to take more economical and effective measures was highlighted to secure seismic performance of each pier by improving the total seismic performance of the whole bridge system not depending on local improvement measures that aim to increase seismic performance of individual substructure of the bridge.

2. OUTLINES OF TOYOSATO BRIDGE

The first object to be examined among large-scaled bridges crossing over the Yodo River was Toyosato Bridge (**Photo.1**). Reinforcement to this bridge, locating on the emergency traffic route, will lead to achieve a global emergency traffic network that connects inside and outside the city.

The outline of Toyosato Bridge is shown in the **Table 1** below. This bridge constructed in 1970 has the structure configured from the main section of three span continuous cable-stayed bridge and approach bridges with the bridge length of 561.25m. As the style of cable arrangement, the bridge applies single-plane fan type cable system and the base of which is free rotating in the bridge axis direction while fixed in transverse direction to the bridge axis. For the designing of tower it has "A" shaped towers and pin bearing construction between the tower basement and the piers. In consideration of wind stability as well as economical performance the main girder of the bridge applies reverse-trapezoidal steel deck box girder construction showing excellent torsional rigidity

Photo.1 Panorama of Toyosato Bridge

Table 1 Outlines of Toyosato Bridge

Name of Bridge	Toyosato Bridge
Completion year	1970
Name of the river	The Yodo River, 1st classed river
Name of the route	National Road No.479
Classification of the bridge	1st Class bridge (TL-20, TT-43)
Bridge length	561.250m
Superstructure type	Simple I-girder+Three-spen continuous cable-stayed bridges +Two span continuous I-girders
Substructure type	Inverted T-type bridge abutment(P1,P7), Wall-shaped pier (P2,P5,P6), Rigid-frame pier (P3,P4)
Foundation type	Pneumatic caisson foundation
Ground type	Type II ground
Specifications applied	Specifications for steel highway bridges (1964)
Design horizontal coefficient	kh=0.2

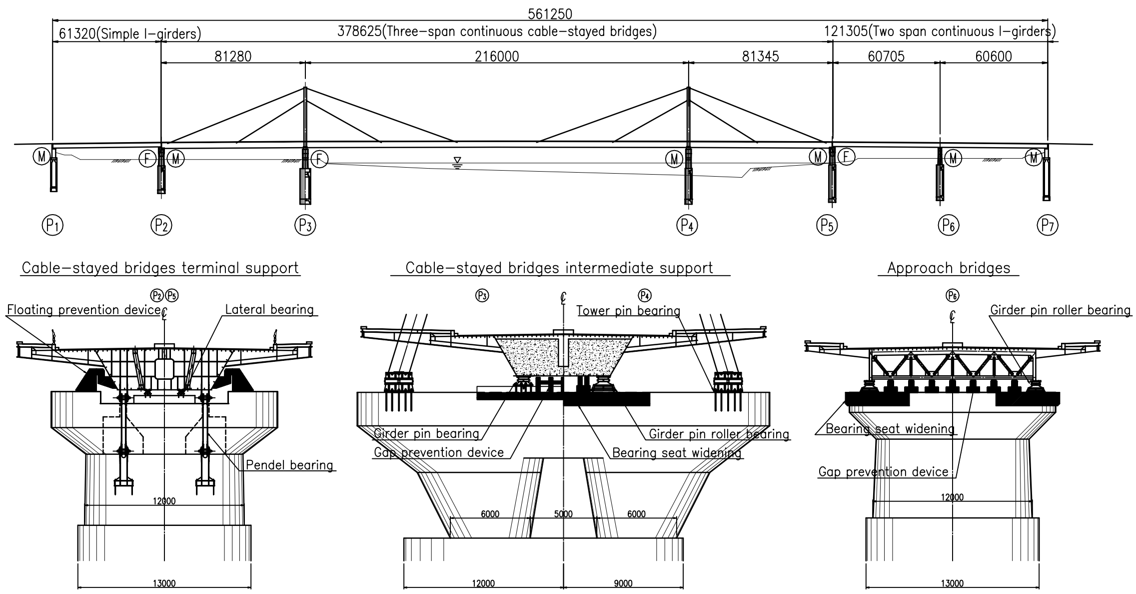


Fig.3 General figure of Toyosato Bridge

(**Fig.3**). All of the foundation structures are supported by the pneumatic caissons laid on the first gravel layer (N value of 50 or more, layer thickness 7m), which is deeper than GL-20m layer. As the existing structure of this cable-stayed bridge, it indicates intermediate support pier P3 as the seismic structure of the pier while the shape of bridge girder adopts rigid-frame type considering self weight reduction. The design of the bridge was made under seismic coefficient method to have performance of horizontal seismic coefficient of 0.2 and vertical seismic coefficient of 0.1.

3. INPUT EARTHQUAKE GROUND MOTION

Based on the assumption that the Uemachi fault systems (Nenbutsu-jisan fault, Uemachi fault, and Nagai fault) lying under the city area become activated synchronizing the earthquake that might give serious effects to the city area, Osaka City authority has established calculation model figures of earthquake ground motions at 38 locations inside the city[1)]. In this examination, the assumption earthquake ground motion calculated in the vicinity of the bridge is used as the Level 2 Earthquake Ground Motion. The **Fig.4** shows the acceleration response spectrum of the input earthquake ground motion.

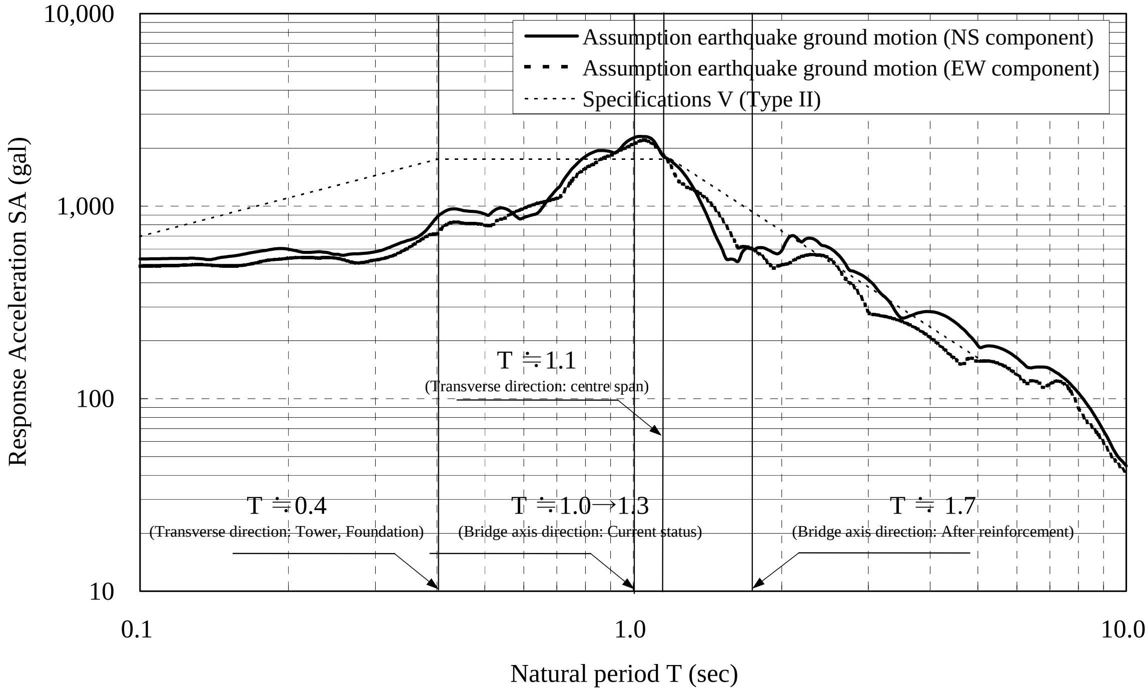


Fig.4 Acceleration response spectrum (h=5%)

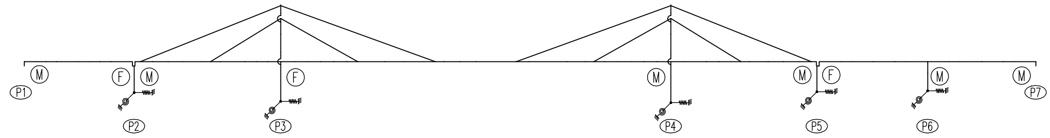
4. SEISMIC PERFORMANCE EVALUATION OF THE EXISTING BRIDGE

4.1 Analytical Model

The integrated analytical model shown in the **Fig.5** that includes both cable-stayed bridge and approach bridges was adopted in this examination and the main girder, towers as well as cables are considered to be as linear member and RC piers are assumed to be nonlinear member of trilinear model. In regard with the caisson foundation, it was evaluated as linear spring in horizontal direction and that in rotating direction.

4.2 Natural Vibration Characteristics

Before analyzing the time-history response of the model, evaluation of natural vibration analysis was conducted in order to examine the vibration characteristics within the elastic ranges. **Fig.6** and **Fig.7** respectively indicate the attention mode of the model in bridge axis direction and that in transverse direction to the bridge axis. The dominant mode

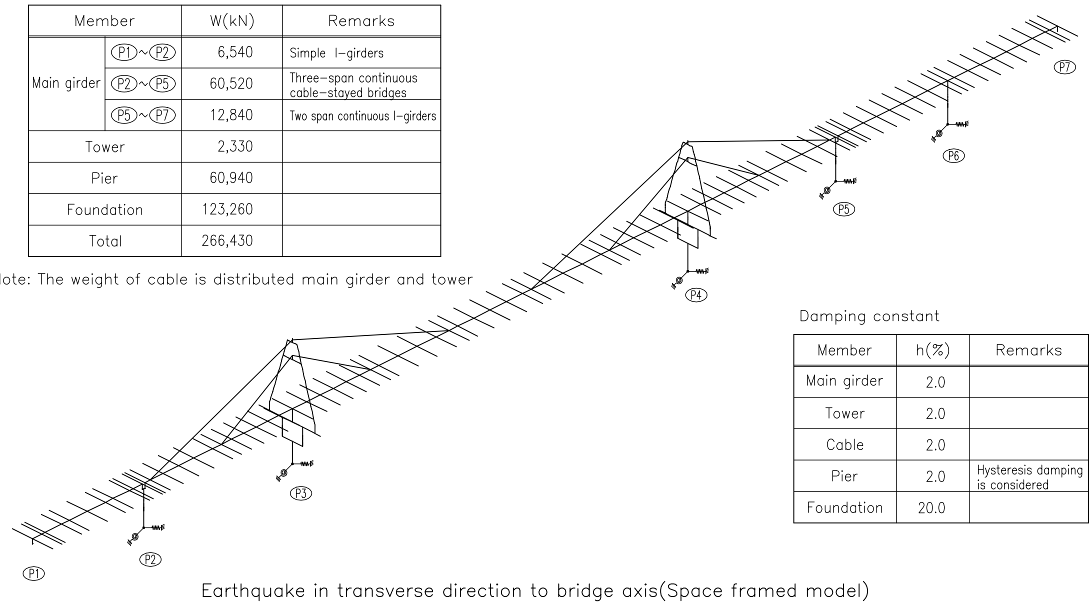


Earthquake in bridge axis direction (Plane framed model)

Weight

Member		W(kN)	Remarks
Main girder	P1~P2	6,540	Simple I-girders
	P2~P5	60,520	Three-span continuous cable-stayed bridges
	P5~P7	12,840	Two span continuous I-girders
Tower		2,330	
Pier		60,940	
Foundation		123,260	
Total		266,430	

Note: The weight of cable is distributed main girder and tower



Damping constant

Member	h(%)	Remarks
Main girder	2.0	
Tower	2.0	
Cable	2.0	
Pier	2.0	Hysteresis damping is considered
Foundation	20.0	

Earthquake in transverse direction to bridge axis(Space framed model)

Fig.5 Dynamic analytical model of Toyosato Bridge

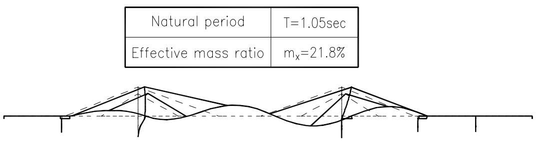


Fig.6 Vibration mode in bridge axis direction

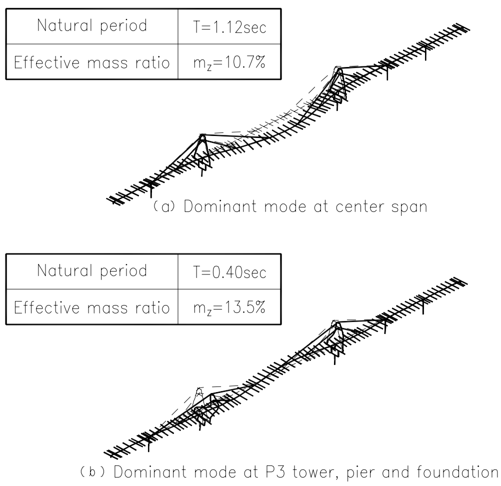


Fig.7 Vibration mode in transverse direction

in bridge axis direction is a mode that both main girder and towers deform in the bridge axis direction accompanying the deformation in vertical direction of main girder. This vibration frequency band coincides to the vicinity of the maximum acceleration figures of the input earthquake. The dominant mode in transverse direction to the bridge axis is the mode in which center span deflect and the mode where towers, piers and the foundations are deformed.

4.3 Earthquake Response Values of the Current Status

1) Bridge axis direction

Fig.8 shows the response values caused by the earthquake in bridge axis direction. As show in this figure both the horizontal reaction at the girder pin bearing and responding curvature of the piers are largely exceeding the yield value because the supporting method applied by both cable-stayed bridge section and approach bridge sections are of one fixed support. Furthermore, it is resulted that insufficient durability against shear strength is indicated even by the inertia mass of the pier itself because the supporting condition of the P4 pier is movable. In addition, the horizontal displacement of the main girder of the approach bridge greatly exceeds 150mm, which is the gap prepared between the girders. In regard with the cable tension, this stays within the permissible range of its elasticity and the rotation angle of tower pin bearing also stayed below the allowable figure of its rotation angle.

2) Transverse direction to the bridge axis

Fig.9 shows the response values caused by the earthquake in transversal direction to

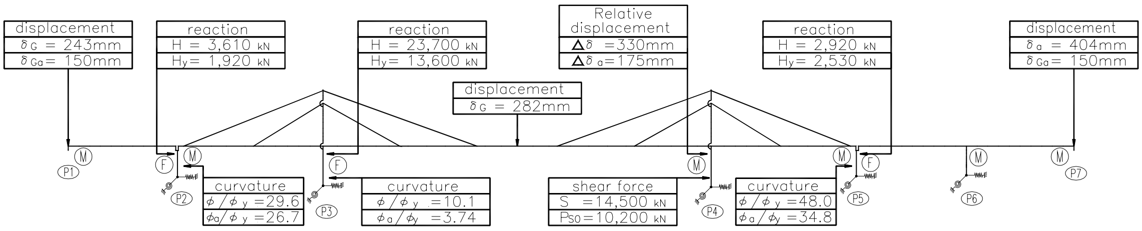
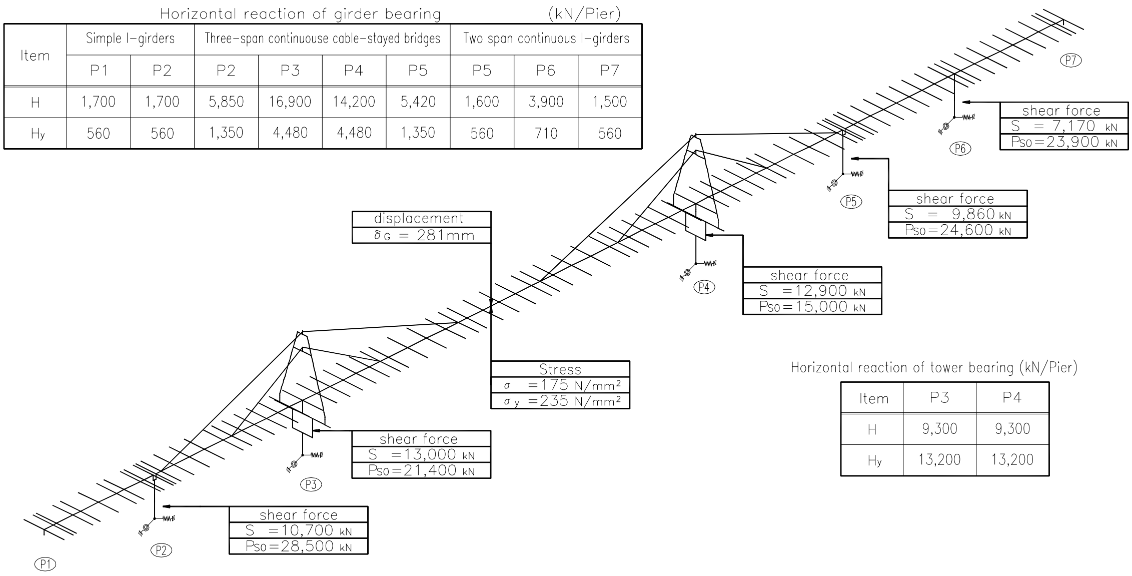


Fig.8 Response values at major member against earthquake in bridge axis direction



Horizontal reaction of girder bearing (kN/Pier)

Item	Simple I-girders		Three-span continuouse cable-stayed bridges				Two span continuous I-girders		
	P1	P2	P2	P3	P4	P5	P5	P6	P7
H	1,700	1,700	5,850	16,900	14,200	5,420	1,600	3,900	1,500
Hy	560	560	1,350	4,480	4,480	1,350	560	710	560

Horizontal reaction of tower bearing (kN/Pier)

Item	P3	P4
H	9,300	9,300
Hy	13,200	13,200

Fig.9 Response values at major member against earthquake in transverse direction

the bridge axis. Although the horizontal reaction that works to the pin bearing of the tower only shows the figure lower than the yielding lateral strength, the horizontal reaction at all other girder bearings indicates figures largely exceeding the yielding lateral strength of the same point. However, the behavior has barely stayed within the range of elasticity because the pier has comparatively big flexural rigidity as the bridge adopts wall-shaped piers or rigid-frame piers. Furthermore, the response shearing force is also indicating the level below the shear strength, to which the effect of deep beam is reflected. [2)]

5. DAMAGE SITUATION FORECAST AND BASIC POLICY FOR SEISMIC PERFORMANCE IMPROVEMENT

5.1 Damage Situation to be Forecasted

On the assumption of level 2 Earthquake Ground Motion attack to the bridge (Cable-stayed bridge section), the following status of damage receiving are assumed from the result of seismic performance verification made on the current condition of the bridge, which was obtained by time-history response analysis.

1) Bridge axis direction

① After receiving first destructive damage at girder pin bearing, it have repercussions to give damages to girder pin roller bearing then in sequence to pendel bearing and finally turns out unseating of the bridge.

② The piers are destroyed in brittle fracture due to shortage of shear strength.

2) Transverse direction to the bridge axis

① After receiving destructive damages at girder pin bearing and at girder pin roller bearing, tower columns are destroyed by the colliding motion of main girder, which moves in transverse direction to the bridge axis.

② Due to the damage received at lateral bearing, pendel bearings at girder end are destroyed by spread effect and results to unseating of the bridge.

5.2 Basic Policy for Seismic Performance Improvement

As the bridge is specified as important route on the emergency traffic road system prepared for large-scaled disasters, performance against damages is severely requested. Followings are the briefly summarized subjects of demand performance.

① Damage level shall be maintained as minimum within the negligible range, even when the bridge is attacked by level 2 earthquake. It shall be sufficiently available as an emergency traffic route after occurrence of earthquake.

② Critical damage such as unseating of bridge should not be caused even unanticipated situations are caused.

Fig.10 shows the basic policy for seismic performance improvement of the bridge considering the requested conditions mentioned above ①and ②.

The scale of the reinforcement to be worked for the substructure of the bridge shall be reduced by controlling the earthquake effecting force to be charged at cable-stayed bridge section by lengthening of the natural period and by improving of the damping effect. Furthermore, the basic policy of the measures shall be set to avoid collision of the girders with abutment parapet considering the fact that the approach bridge section has steel 2-I girders.

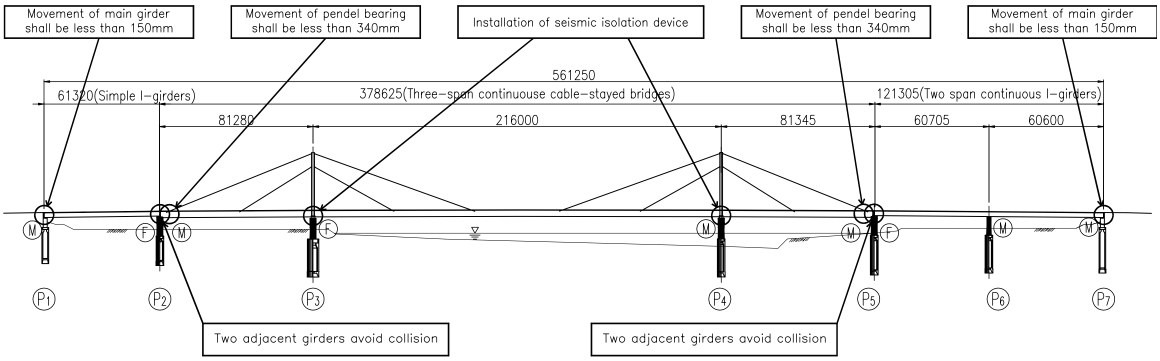


Fig.10 Basic policy for seismic performance improvement

6. CONSIDERATION FOR THE SEISMIC PERFORMANCE IMPROVEMENT

Based on the demand performance and the basic policy shown in the paragraph 5-2 above, the measures for seismic performance improvement indicated in **Fig.11** were selected.

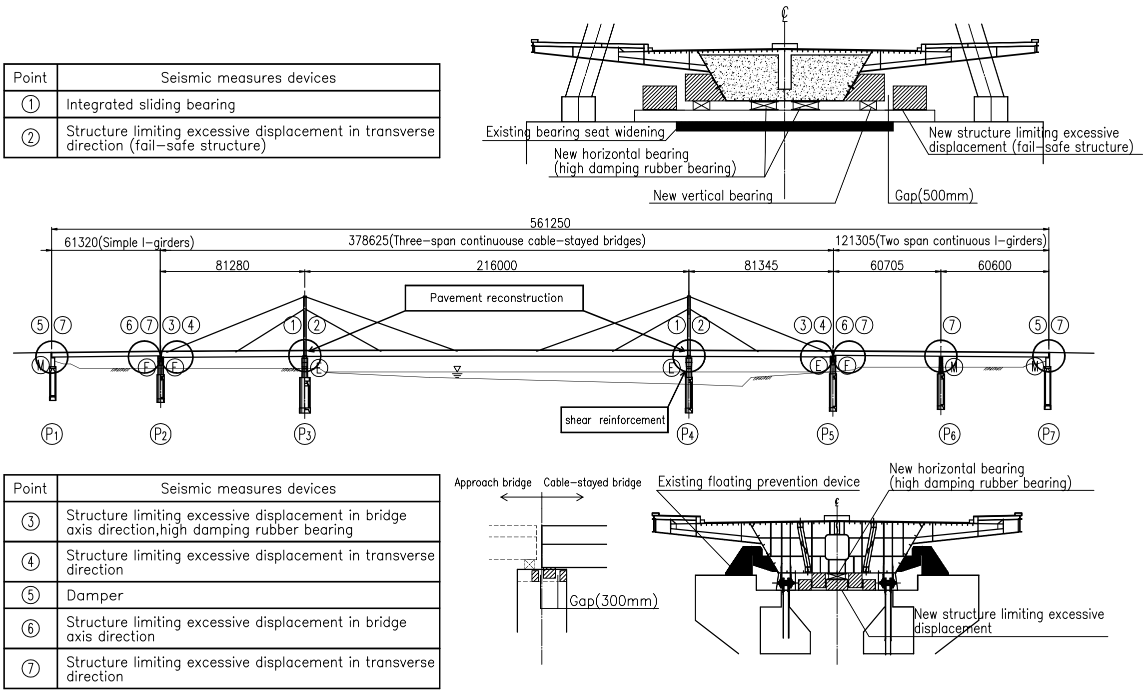
6.1 Bridge Axis Direction

1) Cable-stayed bridge intermediate support points P3 and P4 (**Fig.11**: Measures ①)

As the result of our examination made on the assumption to replace existing bearings with rubber bearings, it was found that the amplitude of compressive stress along with the change of live load becomes too large. In this relation, the designing of rubber bearings was actually found not possible and instead of this using of integrated sliding bearing which is configured from the combination of horizontal bearings and vertical bearings is adopted. In addition, this bridge has the characteristics that dead load reaction at girder bearing is small because of the individuality of this bridge and thus higher seismic effect cannot be obtained due to slipping friction damping at vertical bearing. Under such circumstance it was decided to use high damping rubber bearing for horizontal bearing in order to control the moving amount of main girder.

2) Cable-stayed bridge girder end points P2 and P5 (**Fig.11**: Measures ③ and ⑥)

Proper technique to control the relative displacement of main girder with the piers at girder end shall be examined because it becomes larger when existing bearings at intermediate support points are replaced with rubber bearings for the purpose to lengthen the natural period. The demanded performance as seismic measures in this subject shall be interpreted to maintain movable amount of the pendel bearing less than 340mm. It was decided to install the structure limiting excessive displacement to control relative displacement and the movable gap of which was set less than 300mm below the movable amount of the pendel bearing. This arrangement was prepared by securing more room than actual movable amount for ensured prevention of damage because the pendel bearing is the most important key point of the structure of the bridge. However, horizontal force acts on the structure limiting excessive displacement and response value of piers at girder end shall be properly decreased as they are expected to be increased in this case. For this purpose it



Point	Seismic measures devices
①	Integrated sliding bearing
②	Structure limiting excessive displacement in transverse direction (fail-safe structure)

Point	Seismic measures devices
③	Structure limiting excessive displacement in bridge axis direction,high damping rubber bearing
④	Structure limiting excessive displacement in transverse direction
⑤	Damper
⑥	Structure limiting excessive displacement in bridge axis direction
⑦	Structure limiting excessive displacement in transverse direction

Fig.11 Selected seismic performance improvement

was decided to install high damping rubber bearing together with the structure limiting excessive displacement on the piers at girder end.

3) Approach bridge girder end points P1 and P7 (**Fig.11**: Measures ⑤)

In regard with the horizontal displacement at girder end, it shall be made as the basic policy to maintain the gap less than 150mm, which is the current figure of the movable gap at the moment. Though the case examination was examined to replace existing bearings with rubber bearings, it was found this would make horizontal displacement of the main girder larger than the current figure. With such reason it was concluded to install damper at girder end point to reduce horizontal displacement of the main girder lower than the movable gap instead of bearing replacement.

6.2 Transverse Direction to Bridge Axis Direction

1) Cable-stayed bridge intermediate support points P3 and P4 (**Fig.11**: Measures ②)

There are two types of measures as effective method to be taken at intermediate support points that are (1) fixed support and (2) elastic support. However, the method (1) is considered to require structure limiting excessive displacement in large-scale because the horizontal reaction acts on existing girder becomes too large as 3 to 4 times as that of the yielding lateral strength. On the other hand, it was vilified that the horizontal reaction at girder end points hardly changes when the method (2) was taken because of its reduction effect of earthquake force by lengthening the natural period. It was decided to take elastic support method as the condition of support at intermediate support points in transverse direction to bridge axis direction. However, it was decided to add structure limiting excessive displacement and enough movable gaps was secured as the fail-safe measures because damages given to the tower might become critical damage directly resulted unseating of the bridge.

2) Cable-stayed bridge girder end points P2 and P5 (**Fig.11**: Measures ④)

For the measures to be taken at this point it was decided to secure lateral strength by installing structure limiting excessive displacement. This is because the damage received at lateral bearing may produce horizontal force in transverse direction, which may affect pendel bearing to destroy it.

3) Approach bridge girder end points P1, P6 and P7 (**Fig.11**: Measures ⑦)

Also at this point it was decided to secure lateral strength by setting up the structure limiting excessive displacement that combine the function of structure to prevent the superstructure from settling because the horizontal reaction works at girder bearing was greatly exceeding the yielding lateral strength.

6.3 Seismic Performance Improvement of Piers

As the result of seismic measures to be taken at the paragraphs 6-1 and 6-2 above it was concluded that any shear reinforcement should be taken for P4 pier although reinforcement for all other piers is not necessary. By taking these measures all piers of the bridge do not require any bending reinforcement and this was resulted to minimize the repercussion given to the foundation of the bridge.

7. OBTAINABLE EFFECTS FROM REDUCTION OF REINFORCEMENT SCALE AND COST MINIMIZING

The **Table 2** shows the comparison result made between the seismic performance improvement based on the static checking method (ductility design method) and that based on the dynamic checking method that has been selected. Drastic cut down of the construction expenses became to be realized including the temporary bridge construction work by reducing the number of piers that need reinforcement for seismic performance improvement as well as minimizing of reinforcing scale in large degrees. In addition, this method gives side effect to avoid damages to the existing structures because it became unnecessary to install the PC steel rods that penetrate through all piers of the bridge.

Table 2 Effect obtained by the selected seismic performance improvement

Item		Static checking method		Dynamic checking method	
Details of reinforcement	P2 pier	t=300mm	Bending, Shear and ductility reinforcement	-	Reinforcement unnecessary
	P3 pier	t=500mm	Bending, Shear and ductility reinforcement	-	Reinforcement unnecessary
	P4 pier	t=300mm	Bending, Shear and ductility reinforcement	t=250mm	Shear reinforcement
	P5 pier	t=300mm	Shear and ductility reinforcement	-	Reinforcement unnecessary
	P6 pier	t=300mm	Bending, Shear and ductility reinforcement	-	Reinforcement unnecessary
	Remarks	PC steel rods necessary		PC steel rods unnecessary	
Effect given to foundation		Large		Small	
Approximate cost ratio	Superstructure	0.21		0.67	
	Substructure	1.47		0.33	
	Total	1.68		1.00	

8. CONCLUSIONS

As the result of thorough evaluation made on the seismic performance improvement to be examined on the Toyosato Bridge, minimizing of the reinforcement scale was enabled as a result by the lengthening of the natural period and by the improvement of damping performance. Now that it was confirmed the cost minimizing effect of the seismic performance improvement based on the dynamic checking method, further steps shall be performed for the detailed designing based on the selected seismic performance improvement.

REFERENCES

1) Osaka City: Report from Osaka Civil Engineering and Architectural Structure Seismic Measures Technological Investigative Commission, 1997.3 (In Japanese)
2) Japan Bridge Engineering Center: Examples of seismic reinforcement method for existing bridges, 2005.4 (In Japanese)