13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No.1302

SEISMIC ANALYSIS OF CABLE-STAYED BRIDGE WITH 2-EDGE GIRDERS

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SUMMARY

The cost of composite cable-stayed bridge with 2 edge girders is superior to any other types of cable-stayed bridges. So many bridges of this type were constructed in overseas. This paper reports seismic resistance of composite cable-stayed bridge. As the model of composite cable-stayed bridges, Owensboro Bridge which was actually constructed in the U.S in 2002, was used. To investigate the seismic behavior, 3-dimentional dynamic analysis and inspection was carried out using Japanese Specification for Highway-bridges Part V(Seismic Design). As the result of the analysis , it was clear that Owensboro Bridge has not the required seismic resistance by Japanese Specification, but seismic resistance of this bridge was improved by using isolation bearing, damper and changing bearing layout without loosing cost benefit.

INTRODUCTION

Cable-stayed bridge with 2-edge girders has not been constructed in Japan, but the constructions of this type of bridge are increasing in U.S. and China. It was reported by the authors that a cable-stayed bridge of this type had the same cost efficiency compared to PC rigid frame bridge in Japan. The authors have already studied on the cost efficiency and seismic performance of the cable stayed bridge with 2-edge girders[1)~5)], but in this paper investigates seismic performance of the cable stayed bridge with 2-edge girders constructed in U.S. is discussed using Japanese Specification for Highway-bridges Part V(Seismic Design). The purpose of this paper discuses the rationality and the improvement method of seismic performance by using the result of the above investigations assuming this bridge would be constructed in Japan.

OUTLINE OF THE BRIDGE

The Owensboro Bridge over Ohio River in Kentucky State in U.S. are selected as model bridge. This bridge was

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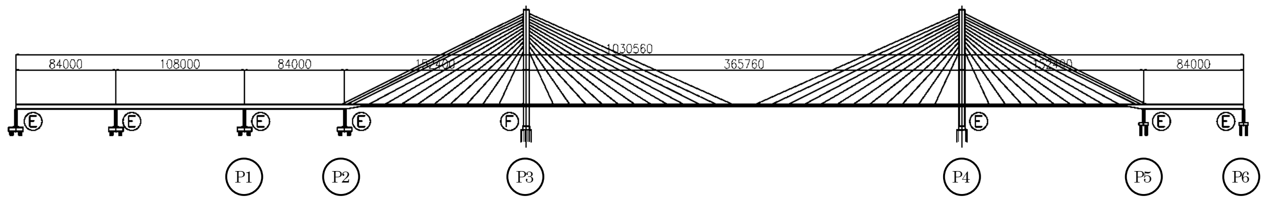


Figure 1 General View (unit：mm)

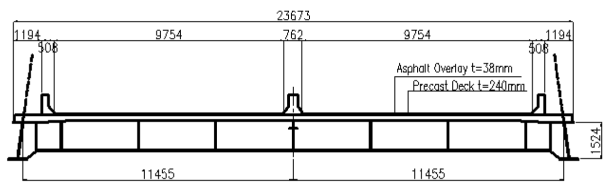


Figure 2 Cross Section of Main Girder (unit：mm)

completed at 2002.This bridge is composed of three bridges, main bridge located in the center is the cable stayed bridge with 2 edge girders and two bridges neighboring to the cable stayed bridge are plate girder bridges with RC slab. Total bridge length is 1,373m, and the length of the cable-stayed bridge is 1,031m. Figure 1 shows the general view of this cable-stayed bridge. Figure 2 and Figure 3 show the typical structure drawing. Structure of this bridge is outlined below. Span: center span length=366m, side span length=152m. Type of the cable configuration : fan type. Girder type: 2 edge girders (girder height =1.52m). Type of the slab :pre-cast reinforced-concrete slab. Main structure is composed of the pre-cast reinforced concrete slab (thickness is 240mm) and 2-edge girders as Figure 2 indicates. Two edge girders are connected by cross beams placed at intervals 4.6m. RC(reinforced concrete) tower is the hollow section and the tower height is 83m, the ratio of the tower height to span length is 4.4. This ratio is classified into that of the general cable stayed bridge.

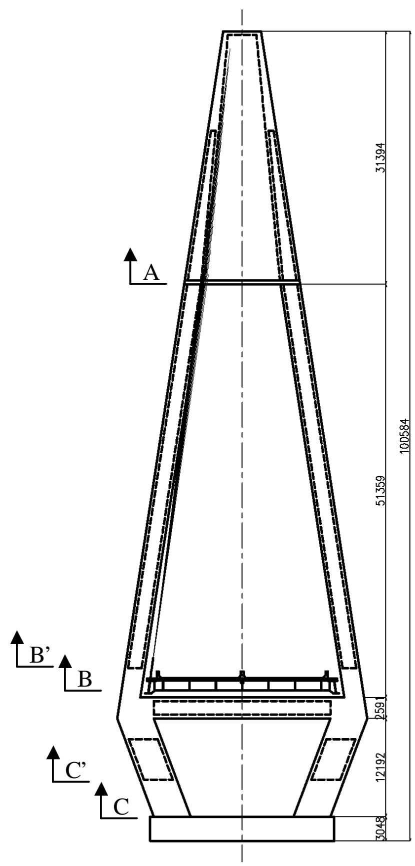


Figure 3 Tower Elevation (Hollow Column)

Support conditions of the girders at the tower and piers are following. One of two towers are supported by longitudinal fixity and another tower and piers are supported by longitudinal elasticity.

This bridge was designed by using AASHTO. Erection site in Kentucky State is the region where seismic motion is small, so the dominant force acting on the pier and the tower is live load. Seismic motion is not dominant factor. Therefore the tower and piers were only checked using the response spectrum defined by Type I Soil in AASHTO. Type I Soil is classified into rock ,so the soil condition is good. Two support conditions affecting to the seismic performance were investigated. One is the case that elastic bearings were installed at all piers and another is the case that the longitudinal movement is fixed at one pier. As the result, this bridge is fixed by using the bracket as shown Figure 4 for longitudinal seismic motion. The movement in transverse direction is fixed by the equipment shown Figure 5.

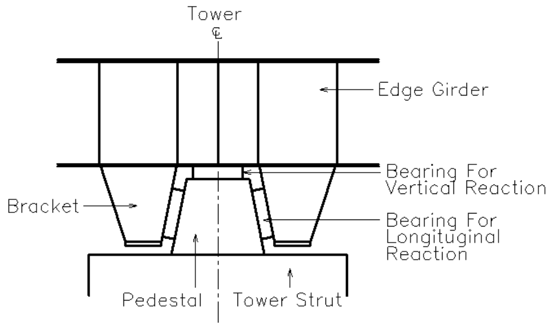


Figure 4 Longitudinal Fixity

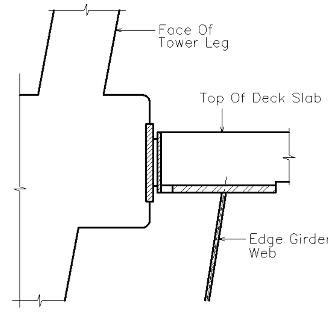


Figure 5 Sway Bumpers

SEISMIC BEHAVIOR OF THE MODEL

Outline

The seismic behavior of each member of the bridge was investigated by using seismic motions level 2 defined by Japanese design specifications of highway bridges, Part V, seismic design.

Modeling

Assuming that two main girders were composed of the independent beam element respectively, each girder was connected by rigid cross beams in analytical model. However, for the simplification of the calculation, the girders of the approach bridges were made using single beam element. Actually, this bridge is 7-span continuous bridges including the approach bridge, this paper dealt with the bridge as 5-span continuous bridge which was composed of 3-span continuous cable stayed bridge and neighboring spans, because this calculation is focused on the behavior of 3-span continuous cable-stayed bridge.

The models were defined as below. The cables are linear truss elements, the girders and the cross beams are linear beam elements, RC towers and RC piers are nonlinear beam element. Nonlinear characteristics of RC members are tri-linear type for the skeleton curve and Takeda model for the hysteresis characteristics. The stiffness of the main girders includes the slab, because this bridge is designed as the composite bridge. The support condition between the foundation and the soil is considered as infinitely large spring, because of the Ground class is Class Ⅰ. The elastic bearing was modeled by using linear spring. Figure 6 shows analytical model. Standard seismic wave presented in "Design Specifications of Highway Bridges, Part V, Seismic Design" was used as input seismic motions. Seismic wave forms of Kaihoku LG for Type Ⅰ and JME Kobe NS for Type Ⅱ were selected. Input seismic motion was inputted at a time in each support to calculate the longitudinal behavior of the bridge.

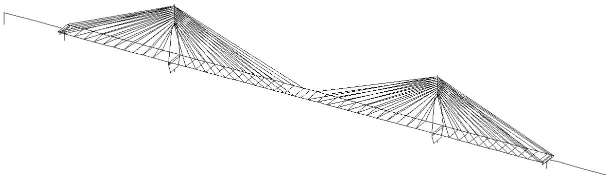


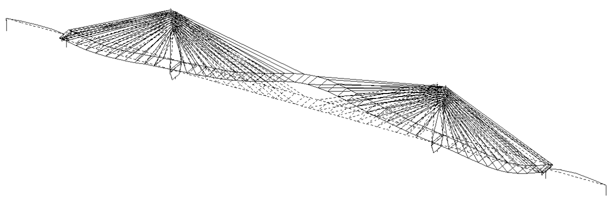
Figure 6 Analytical Model

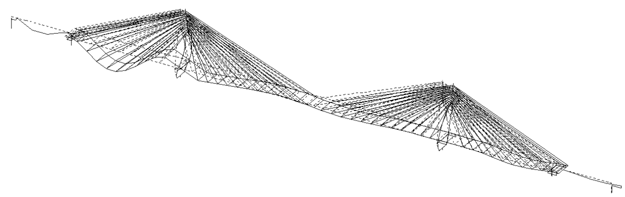
Table 1 Period and Effective Mass Ratio

mode	Period (sec)	Effective Mass (%)	Notes
1	3.816	3.1	Vertical Bending(Symmetry-First)
2	2.665	3.8	Vertical Bending(re-Symmetry)
3	1.905	4.3	—
4	1.796	4.3	—
5	1.613	7.4	Vertical Bending(Symmetry-Second)
6	1.397	9.8	Vertical Bending(re-Symmetry-Second)
7	1.298	18.8	Vertical Bending(Symmetry-Third)
8	1.207	37.8	Axial Sway
9	1.122	38.3	—
10	1.112	41.2	Torsional

Analysis of eigen value

First natural period is 3.82 sec, this mode shape is the vertical deflection mode of the girder. Table 1 shows the result of the eigen value analysis and the effective mass ratio from first to tenth. From this table it is clear that the effective mass of the low order are small compared to the high order. This is the reason why the girder was fixed at the tower. For this reason, the superior mode did not appear at low order. As the order become higher, the effective mass increase gradually. Figure 7 shows the typical mode shapes.





Mode 1 Mode 8

Figure 7 Mode Shapes

Section force of each members

Time history analysis was carried out and the section force and the displacement were calculated. These responses were checked by reference to the allowance described in "Design Specifications of Highway Bridges Part V

Seismic Design". Focusing members are the tower (the cable anchorages, the bottom section of the tower and the pier), piers of the neighboring bridges and bearings. Analytical method is Newmark's β (β=1/4) method and the integral interval is 0.002sec. Damping constant was calculated by Rayleigh's damping focusing on primary vibration mode. From Figure 8 to Figure 10 shows the bending moments skeleton calculated at each section in fixed side tower P3. Each graph shows the nonlinear characteristics (moment-curvature) through Crack, Yield, and Ultimate point. Maximum response of the each section is plotted on this line. From this figure, it is clear that the result of the calculation doesn't satisfy the allowance for the seismic force concentrating at the bottom section C (C-C section in fixed side tower) of the pier 3. Especially it is clear that when Type I seismic motion attacked, the bending moment exceeds ultimate bending moment. The value of the bending moment at cable anchorage section A (A-A section) and the bottom section B of the tower (B-B section) except the bottom of the piers dose not exceed the yield bending moment. At the same time, the response of the P4 tower at the movable side dose not exceed the yield bending moment at any points. And as the Table 2 indicates, the shearing force excesses the shearing proof at the bottom of the tower in a fixed side tower.

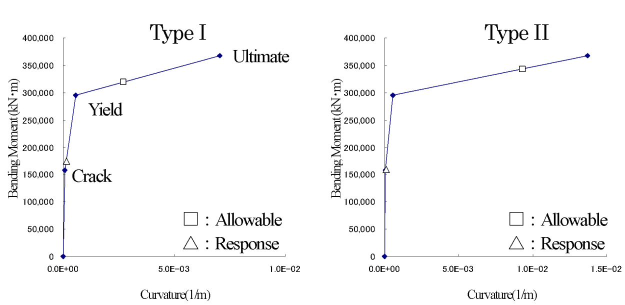


Figure 8 Moment-Curvature relation at Cable Anchorage Section A in P3 Tower (ref .Figure3)

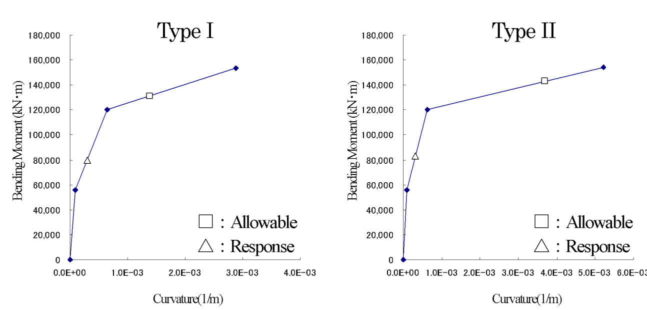


Figure 9 Moment-Curvature relation at Bottom Section B in P3 Tower(ref.Figure3)

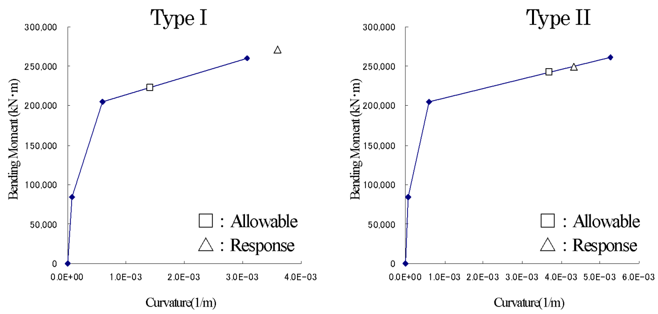


Figure 10 Moment-Curvature relation at Bottom Section C in P3 Pier (ref.Figure3)

Table 2 Verification of Shearing Proof

		Level 2 Earthquake Ground Motion					
		Type I			Type II		
		Sharing Force kN	Sharing Proof kN		Sharing Force kN	Sharing Proof kN	
P3 Fixed	Section A	6902	20147	OK	8128	20620	OK
	Section B	2715	15468	OK	3868	16073	OK
	Section C'	32244	11932	NG	31774	12215	NG
	Section C	27771	26025	NG	25450	27105	OK
P4 Movable	Section A	3536	20147	OK	3940	20620	OK
	Section B	2905	15468	OK	3912	16073	OK
	Section C'	4107	11932	OK	4752	12215	OK
	Section C	4500	26025	OK	5900	27105	OK

Pier of approach bridges

Figure 11 and Figure 12 show the response at the bottom of the pier of the approach bridges. The response exceeding the yield bending moment does not occur at the pier of approach bridges. Shearing force calculated at each pier does not exceed the shearing proof.

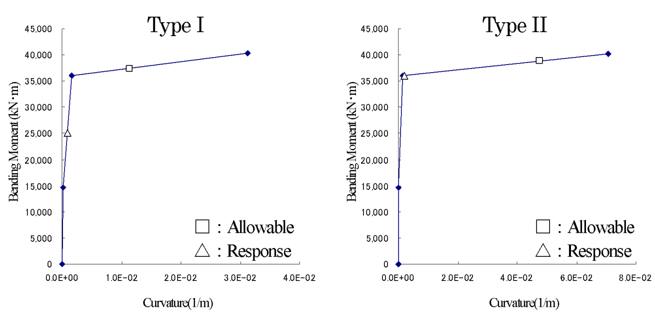


Figure 11 Moment-Curvature relation at Bottom Section in P1 Pier

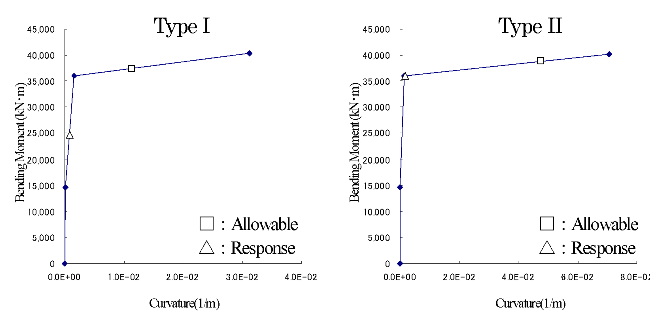


Figure 12 Moment-Curvature relation at Bottom Section in P2 Pier

Displacement

Table 3 shows the displacement of pier and tower. Assuming that the allowable displacement is 250% strain of the thickness of rubber bearing, the displacement of the bearing at movable side tower (P4) exceeded the allowable displacement.

Table 3 Maximum Response Displacement

		Type I m	Type II m	Allowable Displacement 2.5Σte
P1 Pier (Movable)	Top Of The Pier	0.037	0.083	—
	Bearing	0.226	0.166	0.476
	Superstructure	0.198	0.198	—
P2 Pier (Movable)	Top Of The Pier	0.034	0.080	—
	Bearing	0.165	0.159	0.476
	Superstructure	0.197	0.197	—
P3 Tower (Fixed)	Top Of The Pier	0.138	0.147	—
	Superstructure	0.188	0.186	—
P4 Tower (Movable)	Top Of The Pier	0.025	0.016	—
	Bearing	0.230	0.247	0.195
	Superstructure	0.229	0.250	—

Summary

Time history analysis for this model was carried out using level 1 seismic motion and level 2 seismic motion. As the result, it is concluded that the section force are concentrated to fixed side tower P3 and the section force exceeds the allowance if the calculation of the seismic performance is carried out by "Design Specifications of Highway Bridges Part V Seismic Design". And the displacement of the movable side tower P4 exceeded the allowable displacement. This reason is the difference of the size of seismic motion considered between AASHOTO and Japanese Design Specifications of Highway Bridges.

The following becomes clear. When the bridge having the dimensions and bar arrangements designed by the relatively small seismic motion as this bridge was checked by Design Specifications of Highway Bridges Part V Seismic Design, there are members which does not satisfy the seismic performance.

IMPROVEMENT OF THE SEISMIC PERFORMANCE

Improvement due to reinforcement

It was clear that after comparisons of the calculation result and the proof of members, at fixed side tower the bending moment and the shearing force does not satisfy the allowance. By modifying the quantity of the reinforcement in the section, to satisfy the allowance was investigated. Figure 13 shows C-C section at the bottom of the existing tower. The section is composed of the lateral hoop tie of 2.84cm^2 and the intermediate tie bars of 2.00cm^2. Intermediate ties is set at 450mm～750mm intervals. This interval is narrow compared with the interval provided for in "Japanese Design Specifications of Highway Bridges ".

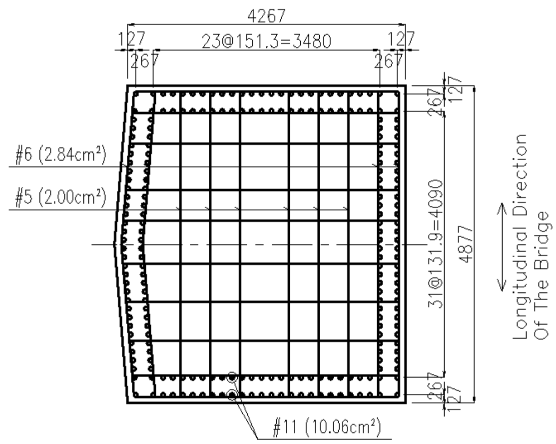


Figure 13 Bottom Section at Tower(sec C)

However for the shearing force exceeding the shearing poof, the quantity of tie bars need to increase. Here, assuming the commonly used diameter and interval of tie bars in the Japan seismic specification, the intervals of tie bars were changed to following.

The diameter of the tie bar is D25 (area is 5.067cm^2), the interval of tie bars is 1m respectively. These size and dimension are the value adopted generally in Japan. And for increasing the proof of the bending moment, the diameter of the main reinforcement was changed to D41 (area is 13.40 cm^2) and the proof of the section was calculated after the reinforcement again. This bar arrangement is the maximum quantity of reinforcement if the size of the bottom section is not changed. Figure 14 shows the verification after having increased the quantity of the reinforcement. When Type I seismic motion attacked, the response exceeding ultimate bending moment decreases within ultimate bending moment. But the response did not satisfy the allowance. Despite of the improvement of shearing proof by increasing the quantity of tie bars, shearing force at the bottom of fixed side tower exceeded the shearing proof as Table 4 indicates. Therefore it follows that the size of this section itself is small.

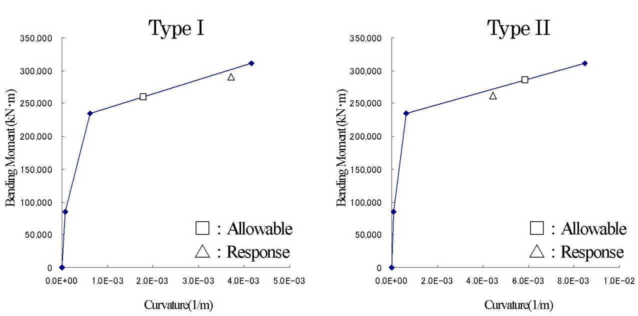


Figure 14 Moment-Curvature relation at Bottom Section C in Pier (after Reinforce)

Table 4 Verification of Shearing Proof (after Reinforce)

		Level 2 Earthquake Ground Motion					
		Type I			Type II		
		Sharing Force kN	Sharing Proof kN		Sharing Force kN	Sharing Proof kN	
P3	Section C'	33287	20621	NG	32918	20904	NG
	Section C	28758	52954	OK	26091	54131	OK

Improvement due to bearing

All elastic bearing model

For distributing the seismic force concentrated at one pier to the other piers, the analysis was carried out by changing the support condition at P3 tower from the fixed bearing to the elastic bearing, and at other piers from movable to elastic bearing. Analysis was carried out by assuming that the reinforcement is same as original model. Figure 15 shows the verification of response of bending moment at the bottom of the tower after changing the bearing condition. The response exceeding ultimate bending moment at type I seismic motion (Figure 10) decreases to within yield bending moment. Because seismic motion was distributed, the bending moment of the approach piers increase and the response of the pier exceed the allowance when type I seismic motion attacked, as Figure 16 shows.

However because the value in excess is small, by increasing the size of the section little more, seismic performance would be satisfied the allowance. Figure 17 shows the section of the P1 pier. In this paper, the detail result of the calculation is omitted, but we confirmed that the seismic performance will be satisfied allowance by changing the main reinforcement #10 (area is 8.19cm^2) to D41(area is 11.4cm^2) without changing the cross-sectional dimensions of the pier. And the shearing force exceeding the shearing proof at the bottom of the P3 tower decreased within the shearing proof as Table 5 shows (ref. Table 2).

As above mentioned, by changing the bearing support condition, the section force at the tower decreases. And it was confirmed that the piers in approach spans satisfied the allowance by increasing the diameter of the main reinforcement. However, focusing on the bearings, the excessive displacement of the superstructure occurred by using the elastic bearings at all supports as Table 6 indicates.

As the result, the displacement of the bearing at the tower greatly excesses the allowable displacement. The excessive displacement like this is not acceptable from the view point of seismic performance. By using the hysteresis dumping of the LRB(Lead Rubber Bering) at the bearings, the decreases of the section force and the displacement are expected.

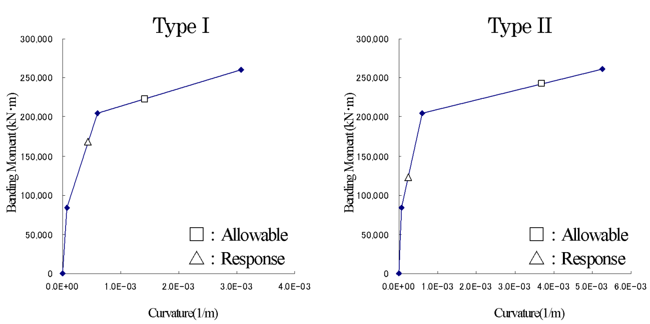


Figure 15 Moment-Curvature relation at Bottom Section C in P3 Pier (Elastic Bearing)

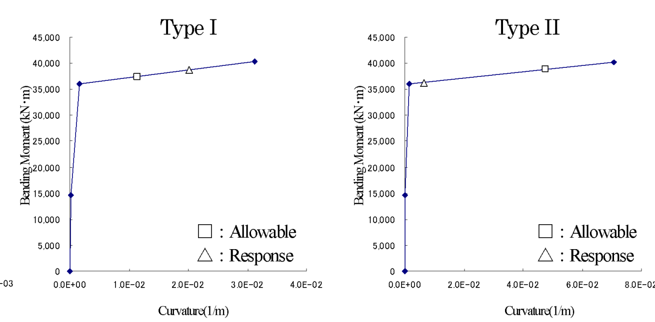


Figure 16 Moment-Curvature relation at Bottom Section C in P1 Pier (Elastic Bearing)

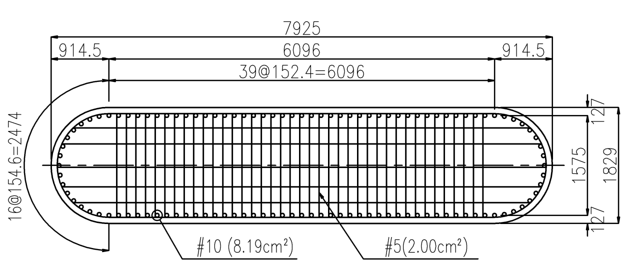


Figure 17 Bottom Section at P1 Pier

Table 5 Verification of Shearing Proof (Elastic Bearing)

		Level 2 Earthquake Ground Motion					
		Type I			Type II		
		Shering Force kN	Shering Proof kN		Shering Force kN	Shering Proof kN	
P3	Section C'	7262	11932	OK	5628	12215	OK
	Section C	7443	26025	OK	6813	27105	OK
P4	Section C'	7425	11932	OK	5526	12215	OK
	Section C	7483	26025	OK	6840	27105	OK

isolation bearing model

The size of the isolation bearings at P1, P2, P5 and P6 piers are 1000mm×750mm, thickness is 190.5mm, and at the P3, P4 towers size is 750mm×750mm and same thickness. If the thickness of the bearing at the tower is 76mm (this is the thickness decided by the displacement of vertical loads), the displacement become large, as the result it is impossible that the displacement of the bearing satisfies the allowance. And the figure of the lead was decided by using the general ratio the rubber to the lead,i.e 6%～10%. Figure 18 shows the verification result of the bending moment at the bottom of the tower when the isolation bearings were used at all supports.

Table 6 Verification of Shearing Proof (Elastic Bering)

		Type I m	Type II m	Allowable Displacement 2.5Σte
P1 Pier	Top of The Pier	0.477	0.157	—
	Bearing	0.429	0.350	0.476
	Superstructure	0.797	0.458	—
P2 Pier	Top of The Pier	0.449	0.155	—
	Bearing	0.424	0.340	0.476
	Superstructure	0.797	0.459	—
P3 Tower	Top of The Pier	0.037	0.019	—
	Bearing	0.777	0.452	0.195
	Superstructure	0.798	0.460	—
P4 Tower	Top of The Pier	0.040	0.023	—
	Bearing	0.772	0.450	0.195
	Superstructure	0.797	0.460	—

In comparison with the elastic bearing at all supports, the bending moment at the bottom of the pier decreases furthermore(ref. Figure 15). And the bending moment at the bottom of the P1 pier in the approach bridge was decreased due to dumping effect of the isolation bearings. But the bending moment excesses the allowance (ref. Figure 19). Table 7 shows the summary of the displacement. In comparison with the elastic bearing, the displacement of the bearing decreases on the whole, but the displacement of the bearing at the tower dose not satisfy the allowance. For this reason, the size of the isolation bearing and the ratio of the lead to the rubber area were increased in comparison with typical ratio of the isolation bearing. The calculation was carried out again by increasing the effect of the dumping. Table 8 shows the bearing size after increasing the dumping. This result is shown at Figure 20 and Table 9. In case of the original bearing size, the result of the calculation dose not satisfy the allowance, but by improving the isolation bearing, bending moment at the pier in approach bridge and the displacement of the bearing at the tower satisfied the allowance.

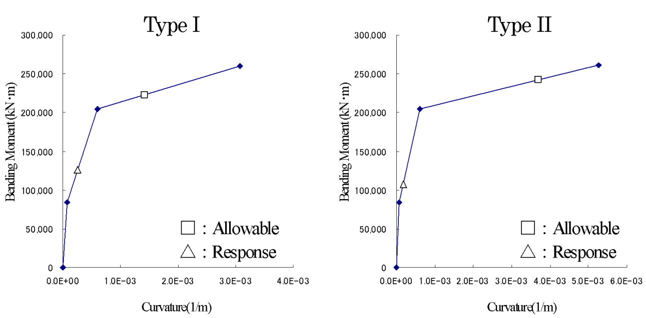


Figure 18 Moment-Curvature relation at Bottom Section C in P3 Pier (Initial Isolation Bearing)

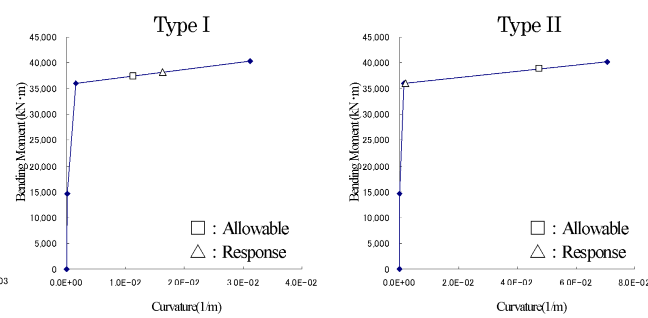


Figure 19 Moment-Curvature relation at Bottom Section in P1 Pier (Initial Isolation Bearing)

Elastic bearing and Damper

The elastic bearing were installed at every support and LED damper shown Figure 21 was installed between the pier and the superstructure. This damper is the hysteresis damper with the help of plasticity of the lead and this damper has the hysteresis characteristics of bi-linear as shown Figure 22. Secondary stiffness is approximately zero. As the result, the characteristics of the hyiteresis become the rectangular and the effect of the dumping is expected. Because the displacement for the yield load is small, lateral displacement of the bridge can be controlled. 4 dampers having the resistance force 500kN was installed on the each pier and tower respectively. Summation of the resistance force becomes 2000kN. Figure 23 shows the result of the calculation focusing on the bending moment at the bottom of the

Table 7 Verification of Bearing Displacement (Initial Isolation Bearing)

		Type I m	Type II m	Allowable Displacement 2.5Σte
P1 Pier	Top of The Pier	0.383	0.094	—
	Bearing	0.274	0.233	0.476
	Superstructure	0.649	0.296	—
P2 Pier	Top of The Pier	0.376	0.093	—
	Bearing	0.274	0.234	0.476
	Superstructure	0.649	0.296	—
P3 Tower	Top of The Pier	0.023	0.015	—
	Bearing	0.644	0.295	0.476
	Superstructure	0.651	0.297	—
P4 Tower	Top of The Pier	0.022	0.013	—
	Bearing	0.646	0.293	0.476
	Superstructure	0.651	0.295	—

Table 8 Bearing Size

P1,P2,P5 and P6 Pier

	Bearing Size			Lead Plug	Rubber-To-Lead
	Width		Thickness	Diameter	Ratio
	mm	mm	mm	mm	%
Initial	1000	750	190.5	140	8.21
final	1000	750	200.0	170	12.11

P3 and P4 Pier

	Bearing Size			Lead Plug	Rubber-To-Lead
	Width		Thickness	Diameter	Ratio
	mm	mm	mm	mm	%
Initial	750	750	190.5	125	8.73
final	1000	750	240.0	200	12.57

Table 9 Bearing Displacement (Isolation Bearing Last)

		Type I m	Type II m	Allowable Displacement 2.5Σte
P1 Pier	Top of The Pier	0.300	0.108	—
	Bearing	0.266	0.182	0.500
	Superstructure	0.520	0.259	—
P2 Pier	Top of The Pier	0.284	0.107	—
	Bearing	0.253	0.190	0.500
	Superstructure	0.521	0.259	—
P3 Tower	Top of The Pier	0.019	0.015	—
	Bearing	0.515	0.256	0.600
	Superstructure	0.523	0.259	—
P4 Tower	Top of The Pier	0.020	0.014	—
	Bearing	0.515	0.252	0.600
	Superstructure	0.523	0.257	—

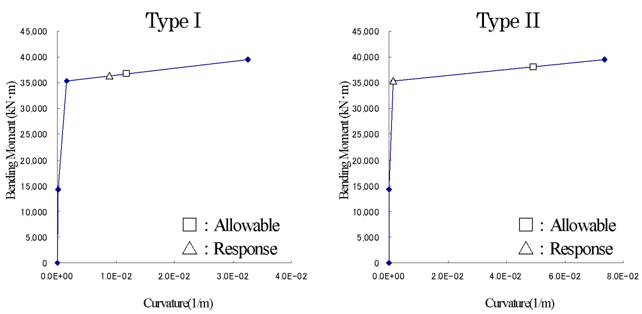


Figure 20 Moment-Curvature relation at Bottom Section in P1 Pier (Final Isolation Bearing)

tower when the damper was installed. Compared with the case that elastic bearings were installed at all supports, the bending moments at the bottom of the towers decrease due to the effect of hysteresis dumping as same as the isolation bearings.

When the elastic bearings or isolation bearings were installed at the supports, the bending moments at the bottom of the pier exceeded the allowance. But by using the damper, the bending moment satisfied the allowance as Figure 24 indicates. Table 10 shows the total of displacement. The displacement of the superstructure decreases dramatically compare with those of the elastic bearings and the isolation bearing. But the displacement of the bearing at the tower slightly exceeds the allowable displacement, it would be possible to satisfy the allowable displacement by increasing the size of the bearing. The damper is defferent from the bearing in the point of the size and the structure, so restrictions for the install are less than the bearings and expected hysteresis damping can be obtained easily. So we concluded that the install of the damper is one of the effective methods for improving seismic performance.

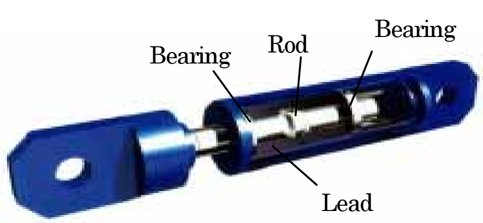


Figure 21 LED Damper

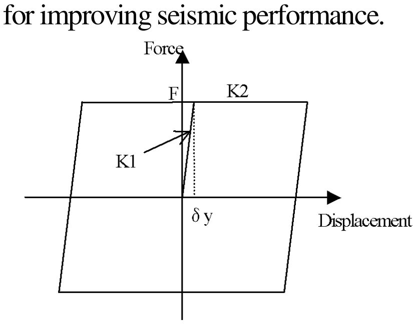


Figure 22 Hysteresis Characteristic of LED Damper

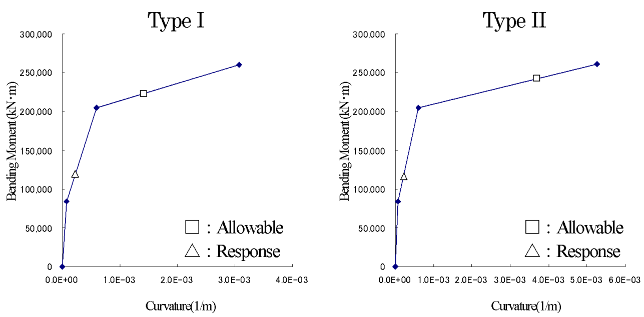


Figure 23 Moment-Curvature relation at Bottom Section C in P3 Pier (Damper)

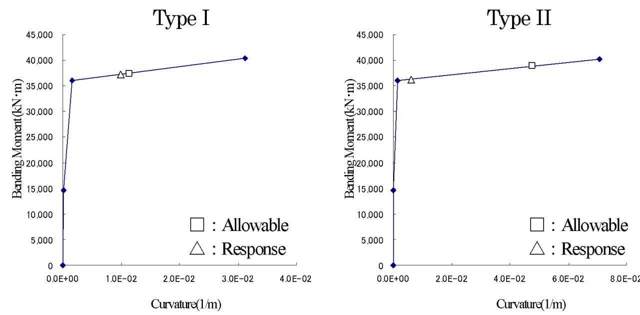


Figure 24 Moment-Curvature relation at Bottom Section in P1 Pier (Damper)

CONCLUSION

Composite cable-stayed bridge with 2 edge girders constructed in the region where large seismic motion does not affect the structure was checked by "Design Specifications of Highway Bridges, Part V, Seismic Design" . The dead load of composite cable stayed bridge that has a concrete slab is heavy compared to the steel deck girder. Therefore, when the excessive seismic motion affects the piers and the towers in Japan, it is concerned that the cost effect loses. In this paper, four methods for improving the seismic performance was applied to the model of the bridge constructed in U.S. And without changing the dimensions of piers and the towers, the improvement of the seismic performance was investigated. The result of the analysis is follow.

Calculation result of the original model

1)The seismic force gathers at the bottom of the pier at the fixed side tower. So the section force exceeds the proof, of displacement of the bearing at movable side tower exceeds the allowable displacement.

2)As the result the original model does not satisfy allowance described in "Design Specifications of Highway Bridges, Part V ,Seismic Design".

Table 10 Verification of Bearing Displacement(Damper)

		Type I m	Type II m	Allowable Displacement 2.5Σte
P1 Pier	Top of The Pier	0.232	0.177	—
	Bearing	0.051	0.038	0.476
	Superstructure	0.280	0.218	—
P2 Pier	Top of The Pier	0.228	0.189	—
	Bearing	0.053	0.041	0.476
	Superstructure	0.280	0.219	—
P3 Tower	Top of The Pier	0.018	0.017	—
	Bearing	0.271	0.206	0.195
	Superstructure	0.281	0.218	—
P4 Tower	Top of The Pier	0.017	0.017	—
	Bearing	0.269	0.210	0.195
	Superstructure	0.279	0.215	—

Improvement due to reinforcement increasing

1)Without changing the bearing conditions and the dimensions of the P3 tower, maximum of the reinforcement in section was increased as much as possible.

2)But due to this improvement, the bending moment and shearing force did not satisfy the allowable values. Therefore the cross sectional dimensions of the tower needs to change.

Improvement due to elastic bearing arrangement

1) Due to the distribution of the seismic force, the section force of the tower in fixed side is decrease drastically. But the displacement of the bearing increases and maximum displacement becomes 777mm. The displacement of the bearing at the tower exceeds the allowable displacement. Change of the bearing size is desired.

2)Section forces of the pier in approach bridges increase and bending moments exceeds the allowable values. Without changing the cross-sectional dimensions of the pier, section forces satisfy the allowable values by increasing the diameter of the reinforcement.

Improvement due to isolation bearing arrangement

1) Due to the effect of the hysteresis damping of the isolation bearings, the section forces decrease compared with elastic bearings.

2) Bending moments of the pier in approach bridges decrease too, but the response dose not satisfy the allowable values. But if the diameter of the isolation bearings is increased, it will be possible that the response satisfies the allowable values.

3) The displacement of the isolation bearing becomes small compared with those of the elastic bearing, maximum displacement is 646mm. But the displacement of the bearing dose not satisfy the allowable displacement at the tower. By increasing the area of the lead and upgrading the damping effect, the displacement can decrease furthermore.

Improvement due to damper arrangement

1) Due to the effect of the hysteresis damping of the damper, section forces decrease compared with the case using elastic bearings at all supports.

2) Bending moments of the piers in the approach bridge decreased too, and satisfied the allowable bending moment.

3) The displacement of the bearing decreased dramatically compared with the isolation bearings and elastic bearings, maximum displacement becomes 272mm. The displacement of the bearings exceeds the allowable displacement. This displacement will be satisfied the allowable displacement by increasing the size of the bearing.

In this paper, it is assumed that composite cable stayed bridge with 2 edge girders which is superior to cost performance will be constructed in Japan. At the same time, to know the improvement method of the seismic performance, actually constructed bridge in U.S. was selected as the model. From the results of nonlinear dynamic analysis, dynamic behavior of this type of bridge becomes clear. The isolation bearing and the damper are effective for improving the seismic performance of cable stayed bridges. Especially obvious improvement of seismic performance is expected by using the damper.

As the result, it became possible that the composite cable stayed bridge with 2 edge girders designed by using the small seismic force in U.S. satisfied the seismic performance in Japan without changing the cross-sectional dimensions of the pier and the quantity of the reinforcement. Furthermore, when this type of bridge is designed by using the seismic design specifications of Japan, the size of the piers and the tower dose not become drastically big and cost performance will not lose. By using the isolation bearing and the damper, the cost efficiency of the composite cable stayed bridges increases all the more.

Dimensions of the bridge in this paper was quoted from public relation magazine「NETWORK SUMMER 1994」of PB Consultant in U.S..

REFERENCES

1) Suzuki, Tanaka ,"A propose of 3-span continuous hybrid cable stayed bridge", Proceeding of the 56th Annual Conf. of JSCE, I -A218, 2000.9.,pp436-437.

2) Suzuki, Tsuchida, "Structural Characteristic and Economic Superiority of Hybrid Cable Stayed Bridge with Two-Edge Girders", Journal of Constructional Steel Vol.8, Nov.2000, Japanese society of Steel Construction

2000.11.,pp587-594
3) Otsuka, Suzuki, “Seismic Analysis of cable-stayed bridge with 2 girders with high pier”, Proceedings of the 56th annual Conf. of JSCE, Ⅰ-A299, 2001.10.,pp598-599
4) Y.SUZUKI, H.Otsuka, A.Otsuka ,“Investigation of earthquake resistance and cost effect on hybrid cable-stayed bridge with 2-edge girders” ,Earthquake Resistance Engineering StructureⅢ, 2001.9, pp651-660
5) Otsuka, Suzuki ,”A proposal of Improvement on Earthquake-Resistant for Composit Cable-Stayed Bridge with 2 Edge Girders and PC Ramen Bridge” ,Journal of Structual Engineering ,JSCE Vol.49A ,2003.3.