SEISMIC DESIGN OF MULTI SPAN CONTINUOUS RIGID-FRAME BRIDGE WITH PRESTRESSED CONCRETE BOX GIRDER —NEW-TOMEI GUNKAI-GAWA BRIDGE—

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Abstract

The Gunkai-gawa Bridge is a seven-span continuous rigid frame box girder bridge made of prestressed reinforced concrete, which will be 740 m long and located near the Toyota-higashi Junction of the New Tomei Expressway. It was designed to have strong earthquake-resisting capacity and simple maintenance, by means of rigid-coupling construction between all piers and main girders. There are three techniques: first, lightweight upper structure, second below-grade footings for short piers, and the last horizontal stress application for displacement adjustment during closure of the main girders. This paper presents an overview of each process for the seismic design and the approach to construction of the bridge.

1. Introduction

The Gunkai-gawa Bridge will be located 3 km east of the Toyota-higashi junction on the New Tomei Expressway. The bridge will be 740 m long, a seven-span continuous rigid frame box girder bridge made of prestressed reinforced concrete. The building construction work of this bridge was ordered including designing and planning to construction. This ordering system enables us to require structural feasibility, of course, simple maintenance, consideration for surrounding environment, and what is more, shortened work schedule. The selected design was the bridge emphasized simple maintenance having no supports due to achieving a rigid-coupled prestressed reinforced concrete structure between all of the piers and main girders. Generally it is difficult to have rigid-frame bridge because the height of the bridge piers are lower in comparison to a span of 740 m long bridge. Here, there are three methods enabling the structure feasible:
(1) Lightweight superstructure
(2) Below-grade footings applied to short piers
(3) Horizontal stress application method for displacement adjustment during closure of the main girders

This paper presents an overview of the process and its each issue through which the structural form of the Gunkai-gawa Bridge was determined. It also treats the

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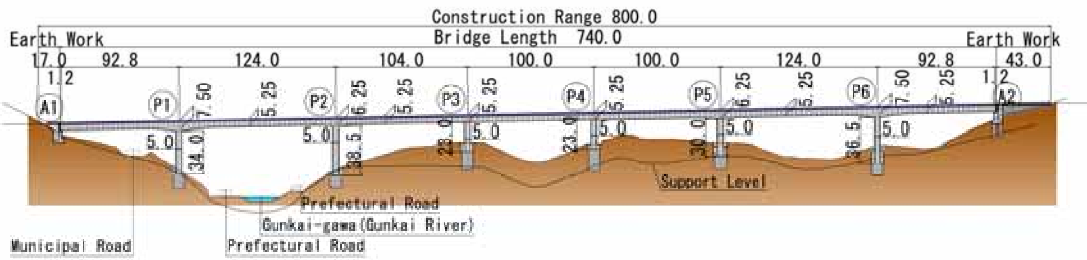
approach to construction of the bridge.

2. Overview of Gunkai-gawa Bridge

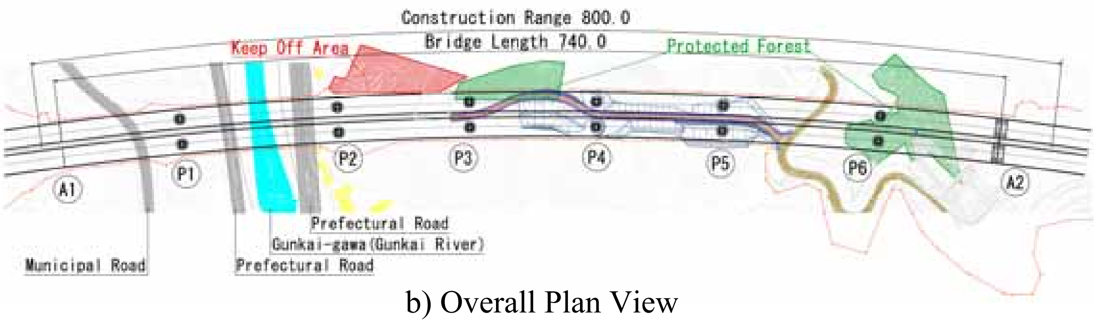
The control points of the Gunkai-gawa Bridge are the Gunkai-gawa (Gunkai River) and two prefectural roads, located along each bank of the river. Trying to save the span crossing these control points 100 m level, a bridge pier would need to be placed between them and the A1 abutment. Accordingly, the length of the span crossing the river was made 124 m in order to reduce the number of bridge piers. It was also determined that using 100 m spans for the other sections would further reduce the number of bridge piers. Consequently, a seven-span structure is confirmed optimal. Since simple maintenance was also required as a designing performance, concrete girders were used throughout the construction, and a rigid frame structure was used for all bridge piers in order to reduce the number of bearings.

As a result of the establishment of these constraints and the determination of the bridge pier and abutment positions based on further consideration of economy, simple maintenance and high construction performance, the bridge structure was finalized as a prestressed concrete seven-span continuous rigid frame box girder bridge measuring 740 m long. (Fig. 1)

The ratio of fixed span length to bridge pier height of this bridge is approximately 8:1. Considering the ratio in an ordinary rigid frame bridge is 5:1 or less, the bridge piers will be affected a major impact from the expansion and the contraction of the main girders. Therefore, the structure was achieved through the incorporation of measures such as a lightweight superstructure, below-grade footings for short piers, and horizontal stress application for displacement adjustment prior to closure of the



a) Side View



b) Overall Plan View

Fig.1 General View of Gunkai-gawa Bridge cantilever erection for the outermost piers.

As shown in Fig. 2, the main girders have a wide sectional shape, so a single-cell box girder section with an extended slab reinforced with struts was adopted. Moreover, the use of a diagonal web enabled the main girder weight to be reduced by 18% as compared to a conventional single-cell box girder section. Long sections with the same girder height were provided and struts of the same length were used, greatly increasing the ease of construction. Moreover, to reduce the number of external tendons within the narrow box girders, high-strength prestressing strands were used. The main girders were designed using the limit state design method, and concrete with a design standard strength of $\sigma ck = 50\ N/mm^2$ was used in places to reduce the weight.

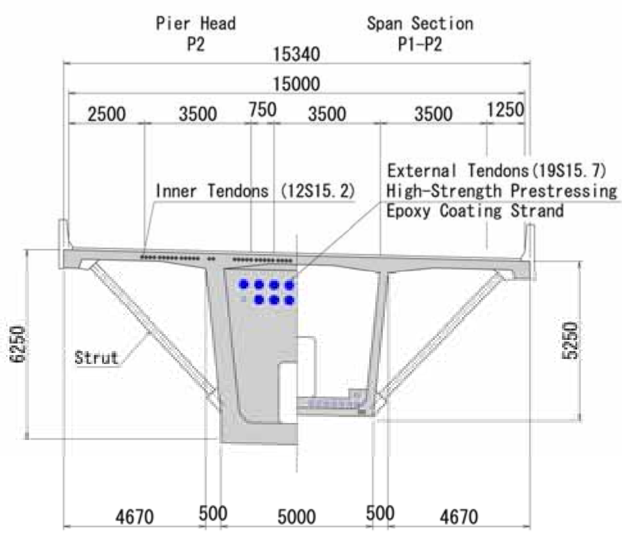


Fig.2 Main Girder Cross Section

The design strength of the bridge pier concrete was $\sigma ck = 30\ N/mm^2$ ($40\ N/mm^2$ in some sections), and SD 345 was used for the reinforcements. The types of foundation used were a large diameter caisson foundation for all bridge piers, a spread foundation for the A1 bridge abutment, and a caisson pile foundation for the A2 abutment. As shown in Fig. 3, shapes of all bridge piers were 5.0 x 5.0 m hollow section, in which D51 (SD 345) in two planes at a maximum as axial reinforcements and D29 hoop ties were provided.

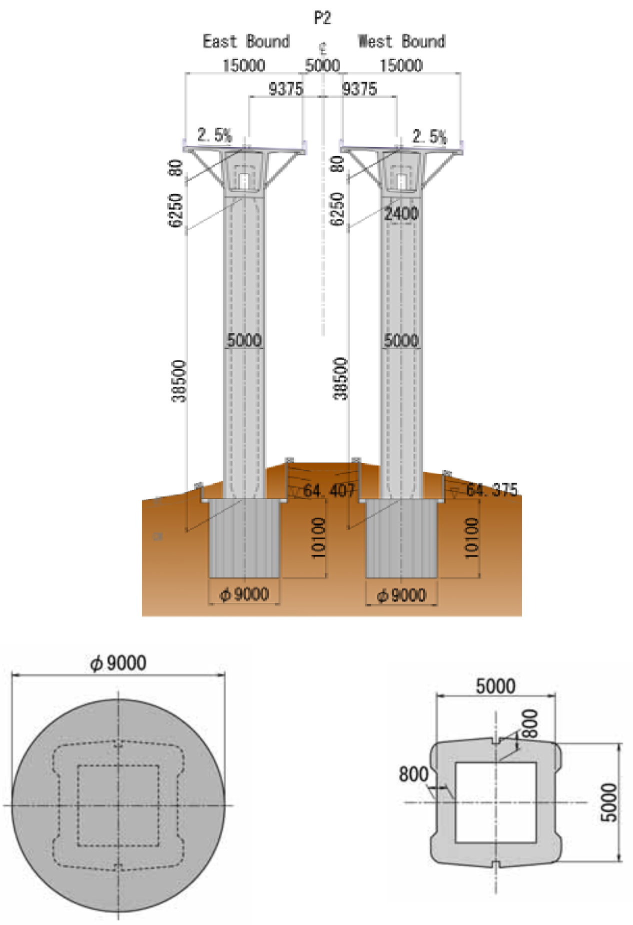


Fig.3 Pier Cross Section

3. Seismic design

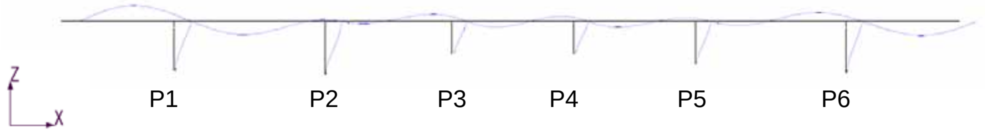
Two stages of ground motion, Level 1 and Level 2, were considered in the seismic design of the bridge. As the bridge was a continuous rigid frame bridge and the 1st vibration mode would be dominant, the static check method was used as the method of checking earthquake-resisting capacity with respect to a Level 1 ground motion. As adequate study has not been conducted for the applicability of the equal energy assumption based on the non-linear hysteretic behavior of the structural members and the bridge as a whole, the active check method was used for the Level 2 ground motion.

3-1. Analysis model and analysis method

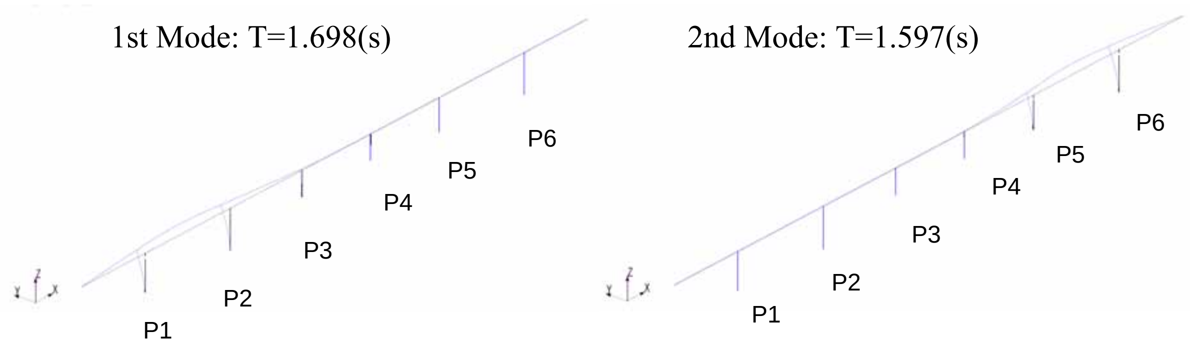
A three-dimensional framework model created by modeling the bridge piers and main girders as bar elements was used as the analysis model. The A1 and A2 ends are support structures, so spring support was used for the bridge axial direction (X direction) and vertical direction (Z direction). As a displacement limit structure was employed for the perpendicular direction (Y direction), fixed support was used for this direction. For the bottom end of each bridge pier, a spring constant was considered for the large diameter caisson foundation. The analysis took into account material non-linearity for all members (M-φ model).

3-2. Eigenvalue analysis

Fig. 4 shows the major natural periods and vibration modes for the bridge. For the bridge axial direction, the 1st mode was dominant, and the natural period was 0.948 seconds. In the direction perpendicular to the bridge axis, the 1st and 2nd modes were dominant. In each of these modes, the main girder with the longest span length (124



a) Axial Direction (X direction)



b) Perpendicular Direction (Y direction)

Fig.4 Vibration Mode

m) was deformed in the direction perpendicular to the bridge axis. The natural periods were 1.698 and 1.597 seconds, respectively.

3-3. Check of Level 1 ground motion

Under Level 1 ground motion, the design horizontal seismic coefficient was determined by the static check method. That is, the value was kH=0.2 calculated from its own natural period T=0.948(s) in the bridge axial direction, and the value was kH= 0.15 with natural period T = 1.698 (s) in the direction perpendicular to the bridge axis.

Table 1 shows the results of the check for the bridge piers for the bridge axial direction and the direction perpendicular to the bridge axis. In the bridge axial direction, piers P3 and P4 in the center had a lower height than the other bridge piers. The section force during an earthquake will be concentrated on these lower piers, which will be subjected to the most intense reinforcement stress.

In the direction perpendicular to the bridge axis, as noted in the former section about natural period modes, the dominant mode was the one in which the main girder with the longest span length of 124 m was deformed in the direction perpendicular to the bridge axis. Hence the bridge pier supporting this span, P2 and P3, will be subjected to substantial reinforcement stress.

Table1. Pier Check of Level 1 Ground Motion

		P1	P2	P3	P4	P5	P6
Design Strength of Concrete		40N/mm^2	30N/mm^2	30N/mm^2	30N/mm^2	30N/mm^2	40N/mm^2
Axial Reinforcing Bar	Axial Direction	SD345 D51@150 2 line	SD345 D51@150 2 line	SD345 D51@150 2 line	SD345 D51@150 1.5 line	SD345 D51@150 2 line	SD345 D51@150 2 line
	perpendicular direcion	SD345 D51@150 2 line	SD345 D51@150 2 line	SD345 D51@150 1 line	SD345 D51@150 1 line	SD345 D51@150 2 line	SD345 D51@150 2 line
Hoop tie	Axial Direction	D29@150	D29@150	D29@125	D29@150	D29@150	D29@150
	perpendicular direcion	D29@150	D29@150	D29@125	D29@150	D29@150	D29@150
Axial Direcion	σc(N/mm^2)	13.2	11.9	14.1	13.3	11.8	13.7
	σa(N/mm^2)	19.5	15.0				19.5
		O.K.	O.K.	O.K.	O.K.	O.K.	O.K.
	σs(N/mm^2)	214	189	284	270	194	228
	σa(N/mm^2)	300					
		O.K.	O.K.	O.K.	O.K.	O.K.	O.K.
Perpendicular Direcion	σc(N/mm^2)	11.7	11.3	10.2	8.8	11.0	10.5
	σa(N/mm^2)	19.5	15.0				19.5
		O.K.	O.K.	O.K.	O.K.	O.K.	O.K.
	σs(N/mm^2)	177	172	176	142	171	145
	σa(N/mm^2)	300					
		O.K.	O.K.	O.K.	O.K.	O.K.	O.K.

σc:Concrete Stress
σs:Reinforcing Bar Stres
σa:Limit

3-4. Check of Level 2 ground motion

The required earthquake-resisting capacity with respect to a Level 2 ground motion was based on Specifications for Highway Bridges (Part V: Seismic Design). The seismic design was conducted so as to satisfy earthquake-resisting capacity 2 (which means to be in limited damage resulting from an earthquake Level 2 which can be restored temporary bridge performance quickly). The input ground motion used for the check was an acceleration waveform based on Specifications for Highway Bridges (Part V: Seismic Design).

(1) Check of bridge piers

The bridge piers were figured out to have enough performance by checking in three items in the bridge axial direction and the direction perpendicular to the bridge axis: (1) maximum response curvature at top and bottom ends (2) shear capacity, and (3) residual displacement. The results are shown in Table 2. The values for each check item satisfied the requirements for allowable limits.

Table2. Pier Check of Level 2 Ground Motion

				Unit	P1	P2	P3	P4	P5	P6
(1)Maximum Respons Curvature	Axial Direcion	Type I	response	(rad)	1.5E-03	1.7E-03	3.7E-03	4.0E-03	2.0E-03	1.5E-03
			limit	(rad)	4.4E-03	4.3E-03	4.9E-03	4.8E-03	4.4E-03	4.0E-03
			ratio		35%	39%	76%	84%	47%	37%
					OK	OK	OK	OK	OK	OK
		Type II	response	(rad)	2.0E-03	2.0E-03	6.2E-03	5.9E-03	2.7E-03	1.7E-03
			limit	(rad)	1.3E-02	1.2E-02	1.7E-02	1.6E-02	1.3E-02	1.0E-02
			ratio		16%	16%	36%	36%	21%	16%
					OK	OK	OK	OK	OK	OK
	Perpendicular Direcion	Type I	response	(rad)	3.1E-03	3.8E-03	1.6E-03	1.7E-03	4.3E-03	3.3E-03
			limit	(rad)	4.5E-03	4.4E-03	4.2E-03	4.4E-03	4.5E-03	4.1E-03
			ratio		68%	87%	38%	38%	95%	82%
					OK	OK	OK	OK	OK	OK
		Type II	response	(rad)	1.5E-03	1.7E-03	1.4E-03	1.5E-03	2.1E-03	1.6E-03
			limit	(rad)	1.3E-02	1.3E-02	1.3E-02	1.4E-02	1.3E-02	1.1E-02
			ratio		11%	13%	11%	11%	16%	15%
					OK	OK	OK	OK	OK	OK
(2)Shesr Capacity	Axial Direcion	Type I	response	(kN)	2.7E+04	2.6E+04	3.6E+04	3.4E+04	3.4E+04	2.5E+04
			capacity	(kN)	3.7E+04	3.7E+04	4.4E+04	3.7E+04	3.7E+04	3.0E+04
			ratio		72%	71%	80%	92%	92%	83%
					OK	OK	OK	OK	OK	OK
		Type II	response	(kN)	3.1E+04	2.8E+04	3.8E+04	3.6E+04	3.6E+04	2.7E+04
			capacity	(kN)	3.8E+04	3.8E+04	4.5E+04	3.8E+04	3.8E+04	3.0E+04
			ratio		81%	74%	84%	96%	95%	89%
					OK	OK	OK	OK	OK	OK
	Perpendicular Direcion	Type I	response	(kN)	1.9E+04	1.8E+04	1.7E+04	1.8E+04	2.1E+04	1.9E+04
			capacity	(kN)	3.7E+04	3.7E+04	4.4E+04	3.7E+04	3.7E+04	3.0E+04
			ratio		51%	47%	39%	48%	56%	63%
					OK	OK	OK	OK	OK	OK
		Type II	response	(kN)	1.5E+04	1.5E+04	1.6E+04	1.8E+04	2.0E+04	1.7E+04
			capacity	(kN)	3.8E+04	3.8E+04	4.5E+04	3.8E+04	3.8E+04	3.0E+04
			ratio		40%	40%	35%	48%	53%	55%
					OK	OK	OK	OK	OK	OK
(3)Residual Displacement	Axial Direcion	Type I	response	(m)	0.033	0.030	0.026	0.025	0.026	0.028
			limit	(m)	0.400	0.432	0.267	0.267	0.347	0.425
					OK	OK	OK	OK	OK	OK
		Type II	response	(m)	0.052	0.049	0.044	0.041	0.042	0.044
			limit	(m)	0.400	0.432	0.267	0.267	0.347	0.425
					OK	OK	OK	OK	OK	OK
	Perpendicular Direcion	Type I	response	(m)	0.060	0.064	0.015	0.016	0.082	0.063
			limit	(m)	0.400	0.432	0.267	0.267	0.347	0.425
					OK	OK	OK	OK	OK	OK
		Type II	response	(m)	0.000	0.000	0.003	0.010	0.020	0.000
			limit	(m)	0.400	0.432	0.267	0.267	0.347	0.425
					OK	OK	OK	OK	OK	OK

(2) Check of main girders

In the check conducted for the bridge axial direction, it was confirmed that the response bending moment in the in-plane direction acting on the main girder was within the initial yield moment of the axial direction reinforcements in both the top of the upper slab and the bottom of the lower slab (Fig. 5). To resist positive bending, the axial direction reinforcements in the bottom slab were upgraded throughout the entire length of the bridge, which were replaced up to D25 at 125 mm pitch. In addition, negative bending was increased at side spans according to vibration mode in Fig.4 a). That is due to the great length of the side spans at both ends (92.8 m) and the fact that the ends were supported by bearings. So the axial direction reinforcements for the top slab near the center of the span were also upgraded to D25 at intervals of 125 mm.

The check conducted for the direction perpendicular to the bridge axis confirmed that the response bending moment in the out-of-plane direction acting on the main girder was within the initial yield moment of the axial direction reinforcements on the outside of the web.

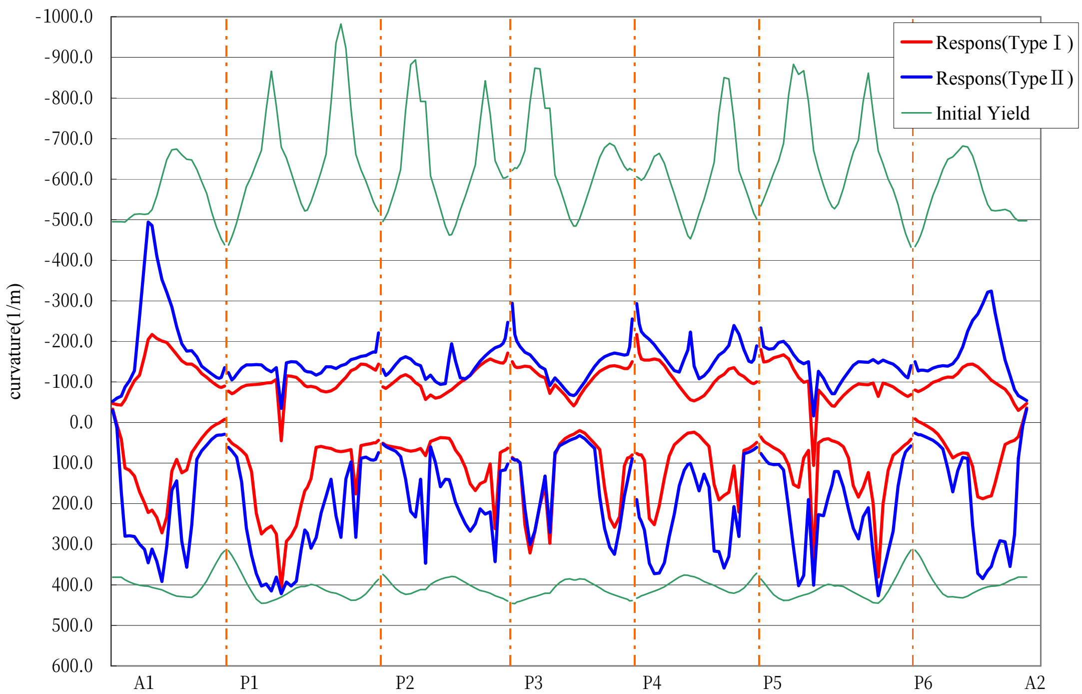


Fig.5 Maximum Response Curvature on the Main Girder of Level 2 Ground Motion

4. Approach to future construction

On this bridge, the eastbound and westbound lines are separated, but as access is restricted in some areas, the eastbound lanes will be constructed first. The major overall processes for the construction are as follows:

1. Ground leveling for construction road and work yard
2. Caisson foundation construction
3. Bridge pier construction
4. Pier caps

5. Cantilever erection work
6. Bridge surfacing

The cantilever erections of the main girders will be built up at once for all bridge piers to shorten the construction time.

Below is a description of the below-grade footings for short piers and the horizontal stress application method for displacement adjustment during closure of the main girders, methods that were adopted to achieve the characteristic rigid frame structure of the bridge.

(1) Below-grade footings for short piers

Below-grade footings are used for short piers to achieve the continuous rigid frame structure. On lower piers P3 and P4, the top surfaces of the foundations are placed below the surface of the ground to adjust the pier height. There are kept hollow with covers without backfilling between these piers and ground to prevent the pier from being subjected to ground resistance in the event of an earthquake. The cover can be removed and steps are hung to enables us to descent to the base of the pier for inspection after an earthquake. (see Fig. 6).

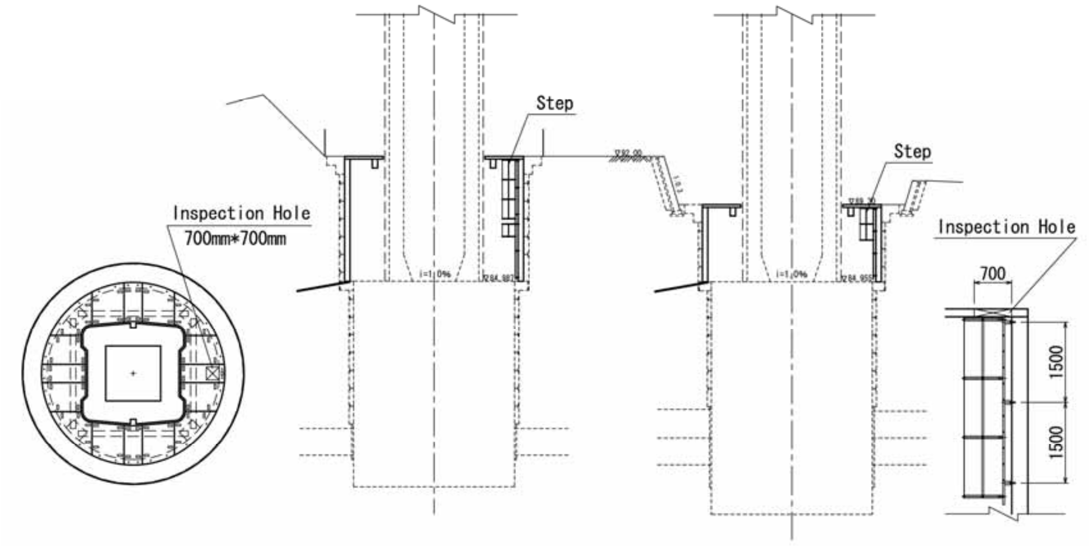


Fig.6 Below-Grade Footings for Short Piers

(2) Horizontal stress application for displacement adjustment

As this bridge has a long fixed span, the creep and dry shrinkage of concrete that may be large after the main girders have been completed will increase the bending moment applied to the end piers. Normally, under the condition of the bridge being subjected to dead load and temperature fluctuations, the section at the base of the end piers must be larger than those of the other bridge piers. As a result, the foundation structure of the end piers must also be larger.

In order to avoid this, before the main girders between end piers P1, P6 and the next pier P2, P5 are closed, horizontal force is applied to reduce the bending moment that acts on the end piers. It enables to reduce the section force about 28% and to keep the reinforcement stress within the acceptable limit. Fig. 7 shows a conceptual diagram of the adjustment method.

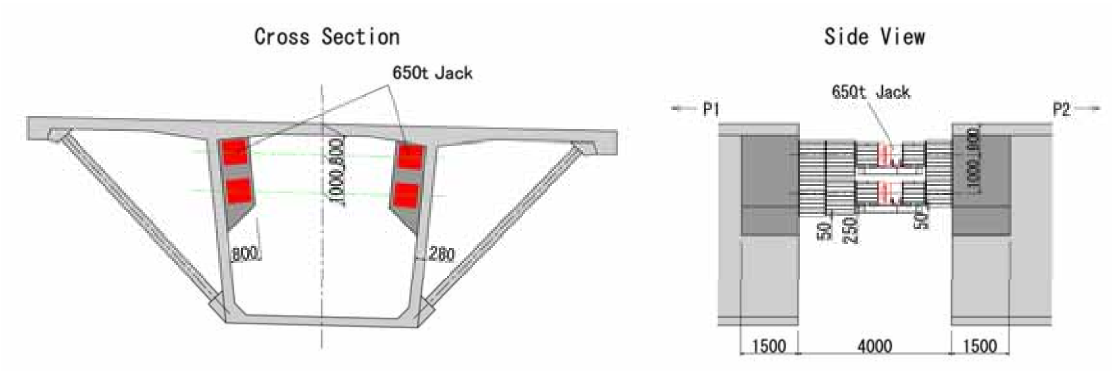


Fig.7 Horizontal Stress Application

5. Conclusion

This paper has described the structural form adopted for the Gunkai-gawa Bridge, which was ordered as a design and build package project. There were three methods used to achieve the structure of this bridge. An overview of the seismic design of the structure was also presented. There are few examples of the construction of long, continuous rigid frame structures such as this bridge that also has a low bridge pier height. It was confirmed that the bridge which has a rigid-coupled structure that emphasizes simple maintenance also provides excellent earthquake-resisting capacity.

As of September 2009, construction of the Gunkai-gawa Bridge was at the stage of leveling the land for the construction road, so the work has just begun. A detailed construction plan will be drafted for the completion of the bridge, and thorough attention will be given to quality control and safety management.