STUDY ON SEISMIC RETROFIT OF THE BRIDGE IN THE HONSHU-SHIKOKU EXPRESSWAY USING ISOLATION BEARINGS

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Abstract

Seismic retrofit work for the bridges in the Honshu-Shikoku Expressway has recently been launched because there is a concern that seismic risk for the bridges increases. In this paper, an example of seismic retrofit using isolation bearings is described. The Innosima Higashi Viaduct, a three-continuous steel truss bridge, turned out not to own enough seismic performance. After a study for seismic performance improvement considering seismic isolation and energy dissipation, replacement of existing steel bearings to seismic isolation bearings was selected as the seismic retrofit measures, which costs less than strengthening the truss members.

1. Outline of Seismic Retrofit for the Honshu-Shikoku Bridges

The Hyogo-ken Nanbu Earthquake in 1995 had brought about a great change in the Japanese seismic design. The earthquake led the Japanese seismic design code for highway bridges to be revised drastically in 1996 and to consider two levels of design seismic motions; seismic motion Level 1 corresponds to an earthquake with high probability of occurrence during the bridge service life, and seismic motion Level 2 corresponds to an earthquake with less probability of occurrence during the bridge service life but strong enough to cause critical damages. Furthermore, seismic motion Level 2 is classified into 2 types; one is a seismic motion generated by an plate-boundary earthquake with a large magnitude (Type I); the other is a seismic motion generated by an inland near-field fault earthquake such as the Hyogo-ken Nanbu Earthquake(Type II), which had not been clearly considered until 1996 in Japan. The target seismic performance of bridges should be selected from 3 performance level, no damage, some damage of easy functional recovery and no collapse, depending on the seismic motion levels and the importance of a bridge.

On the other hand, the Honshu-Shikoku Bridges were designed based on the original design codes, since the Seto Inland Sea region, where the Honshu-Shikoku Bridges were located, had suffered from very severe natural conditions such as typhoon or earthquake so far. For the seismic design, site-specific design seismic motions were determined in consideration of the information regarding earthquake histories, earthquakes occurring in the plate-boundaries and geotechnical conditions around the bridge site. The

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design codes, however, were developed before the Hyogo-ken Nanbu Earthquake; therefore inland near-field fault earthquake such as the Hyogo-ken Nanbu Earthquake (Level 2, Type II) was not considered in the design seismic motions. In addition, information on inland active faults around the bridge site has been recently clarified by the geological investigations vigorously performed after the Hyogo-ken Nanbu Earthquake. According to the information, occurrence of a strong seismic motion exceeding the design seismic motions is estimated in some bridge sites when the assumed earthquake occurs.

Above mentioned background motivated us to commence seismic retrofit work for the Honshu-Shikoku Bridges, which are non-redundant bridges between Honshu and Shikoku Island, as shown in Fig.1. Since the bridges are designated as lifeline corridors for emergency transportation or restoration works immediately after an earthquake, the bridges after a large-scale earthquake are required to be within limited and repairable damage for the immediate opening of the bridge to emergency vehicles and for the easy recovery. Therefore we had started to evaluate seismic vulnerabilities for the bridges in chronological order of design and study the seismic performance improvement for the bridges with insufficient seismic performance. Since cost efficiency is our crucial issue, seismic retrofit measures should be carefully selected, and one possible solution for effective seismic upgrading is to adopt anti-seismic devices, such as isolation and dissipation devices.

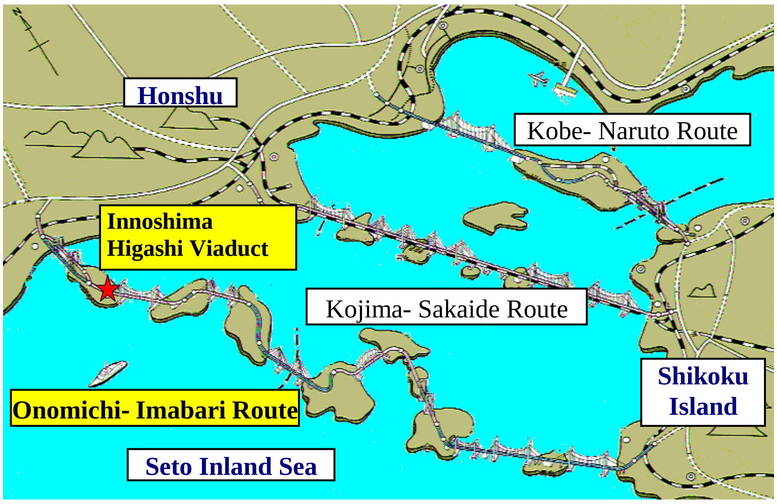


Fig. 1 Geographical location of Honshu-Shikoku Bridges

Seismic retrofit work for Honshu-Shikoku Bridges including long-span bridges has been launched and will be completed within the decades to come.

2. Seismic Assessment of Existing Bridge

The Innoshima Higashi Viaduct on the Onomichi-Imabari Route is a 3-span continuous steel truss bridge with bridge length of 158m and with maximum span length of 59m as illustrated in Fig.2. The bridge is one of the oldest bridges in the Honshu-Shikoku Expressway and was designed by the design specification of 1973. The bridge is located next to the Innoshima Bridge which is a suspension bridge with a center span length of 710m. The substructures are RC double-layer rigid-frame type with relatively tall height and their foundations are direct foundation type. Although the superstructure is supported at 1A, 2P and 3P in the longitudinal direction, inertial force during an earthquake is mainly supported at 1A, which has higher horizontal stiffness than 2P and 3P.

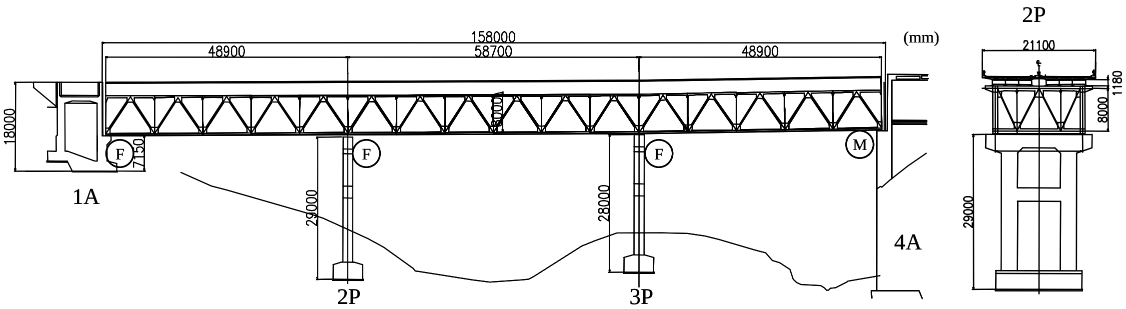


Fig. 2 General View of Innoshima Higashi

For seismic assessment of the existing bridge against the seismic motion Level 2, as shown in Fig. 3, nonlinear time history analyses were conducted with an analytical model as shown in Fig 4. The input seismic motions are corrected considering soil profile at the bridge site. In the analytical model, nonlinear characteristics of the piers are incorporated, whereas other structural members are represented by linear models. The stiffness of RC slab was considered in the stiffness of upper lateral bracings. Table 1 and Table 2 show the results of the seismic vulnerability evaluation for the substructure and the superstructure respectively. The seismic performance of superstructure is verified by yield strength for tension force and buckling strength for compression force respectively. The seismic performance of substructures is verified by allowable curvature for flexural force and by shear strength for shear force respectively. The results tell that the bridge does not have an adequate seismic performance against the seismic motion Level 2. For the substructure, all the bearings except 1A bearings in the transverse direction are vulnerable and piers in the transverse direction get serious damage by shear force. For the superstructure, the steel truss members possibly suffer serious damage. The estimated damage of the steel truss is

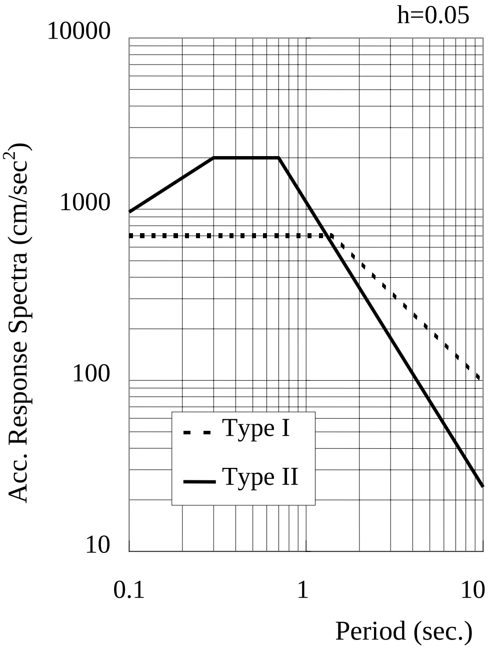


Fig. 3 Seismic Motion Level 2

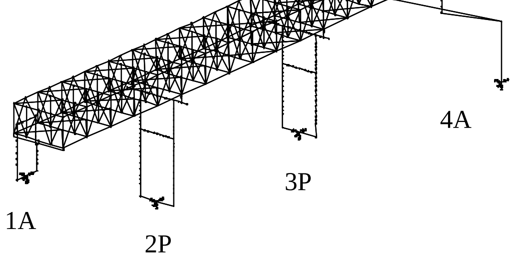


Fig. 4 Analytical model

concentrated on the main chords in the longitudinal direction, and on lateral bracings in the transverse direction. Especially, the maximum response of the lower main chords at 1A, fix support, reaches about 6 times of the buckling strength in the longitudinal direction, as indicated in Fig. 5. This means inertial force due to earthquake concentrates on 1A support in the longitudinal direction.

Table 1 Seismic Vulnerability Evaluation of Existing Bridge (Substructure)

Direction of Input Ground Motion	Evaluation Items	Type of L2 E.Q.	1A		2P		3P		4A	
			Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value	Maximum Response	Allowable Value
Longitudinal	Curvature (10^{-6}/m)	Type I	---	---	1,857	2,781	2,726	2,800	---	---
		Type II	---	---	2,400	5,225	3,772	5,256	---	---
	Shear Force (kN)	Type I	---	---	2,833	3,558	3,415	3,558	---	---
		Type II	---	---	3,883	4,581	*4,348*	*4,228*	---	---
	Reaction Force at Bearing (kN)	Type I	*29,644*	*8,172*	1,725	3,054	1,582	3,054	---	---
		Type II	*34,677*	*8,172*	*3,278*	*3,054*	*3,775*	*3,054*	---	---
Transverse	Curvature (10^{-6}/m)	Type I	---	---	919	2,448	905	2,201	---	---
		Type II	---	---	1,913	3,811	1,863	3,536	---	---
	Shear Force (kN)	Type I	---	---	*9,414*	*3,458*	*10,035*	*3,458*	---	---
		Type II	---	---	*10,420*	*3,954*	*10,950*	*3,954*	---	---
	Reaction Force at Bearing (kN)	Type I	3,736	8,172	*5,390*	*3,054*	*5,878*	*3,054*	*3,287*	*1,414*
		Type II	*8,215*	*8,172*	*7,052*	*3,054*	*7,887*	*3,054*	*7,911*	*1,414*

*) value at the most critical section is described at each

Table 2 Seismic Vulnerability Evaluation of Existing Bridge (Superstructure)

Direction of Input Ground Motion	Truss Member		Quantum of Members (a)	Quantum of Damage Members (b)	Damage Rate (b/a)
Longitudinal	Main Chord	Upper	64	34	53.1%
		Lower	32	20	62.5%
		Vertical	34	0	0.0%
		Diagonal	64	8	12.5%
	Lateral Bracing	Upper	32	0	0.0%
		Lower	32	0	0.0%
	Transverse Truss	Upper	68	0	0.0%
		Lower	68	0	0.0%
		Vertical	34	0	0.0%
		Diagonal	68	0	0.0%
Transverse	Main Chord	Upper	64	0	0.0%
		Lower	32	2	6.3%
		Vertical	34	0	0.0%
		Diagonal	64	0	0.0%
	Lateral Bracing	Upper	32	28	87.5%
		Lower	32	10	31.3%
	Transverse Truss	Upper	68	0	0.0%
		Lower	68	2	2.9%
		Vertical	34	2	5.9%
		Diagonal	68	8	11.8%

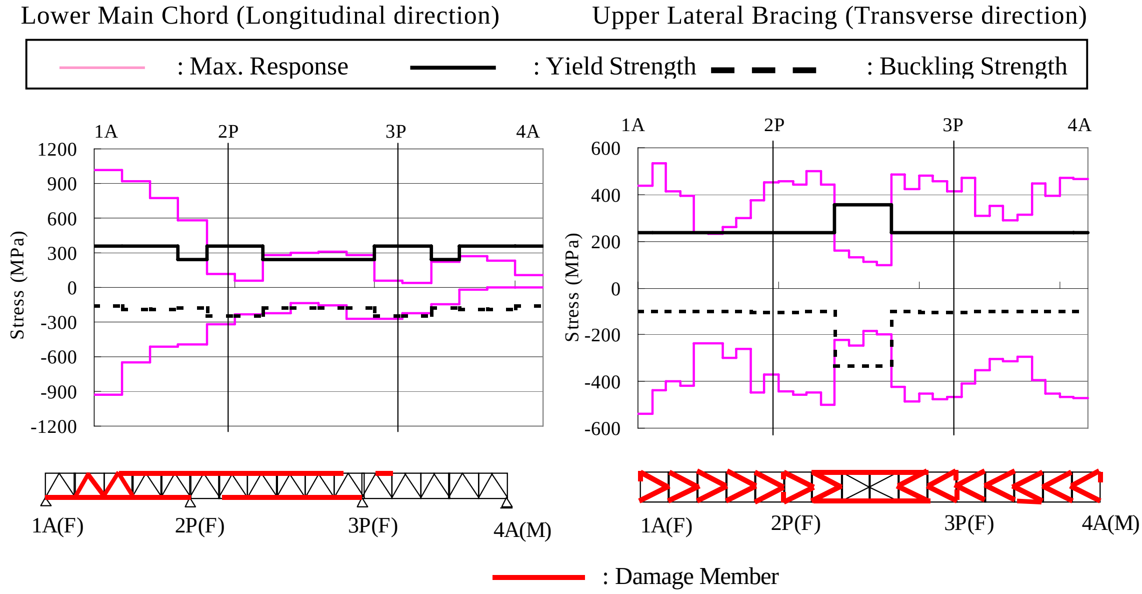


Fig.5 Location of Damaged Steel Truss Member

3. Investigation of Seismic Performance Improvement

For the seismic performance improvement, three methods were evaluated as shown in Table 3. Method (1), strengthening of all vulnerable structural members, is the most expensive; because the most of steel truss members need to be retrofitted and the temporary scaffold is costly. Method (2), installation of viscous dampers, is not the optimized solution, because the dampers have to install in both longitudinal and transverse directions, and unseating prevention system, such as structure limiting excessive displacement, has to also install around bearings. Installation of those devices pushes up cost of the method (2) and makes maintenance of bearings difficult because the piers top become congested. Since a seismic isolation bearing, which is a laminated-rubber bearing as shown in Fig. 6, is effective in both directions, Method (3) becomes the best solution in this case.

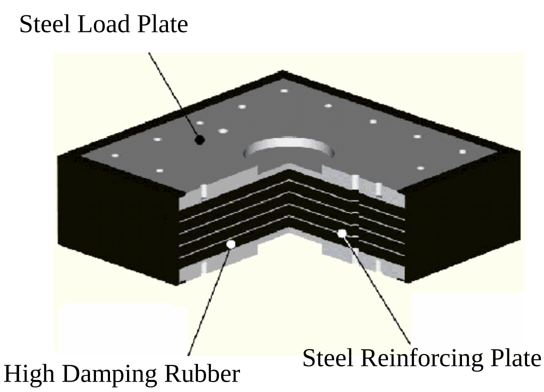
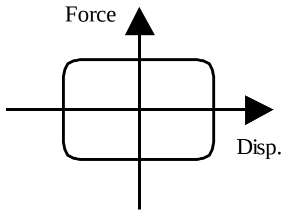
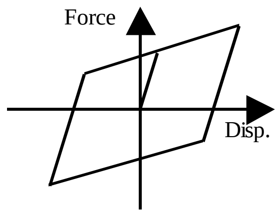


Fig. 6 Seismic Isolation Bearing

Table 3 Comparison of Seismic Performance Improvement Methods

Method		(1) Strengthening of Structural Member	(2) Installation of Viscous Damper	(3) Installation of Seismic Isolation Bearing
Concept		This method is intended to make the structural members stronger by attaching extra materials.	This method is intended to increase the damping characteristics to decrease of the inertial force by installing viscous dampers.  Force vs. Disp. of Viscous Damper	This method is intended to make natural period longer and to increase the damping characteristics to decrease of the inertial force by replacing the existing steel bearings to rubber seismic isolation bearings.  Force vs. Disp. of Seismic Isolation Bearing
Volume of Seismic Retrofit Work	Steel Truss	Strengthening of 122 Members	Strengthening of 5 Members	Strengthening of 3 Members
	Piers	Shear Strengthening of Columns and Cap Beams (2P, 3P)	Shear Strengthening of Columns (2P, 3P)	Shear Strengthening of Columns (2P, 3P)
	Bearings	Strengthening of All Bearings	---	---
	Others	---	Installation of Dampers 1A: 4 Dampers (Long.), 2 Dampers (Trans.) 2P: 2 Dampers (Trans.) 3P: 2 Dampers (Trans.) 4A: 4 Dampers (Long.), 2 Dampers (Trans.)	Replacement of Bearings 1A: 2 Bearings 2P: 2 Bearings 3P: 2 Bearings 4A: 2 Bearings
Cost (Proportion of Method (3))		1.50	1.22	1.00

4. Seismic Retrofit Measures

For a reasonable design of an isolated bridge, the natural period of the bridge should be determined so that while ensuring effective absorption of energy by an isolation bearing, the excessive displacement of superstructure does not adversely affect on the function of the bridge. After performing parametric studies, it is decided that the natural period of the bridge should be elongated from 1.0 sec to 1.5 sec in the longitudinal direction, from 1.0 sec to 1.3 sec in the transverse direction, respectively. Table 4 and Fig. 7, the seismic assessment results by seismic isolation, shows that responses of both superstructure and substructure are reduced substantially and almost all members of superstructure need no strengthening. This is considered to be because damping characteristics are increased and the natural period are made longer. Only 2 diagonal members of end main lateral truss whose stress exceeds the allowable compression stress even after seismic isolation, are decided to be strengthened by steel cover plate, because they support live loads and their failure might affect traffic on the bridge.

Although responses of the substructure are reduced greatly, shear forces of the piers still exceed the allowable values in the transverse direction. After the cost comparison of jackets with RC covers, steel plate covers and epoxy-impregnated carbon fiber sheets for the shear strengthening method of RC piers, jacketing with epoxy-impregnated carbon fiber sheets are selected.

Table 4 Seismic Vulnerability Evaluation with Seismic Isolation Bearing (Substructure)

Direction of Input Ground Motion	Evaluation Items	Type of L2 E.Q.	2P			3P		
			Maximum Response		Allowable Value	Maximum Response		Allowable Value
			Exisiting	Isolated		Exisiting	Isolated	
Longitudinal	Curvature (10^{-6}/m)	Type I	1,857	1,311	2,781	2,726	1,126	2,800
		Type II	2,400	911	5,225	3,772	821	5,256
	Shear Force (kN)	Type I	2,833	2,395	3,558	3,415	2,322	3,558
		Type II	3,883	2,753	4,228	*4,348*	2,870	*4,228*
Transverse	Curvature (10^{-6}/m)	Type I	919	316	2,448	905	279	2,201
		Type II	1,913	566	3,811	1,863	511	3,536
	Shear Force (kN)	Type I	*9,414*	*7,677*	*3,458*	*10,035*	*7,777*	*3,458*
		Type II	*10,420*	*8,804*	*3,954*	*10,590*	*9,011*	*3,954*

*) value at the most critical section is described at each

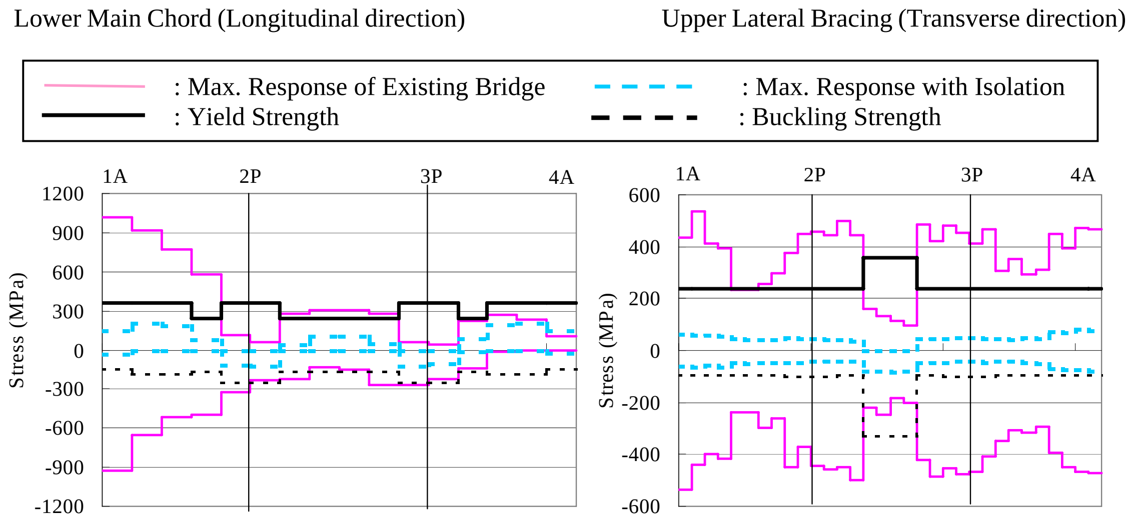


Fig. 7 Effect of Seismic Isolation on Steel Truss Response

Since the isolation bearing can move in all directions, displacement in the transverse direction due to live load or Level 1 earthquake may get damage to the expansion joints. Therefore joint protectors are installed in the isolation bearings in order to prevent the isolation bearings from moving in the transverse direction during live loading or Level 1 earthquake.

5. Execution

Fig. 8 shows the outline on seismic retrofit of the Innoshima Higashi Viaduct.

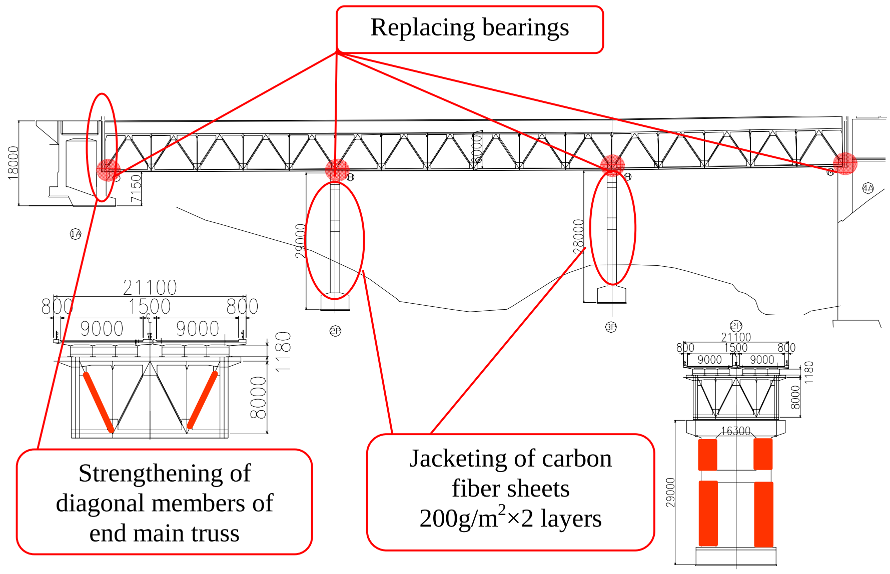


Fig.8 Outline on seismic retrofit of the Innoshima Higashi Viaduct

For the replacement of the bearings, jacking up support points are strengthened by additional gusset plates attached around existing gusset by welding as shown in Fig. 9.

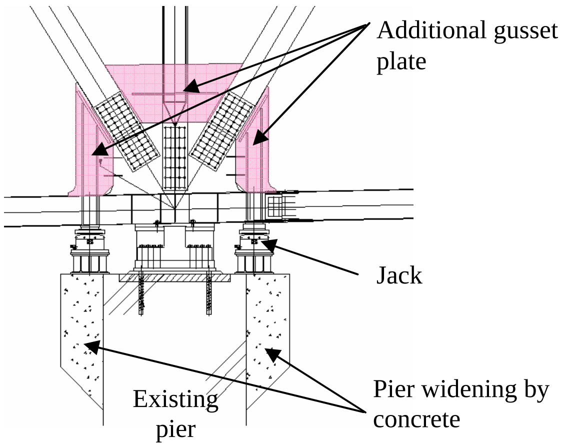


Fig.9 Outline on strengthening of jack up support point

6. Conclusions

(1) Seismic retrofit work for the Honshu-Shikoku Bridges, which are expected to undertake a role as post-earthquake lifelines, has recently been launched and will be completed within some decades. Since cost saving of the seismic retrofit for the bridges is a crucial issue, an introduction of anti-seismic devices, such as isolation and dissipation devices, is considered as one of the reasonable solutions.

(2) A study on the seismic retrofit of a bridge in the Honshu-Shikoku Expressway using isolation bearings is presented in this paper. The costs of seismic retrofit works are extremely reduced by replacing existing steel bearings to high damping rubber bearings.

References

K.Endo, S. Kawabata, S. Ogo ; A Study on Seismic Retrofit of the Honshu-Shikoku Bridges Using Isolation and Dissipation Devices, 2007IABSE, Sep. 2007