SEISMIC VERIFICATION OF LONG-SPAN BRIDGE OF HONSHU-SHIKOKU BRIDGES

Chihiro kawatoh[1], Koji Kawaguchi[2]

Abstract

Honshu-Shikoku Bridge Authority (HSBA) has carried out seismic verification for the very strong ground motion (level2 earthquake) to Long-span bridges which HSBA managed after Hyogoken-Nambu earthquake. This paper presents a policy of the seismic verification including the examples of Akashi-Kaikyo Bridge and Ohnaruto Bridge and indicates principle for seismic retrofit of Honshu-Shikoku Bridges.

1. Introduction

The Hyogoken-Nambu Earthquake, which struck Japan on January 17, 1995, had an epicenter very close to the Akashi-Kaikyo Bridge. A number of investigations into the effects of the earthquake on the bridge were carried out immediately after the event. These checks led to some remedial steps, such as a change in stiffening girder length, but almost no damage to the main structure was found. The bridge was completed successfully with safety and its function was ensured.

On the other hand, long-span bridges over the straits and its approach bridges of Honshu-Shikoku Bridge Authority (HSBA) were designed using seismic design standards of HSBA and seismic design specifications for Highway bridges of 1990 version or specifications before that and were not being taken into consideration as for the strong motion such as a Hyogoken-Nanbu earthquake. HSBA decided that it was necessary to grasp that seismic performance of existing bridges in order to maintain appropriately. Then HSBA selected the target bridges for and have continued out the seismic analysis and verification.

This paper describes the evaluation of the seismic performance of the long-span bridges over the strait, the policy of the seismic retrofit and examples of seismic verification of the Akashi-Kaikyo Bridge and multi-columns foundation of Ohnaruto Bridge.

2. Outline of Honshu-Shikoku Bridges

The Honshu-Shikoku Bridges (HSB), consisting of three routes, connected Honshu-and Shikoku-island across the Seto Inland Sea as follows (from eastern side);
Kobe-Naruto (Kobe-Awaji- Naruto Expressway)
Kojima-Sakaide (Seto-Chuo Expressway & JR Seto Ohhashi Line, often called the Seto Ohashi Bridge)

1. Engineering Information Division, Long-span Bridge Engineering Center, Honshu-Shikoku Bridge Authority
2. Manager, ditto

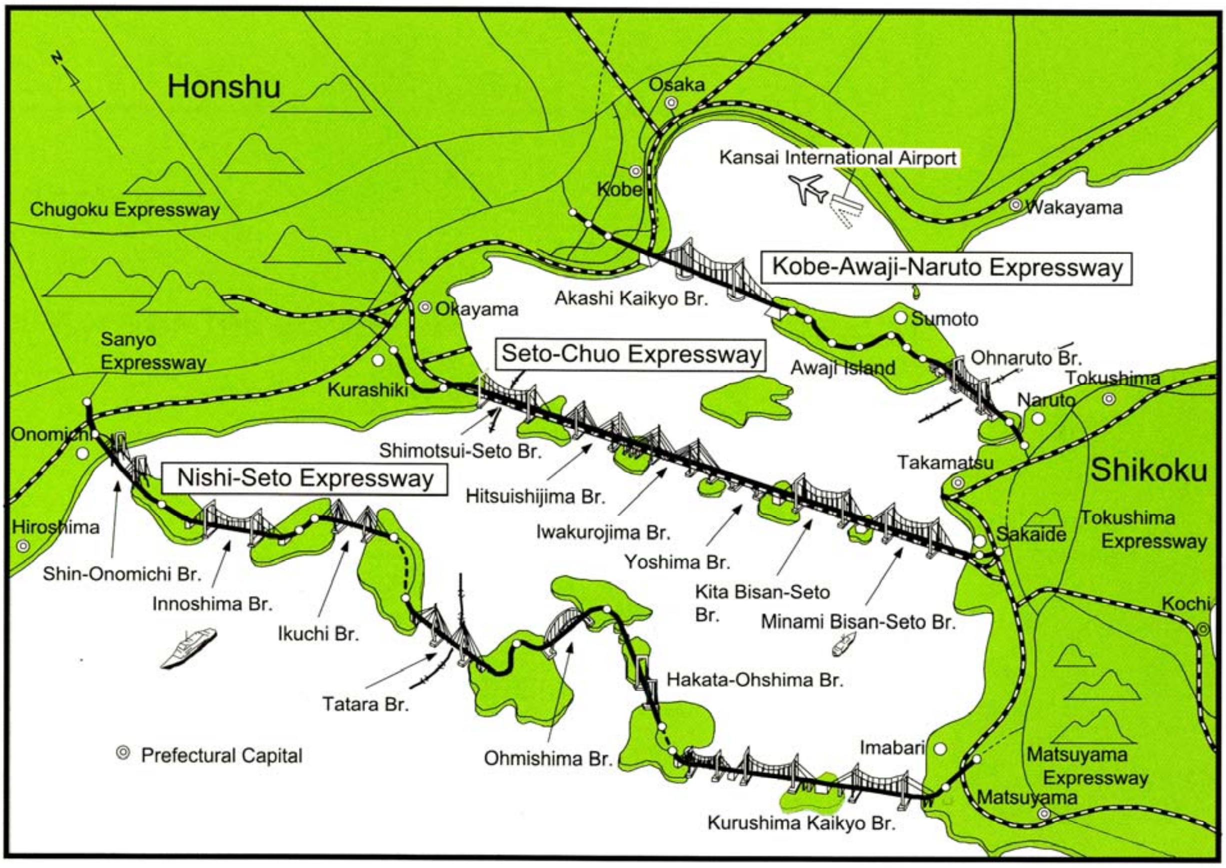

Figure1 Three routes of Honshu-Shikoku Bridges

Onomichi-Imabari (Nishi-Seto Expressway, often called Setouchi-Shimanami Kaido)

The construction of the three routes was finished except about 13 kilometers, two sections located in both Ikuchi and Ohshima islands, on Onomichi-Imabari route. Figure1 shows the three routes of Honshu-Shikoku Bridges.

3. Seismic verification

3.1 Seismic standard and codes for Honshu-Shikoku Bridges

Table-1 shows the seismic standard and codes for HSB [1] [2] [3]. These standards were made for long-span bridges with over 200m-center span. Seismic coeds for Akashi-Kaikyo Bridge and Kurushima-Kaikyou Bridge were special ones, and contained the latest seismic information at that time. But these standards do not reflect the influence of an inland near-field type earthquake (level2 typeⅡearthquake, Seismic Specification for Highway Bridges, Japan Road Association, 1996,[4] here in after JRA). Therefore HSBA decided to verify the seismic performance of bridges for leve2 typeⅡ earthquake.

3.2 Target bridges for seismic verification

HSBA selected bridges over the strait and its approach bridges for seismic verification. The bridges over the straits and its approach bridges should have the same seismic

Table-1 Seismic Standard and Codes for Honshu-Shikoku Bridges

Design Standard		HSBA Seismic Design Standard		Akashi kaikyo Bridge Seismic Design Code		Kurushima Kaikyo Bridge Seismic Design Code	
		Input Earthquake Ground Motion	Seismic Performance Level	Input Earthquake Ground Motion	Seismic Performance Level	Input Earthquake Ground Motion	Seismic Performance Level
Level 1 Earthquake Ground Motion (Highly probable during the bridge service life)		The earthquake of about M=8 that an occurrence is expected 1-2 times in 100years at far out at sea of Kii peninsula or Shikoku island	Not exceeding allowable stress	○ Calculated by statistical theory from historically recorded earthquakes. The acceleration response spectrum has a return period of 150 years. ACC=0.8G	Not exceeding allowable stress	○ Calculated by statistical theory from historically recorded earthquakes. The acceleration response spectrum has a return period of 150 years. ACC=0.75G	Not exceeding allowable stress
Level 2 Earthquake Ground Motion	Type Ⅰ Earthquake Ground Motion (an interplate earthquake)	Type Ⅰ is taken into consideration on small earthquake level		○ Its magnitude and epicenter distance is assumed to be 8.5 in Richter scale and 150km ACC=0.8G	Not exceeding allowable stress	○ Its magnitude and epicenter distance is assumed to be 8.5 in Richter scale and 200km, and another is 8.0 in Richter scale and 150km ACC=0.75G	Not exceeding allowable stress
	Type Ⅱ Earthquake Ground Motion (an inland near-field earthquake)	× Type Ⅱ is not being taken into consideration		Type Ⅱ is taken into consideration on Level 1 earthquake in probability calculation		The same as the left	

performance for the strong ground motion, because there is no alternative route in the over-strait portion.

The number of seismic verification bridges is 44. The total number of seismic verification bridge is 51 which were added the shed structure. Seismic verification of 24 bridges were finished in 51 bridges in substructure and 22 bridges were finished in 53 bridges in superstructure (2 bridges are added on account of superstructure type).

4. Earthquake ground motion

4.1 Level 2 type Ⅰ ground motion

Because HSB are located on the ground with better condition (Vs=600m/s) than type Ⅰ ground(more than Vs=300m/s), earthquake ground motion of site will be different from the characteristic of ground motion defined with Specifications for Highway Bridges.

Based on seismograms recorded at the Kobe Meteorological Observatory of the Japan Meteorological Agency (JMA Kobe), the earthquake wave at the top of the granite bedrock, which was adopted as the seismic bedrock in the seismic design of the bridge, was calculated and used as the input wave for analysis. And for the purpose of the verifying the suitability of this input wave, HSBA calculated the input wave by fault model using the active faults parameters. The strength of acceleration spectrum estimated by fault model was less than the JMA Kobe wave acceleration spectrum. Therefore, HSBA decided JMA Kobe wave as seismic verification wave for the level2 type Ⅱ earthquake.

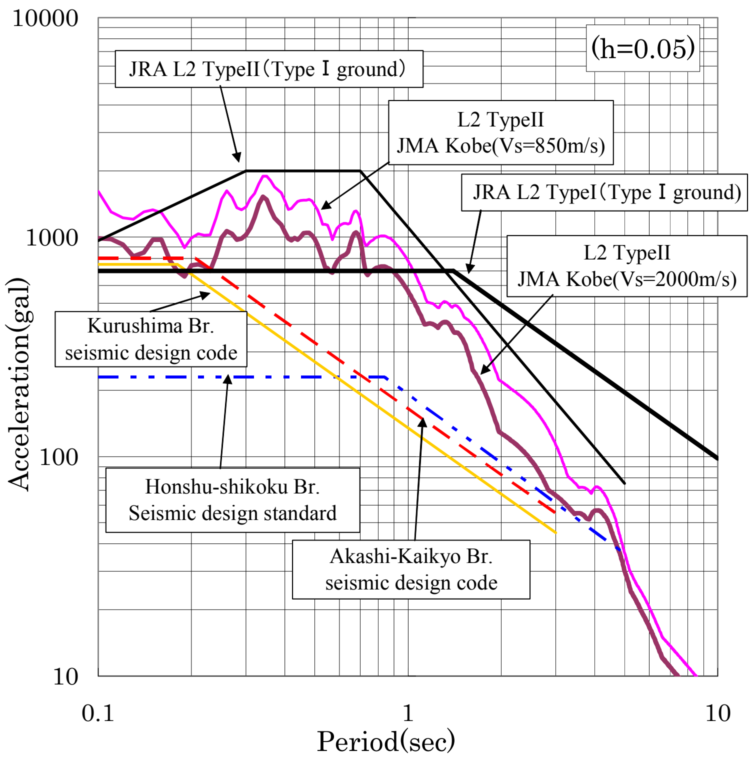

Figure2 Comparison of acceleration of each standard

4.2 Level2 type I ground motion

Earthquake ground motion was calculated by fault model using the Nankai earthquake fault parameters. The calculated acceleration spectrum of the Nankai earthquake was included in the JMA Kobe wave acceleration spectrum. On the other hand, table-1 shows that the level2 type I earthquake is taken into consideration in the seismic design codes of Akashi-Kaikyo Bridge and Kurushima-Kaikyo Bridge as the design earthquake ground motion.

5. Example of seismic verification

5.1 Akashi-Kaikyo Bridge

(1)Outline

The Hyogoken-Nambu Earthquake caused seabed ground movements in the vicinity of the Akashi-Kaikyo Bridge, shown in Fig-3, which was under construction at that time.

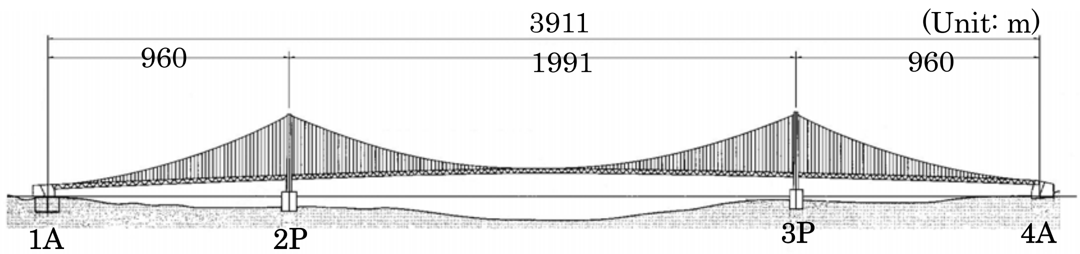

Figure3 General View of Akashi-Kaikyo Bridge after Earthquake

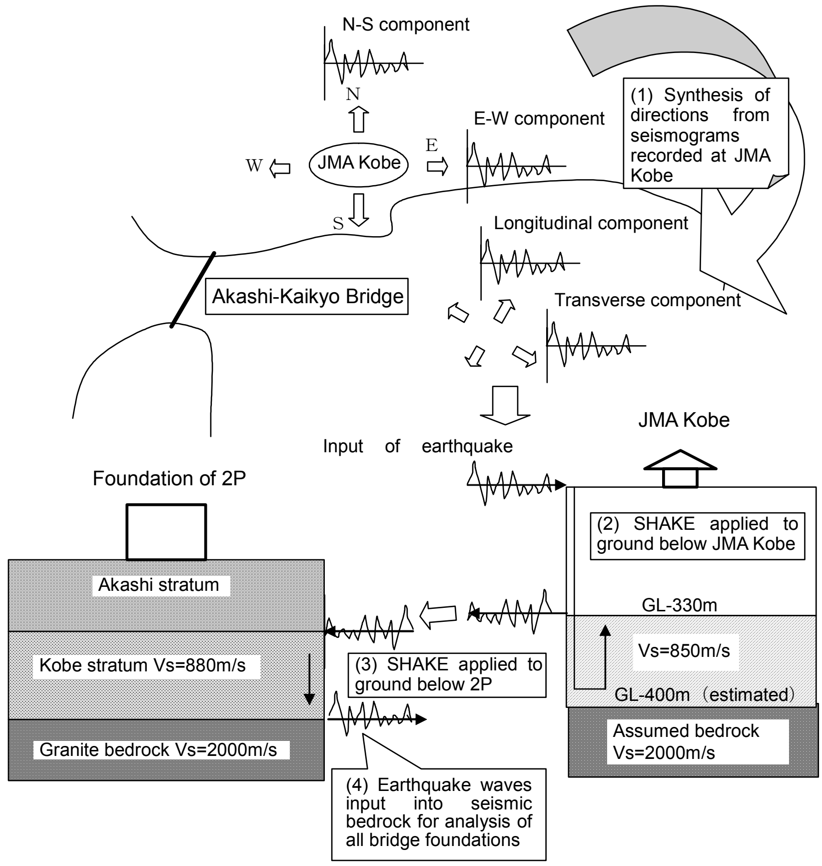

Figure4 Calculation of Earthquake Wave Input into Seismic Bedrock

Since this earthquake occurred directly beneath the bridge, and such an occurrence had not been considered in the original seismic design of this bridge, the HSBA decided to check the bridge in its completed state for resistance to same scale earthquake as the Hyogoken-Nambu Earthquake.

Based on the JMA Kobe earthquake wave, the earthquake wave at the top of the granite bedrock, which was adopted as the seismic bedrock in the seismic verification of the bridge, was calculated and used as the input wave for analysis. Figure 4 shows an outline of this calculation.

(2)Analytical condition

To evaluate that the foundation of the complete bridge are able to withstand earthquake motions on the same scale as those experienced in the Hyogoken-Nambu Earthquake, a time-history response analysis was performed by using the earthquake waves obtained in section 4 and 5.1. Since the bridge foundations would respond nonlinearly to ground motions set up by large earthquake such as the Hyogoken-Nambu Earthquake, a nonlinear time-history analysis of the tower-foundation system using FEM model was performed. Strain dependency of ground and uplift characteristic of foundation were taken into account.

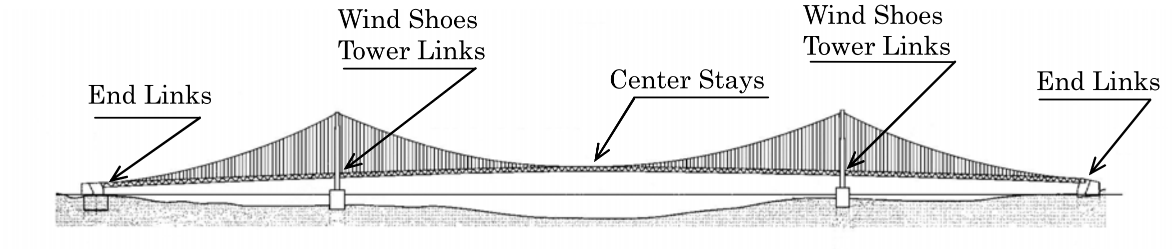

Figure5 Location of Links, Wind Shoes and Center Stays

As same as the foundation, a time-history response analysis was carried out to verify resistance of the superstructure of the complete bridge by using effective earthquake motion. A three-dimensional frame model of the bridge including its foundations was used for the analysis. For damping factor of the superstructure, nonlinear characteristic were not taken into account because it was assumed its response would remain the elastic range. The damping factor of the towers and cables were assumed to be 1%. Further, the damping factors of the girder were assumed to be 2%.

(3)Results of analysis

1) Superstructure

The maximum stresses developed in the tower shaft corner were calculated. These calculations showed that the maximum stresses in 2P and 3P tower would be 434 and 430MPa, respectively, which are below the yield stress value of 451MPa.The axial forces acting on the towers were calculated. Even at minimum, a compressive force of 133MN acted on the base of the east shaft of 2P tower, and there was no tensile force. The maximum axial forces developed in the tower links and end links were within the allowable limit, respectively.

Of the reactions at the wind shoes, the maximum reaction was 22,952kN in the case of the shoe on the central span side of the 2P main tower. Although this value exceeds the limit value of 17,640kN, it seemed to have no effect on the overall stability of the suspension bridge system. The tension induced in hangers and center stays were below the limit values of the original design standards. The relative displacement of the girder ends at the anchorages and towers were found to be within the movable range of the expansion

Table-2 Stability of foundations

	Safety factor for bearing capacity	Safety factor for sliding	Direction of calculation
1A	3.33	4.43	Longitudinal direction
2P	1.67	2.01	Transverse direction
3P	2.27	2.58	Transverse direction
4A	4.39	1.58	Longitudinal direction
Allowable Limit Value	2.00	1.20	-

joints at each point. Figure5 shows the location of links, wind shoes and center stays.

2) Foundations

Based on the forces acting on the base of the foundations, which were obtained from nonlinear time-history analysis of the tower-foundation system using the FEM model, the foundations were checked for stability. As can be seen in table-2, the design safety factors are satisfactory, except at the case of the 2P foundation. Although the safety factor for bearing capacity at 2P foundation is less than the allowable value of the original design code, the 2P foundation is considered to be secured, because the result was obtained by a non-linear FEM time-history analysis method which was considered to be more realistic than the linear FEM time history analysis method used in the original design.

(4)Conclusions

Nonlinear time-history analysis by the FEM model has been applied to a tower-foundation system and the foundations are verified to be stable when exposed to earthquake motion of the same scale as the Hyogoken-Nambu Earthquake. Three-dimensional model considering nonlinearity of dynamic restoring forces is applied to a response analysis of the superstructure and the superstructure of the bridge in its completed state is also verified to be stable.

5.2 Multi-column foundations of Ohnaruto Bridge

(1)Outline

The Ohnaruto Bridge is a 3-span 2-hinged stiffening-truss suspension bridge. Figure6 shows the general view. The piers for towers adopted a multi-column foundation system, in which the tidal current flows between columns, to minimize both the force of tidal current acting on the piers and disturbance of the tidal flow, which creates whirlpools, a well known sightseeing stuff. Photo1 shows the multi-column foundation of 3P tower.

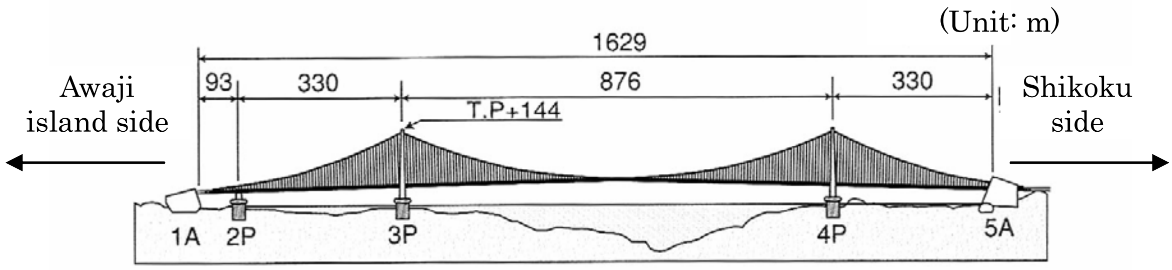

Figure6 General View of Ohnaruto Bridge

The pier was composed of columns and footing. Double sets of 8 columns of a 4m diameter each are aligned in a rectangle with two 7m diameter columns at each center, and the footing caps atop the sets of columns. These columns and footing are RC structure. These columns were composed of RC piers and steel-pipes, which were not strength members, but they were designed as RC piers protection members from outer damage. Since the earthquake motion for seismic verification is stronger than the original design earthquake motion, the steel pipe was regarded as strength member in the seismic analysis. Figure7 shows the outline of structure of the multi-column foundation.

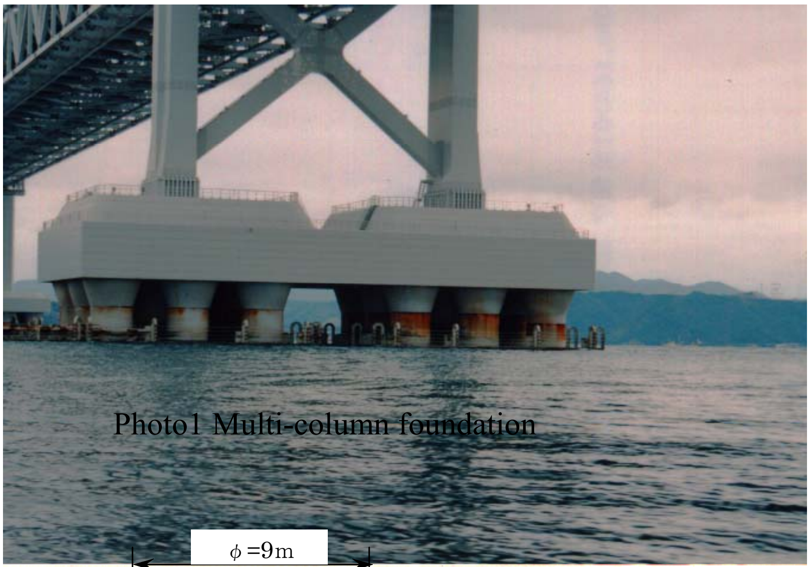

Photo1 Multi-column foundation

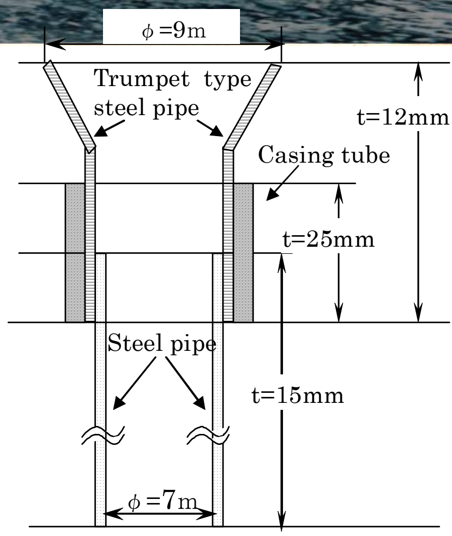

Figure7 Structure of 9m diameter column of multi-column foundation

(2)Analytical condition

A three-dimensional frame model of bridge including multi-column foundations was used for time-history analysis. JMA Kobe earthquake wave calculated in section 4 was used as the input wave for analysis. The influences of ground for multi-column foundation were taken into account by using the Penzien model. The relationship between bending moment and curvature of multi-column was modeled to tri-linear model, and Takeda model was used for a hysteretic model. The damping factor of the superstructure was assumed to be 2%, and Rayleigh damping was adapted.

(3)Results of analysis

The distribution of the bending moment and shear force of the 7m diameter column at 3P tower are shown in Fig-8, 9. Figure8 shows that bending moment is smaller than yield bending moment with all length of column. Although bending moment at some columns exceeds the yield bending moment slightly, bending moment of rests of column were within the yield bending moment. Therefore, multi-column foundations have enough strength for bending moment. It can be seen that shear force exceeds the strength of shear

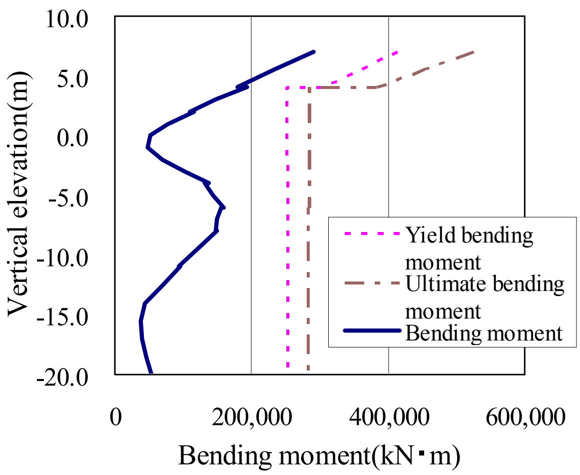

Figure 8 Distribution of Bending moment

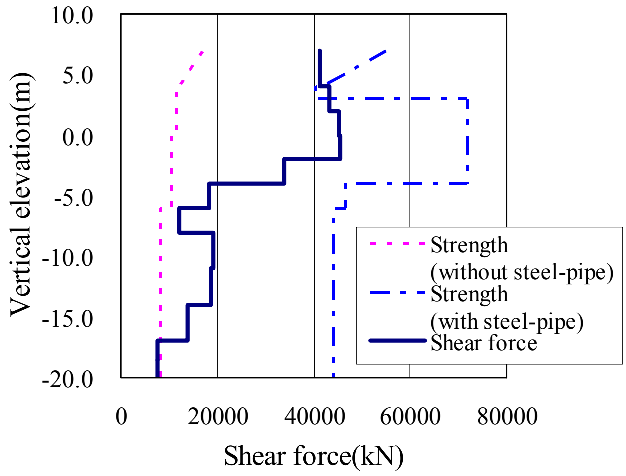

Figure9 Distribution of Shear force

force at small part in transverse direction case in Fig-9. This part over the shear strength is between casing tube and trumpet type steel-pipe and its length is about 2m and thickness of plate is 12mm.

HSBA has judged that there is no possibility of the failure from the following facts; ratio that shear force exceeds the shear strength is small, generally the material strength being used for the steel pipe and etc is higher than nominal value., the section which lacks the shear strength is short in comparison with the diameter of the steel-pipe, if shear failure occurs, direction of the shear failure occurs in the oblique direction. The shear forces of all columns of 4m diameter were within the shear strength in consideration of the steel pipe.

(4)Conclusion

It can be judged that the multi-column foundations as a whole have enough seismic performance against the level2 earthquake, when the steel pipes are regarded as the strength member, though some excess over allowable shear strength appears in small portion

<u>6. The policy of future seismic verification and seismic retrofit</u>

The seismic verification already conducted revealed that the fatal damage will not occur in the long-span suspension bridges and cable stayed bridges, though a few excess over allowable value in some parts of the bridges. However, there are some approach bridges where the piers lack of shearing strength and the shoes lack of ultimate strength.

The seismic retrofit work in those approach bridges will start in accordance with the priority one after another.

All bridges should be evaluated for their seismic performance in consideration of the priority of seismic retrofit. However, the seismic response analysis shall not be conducted for the bridges in case that their seismic performance can be evaluated by analogy with other bridges because of the similarity of structural type, ground condition, etc.

The priority of the seismic retrofit is decided synthetically in consideration of the following conditions.
1) The section of the IC which contains bridges over strait
2) Applied standard (For example, Specification for Highway bridges of 1971, 1980 applicable bridges)
3) Seto-Ohhashi (Highway-railway combined bridges)
4) Occurrence probability of future earthquake (Nankai earthquake)

The fatal damage to the user and the public owing to the bridge collapse must not be caused, and the road should keep a function as an emergency transportation route. Therefore, the unseating prevention system etc should be set up in parallel with the reinforcement of the substructure.

7. Conclusions

The outline about seismic verification of the long-span bridges over the strait of HSB was mentioned in this report.

1) Seismic design standards of HSBA do not consider the influence of an inland near-field type earthquake. Therefore HSBA decided to verify the seismic performance of bridges for Leve2 type Ⅱ earthquake.
2) HSBA decided that the JMA Kobe wave in the Hyogo-ken Nambu Earthquake should be used for the seismic verification wave.
3) The seismic verification already conducted revealed that the fatal damage will not occur in the Akashi-Kaikyo Bridge and the Ohnaruto Bridge, though some excess over limit value appears in some parts of the bridges. The verification results for other bridges are the same as these two bridges.
4) HSBA will continue to execute the seismic verification of the bridges taking the priority of the seismic retrofit into consideration.

References

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