SEISMIC DESIGN AND BEHAVIOR OF THE HIGASHI-KOBE BRIDGE AND RESTORATION AFTER THE 1995 KOBE EARTHQUAKE

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SUMMARY

The Higashi-Kobe Bridge is one of the longest cable-stayed bridges in the world. From the point of seismic design, the Higashi-Kobe Bridge has the unique supporting system in that the main girder is movable in the longitudinal direction. This system gives a longer natural period and can reduce earthquake force of the tower considerably, but has the disadvantage of large displacement of the main girder at earthquakes. In this situation, the 1995 Hyogo-ken Nanbu earthquake struck the Hanshin area. The Higashi-Kobe Bridge also underwent strong earthquake motion, and unexpected damage most of which centered on the shoes of P187 was observed. After the earthquake, an analytical study was made using the observed records, and the results were compared with the damage and movement during the earthquake to evaluate the seismic behavior and damage mechanism. As a result, analytical results show good agreement with the structural damage and analysis is considered satisfactory. In this paper, the seismic design, damage and restoration work of the Higashi-Kobe Bridge, and the seismic performance during the earthquake are reported.

INTRODUCTION

The Higashi-Kobe Bridge is one of the longest cable-stayed bridges in the world, which has the center span of 485m. The bridge is situated in the eastern part of Kobe City and laid across the Higashi-Kobe Channel. It forms a part of the Osaka Bay Route which plays an important role in economic activities of the Osaka-Kobe area. The bridge has the upper and lower roadways 13.5m wide and each deck has 3 lanes. The type of main girder is a Warren truss with no vertical chords and the height of the truss girder is 9m. The tower is H-shaped and consists of 146.5m high columns and curved cross beams. From the point of seismic design, the Higashi-Kobe Bridge has the unique features; the main girder is supported by towers and piers in such a way that the girder is movable in the longitudinal direction. This supporting method was adopted with the aim of changing the fundamental period of bridge to a relatively longer one. By using this supporting method, the seismic force on the bridge towers and foundations is expected to be reduced greatly, resulting in more rational and economical bridge design [Kitazawa, Ishizaki, Emi and Nishimori, 1990].

In this situation, the Hyogo-ken Nanbu earthquake struck the Hanshin area at 5:46 a.m.(JST) on January 17, 1995. It was one of the most powerful earthquake observed by seismometers in Japan, with an intensity of 7.2 on the Richter scale. The major suffered area in Kobe lay along on the north side of the Kobe Route, and tremendous damage was observed on the structures of the Kobe Route. The Osaka Bay Route also underwent strong earthquake motion and unexpected damage was observed on the long bridges of the Route. This paper reports the seismic design, damage and restoration work of the Higashi-Kobe Bridge, and the performance during the earthquake.

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BRIEF REVIEW OF SEISMIC DESIGN

Aseismic design of the Higashi-Kobe Bridge is summarized as follows: By making the bridge structure flexible to a suitable level, reduce seismic force, but provide safety devices to suppress excessive displacement. For evaluating the bridge behavior properly, multi-mode response analysis was adopted to determine sectional forces due to the earthquake. Supporting system of the bridge is very unique reflecting the aseismic design. Vertical support for the main girder at the tower is provided by cable suspension from the upper cross-beam of the tower. Each tower has two vertical cables per side, each holding 850 tons. Two eye-bar pendulum supports are used for each side pier in order to resist the 1,300-ton uplift constantly acting on the piers. Wind shoes are installed on the towers and piers which are designed not to restrain the longitudinal movement of the main girder. There are three main points in seismic design [Kitazawa, Nishimori, Noguchi and Shimoda, 1992].

Selection of the basic structural configuration

In order to determine the ideal supporting system for the Higashi-Kobe Bridge, the earthquake response of several structural configurations were compared. As a result, an all movable support system where the main girder is supported by towers via cables was adopted. This system gives a longer natural period and can reduce earthquake force of tower considerably, but has the disadvantage of large displacement of the main girder at earthquakes. Adopting an all movable support system, wind loads dominate the design of the towers, and the size of the caisson can be made 8m smaller than that of the two-fixed support system

Multi-mode response analysis

From the viewpoint of seismic design, there rises a few problems to be solved in the case of the all movable support system; [1] accurate evaluation of the earthquake response of a long-period structure; [2] large longitudinal displacement of the main girder due to earthquakes and winds. For the first problem, multi-mode response analysis which reflect the response of a long-period range was applied.

As acceleration response spectrum for superstructure design, the most unfavorable case at the construction site was chosen, and the design spectrum was made to have a relatively large safety margin in the long period range. This is because the Higashi-Kobe Bridge has an unprecedented long natural period in the longitudinal sway mode, and the design spectrum around the natural period plays a crucial role in determining the structural response to earthquakes. In determining the design spectrum, the maximum acceleration of seismic wave at the bedrock was assumed to be 160 gal which was an expected value in the 100-year return period, and three earthquake records which were reliable up to the range of 0.07Hz were used to produce the design spectrum. As the result, the design acceleration at 4.4 seconds which corresponds to the natural period of the sway mode of the structure becomes 120 gal. Thus, the design acceleration can be reduced to nearly 1/3 of that in standard bridges in Japan.

Countermeasures for large displacement

The second problem in the previous section can be eased by arranging the cable in a harp pattern and by installing vane-type oil dampers. Arranging cables in harp pattern has the effect of constraining longitudinal movement of the girder. Installation of vane-type oil dampers as a vibration control device aims at preventing excessive girder movement. When the oil passes through the orifice, the turbulent flow is generated and it produces resistance to moderate the movement of the girder. This type of damper is effective in preventing excessive amplitude since the effect of the damper increases as girder movement becomes faster. Also, well-designed damper can reduce the girder movement and control the reaction force on the end piers without changing the characteristics of the natural period.

To investigate the characteristics and applicability of the damper, the test using 1/2-reduced scale models was conducted with different velocities and orifice sizes. In the design of the damper, the earthquake which is 1.4 times stronger than used in the bridge design is considered. This corresponds to the expected value of the mean plus 2 , which was considered to be suitable for design in case of an expectedly strong earthquake. By nonlinear time history response analysis, the girder displacement with dampers was estimated to be 64 cm, which is in the range of the design displacement and below the critical displacement of 74 cm for the tower. The equivalent damping ratio due to the dampers is calculated to be approximately 6% based on the displacement response.

The effectiveness of the damper was also confirmed by full-scale vibration tests using a 1/100-reduced model. The result shows that the displacement response due to the short-period predominant earthquake record is much less than the design value, whereas the displacement response due to the long-period predominant earthquake record without the damper is observed to be nearly equal to the design value. By the energy dissipation of the damper, however, the response is suppressed below the design value. It was also confirmed by vibration tests that the interaction between three directions can be negligible and three dimensional behavior can be predicted from the superposition of a single component result.

DAMAGE OF HIGASHI-KOBE BRIDGE BY THE EARTHQUAKE

The Osaka Bay Route is a relatively new route and this section was opened in April, 1994. The design of the structures in this section was based on the specifications upgraded in 1980. For this reason the damage of the structures due to the earthquake was relatively small although they had not been designed for the earthquake of that intensity. But some of long bridges and approaching bridges had significant damage such as the failures of steel bearings, local buckling and earthquake restrainers. Main damage in the Higashi-Kobe Bridge concentrated on P187 and was as follows [Ishizaki and Sawanobori, 1996]: [1] The upper set bolts of the wind shoe on the end pier (P187) were broken and the upper shoe was taken off. The set bolts of the wind shoes on the other end pier (P182) were loosened. [2] Two vane-type oil dampers on both sides of P187 were broken and taken off. [3] The eye-plates of pendulum shoes on P187 were deformed by transverse force and the pins of pendulum shoes that support vertical forces on both sides of P187 were taken off. [4] The bridge end was lifted up about 0.4 m which caused a step at the expansion joint and railings. [5] The shear type of buckling occurred in the web of the lower beam in P187 and local buckling also occurred at the base of the column on the mountain side (P187). Piers, P182 and P186 had also similar shearing buckling in the lower beam smaller than P187. [6] In addition, the earthquake restrainer between the end pier (P187) and the approach girder was broken at welding due to excessive relative displacement although it seemed to have functioned during the earthquake.

METHOD OF SEISMIC RESTORATION

Restoration work of the Higashi-Kobe Bridge was hurried for early service and basically, the Higashi-Kobe Bridge was restored to the previous shape except that some of key elements damaged by the earthquake were strengthened for securing more safety [Ishizaki and Sawanobori, 1996]. The outline of seismic restoration work is as follows: [1] For the pendulum shoes, damaged connecting eye-plates were cut by gas blaze and carried to the factory for repair, and then they were straightened by heating and reshaped. The upper half of pendulum shoes were also cut and were brought back to the factory for inspection and repair. After the necessary length of the eye-bar being measured in the site, arms of pendulum shoes were newly manufactured. The pins that had come off were also brought back to the factory and were checked by ultrasonic inspection. After the safety of re-used parts were assured and repaired, all parts were assembled temporarily in the shop for checking and then they were carried to the site and set again. [2] The bridge girder was lowered to the designed level using eight sets of wire clamp jacks (each capacity 100tf) and counterweights (130tf), and the pendulum shoes were fixed by connecting the upper part and the rest with high tensile bolts. [3] The wind shoes on the end piers, P182 and P187 were replaced by newly manufactured ones which can sustain the horizontal force 1.5 times as large as the previous design. [4] The oil dampers on the end pier, P187 were seriously damaged and found unable to be re-used. The same type of the vane oil dampers were newly manufactured and broken dampers were replaced. [5] The locally buckled part of the column base of P178 was reinforced with rib plates which can bear the design sectional force by themselves. The shear buckled part in the lower beam web of the pier, P187 was replaced by a new web together with new reinforcing truss members which can sustain the design forces by themselves. The webs of the piers, P182 and P186 that had minor damage were repaired by heating and jacks.

After finishing repair, tension of all the cables was measured by a natural period method before re-opening to confirm that tensile stresses were all within the design tolerances.

PERFORMANCE OF HIGASHI-KOBE BRIDGE DURING THE EARTHQUAKE

As described in the previous chapter, the damage of the Higashi-Kobe Bridge by the earthquake was centered on the shoes and pier of P178. For the purpose of analyzing the behavior of the long cable-stayed bridge, seismometers were placed at several points, and acceleration records can be obtained at three points of tower (h=26m,76m,140m), the base of the caisson (GL-34m), underground level (GL-33m) and ground surface (GL-

1.5m). The maximum horizontal acceleration observed was 326gal at the ground surface which was much less than the acceleration record observed in Kobe-Nishinomiya district. This difference is considered due to the effect of liquefaction in this reclaimed land. Table 1 shows the observed maximum acceleration at each point.

Table 1: Maximum acceleration of the observed records

Observed Position			Maximum Acceleration (gal)		
			Longitudinal	Transverse	Vertical
G1	Underground	GL.-33m	427	443	---
G2	Ground Surface	GL.-1.5m	282	326	396
K1	Caisson Base	GL.-34m	334	355	389
T1	Tower	H=140m	1000	Over 1000	---
T2	Tower	H=76m	386	Around 1000	---
T3	Tower	H=26m	596	807	807

By making use of these seismic records, dynamic response analysis using a three dimensional model was performed and the results were compared with the damage and movement during the earthquake to evaluate the seismic behavior and damage mechanism [Ishizaki, Kitazawa, Nishimori and Noguchi, 1998]. As input earthquake records, the acceleration at the underground level (GL-33m) and ground surface (GL-1.5m) were used each for horizontal directions and vertical direction. The foundation was replaced by a spring-mass model and the effect of adjacent bridge girders was also considered as additional mass. The damping factor of the foundation was decided by parametric analysis so that the analytical results corresponded to the observed record or estimated seismic force. As a result, the damping factor of the foundation was calculated as h=50% which seems to reflect liquefaction, large deformation of soil and disperse damping. It was also assumed that there was no damage of the structure in the dynamic analysis because of difficulty in analysis.

Results of analysis

The maximum acceleration of the tower top is analytically estimated as about 900 gal in the longitudinal direction and 2600gal in the transverse direction, which shows a similar tendency with observed records while the maximum acceleration at the top of the foundation is around 400gal in the longitudinal direction and 580gal in the transverse direction. Table 2 shows the analytically estimated maximum acceleration response at each point of the structure. As for the analytically estimated displacement, the maximum displacement of the girder is about 45cm and that of the tower is about 49cm both in the longitudinal direction, whereas the maximum displacement in the transverse direction is about 90cm in the center of the girder and about 160cm at the top of the tower showing relatively large responses. The maximum stress caused by the earthquake is calculated to be at most 50% of yield stress in the case of the girder and 65% of yield stress for the tower. The total stress including a dead load becomes 75% of yield stress in the P185 due to the large dead load.

Table 2: Analytically estimated maximum acceleration

Element	Position	Maximum Acceleration (gal)		
		Longitudinal	Transverse	Vertical
Main Girder	End Pier (P187)	304	384	358
	Tower (p185)	217	731	495
	Center of Span	128	767	509
	Tower (p184)	265	724	482
	End Pier (P182)	331	418	361
Tower P185 (sea side)	Top of Tower	899	2634	309
	Upper Beam	454	1163	313
	Lower Beam	408	759	346
	Base of Tower	397	579	372
Tower P184 (sea side)	Top of Tower	838	2645	311
	Upper Beam	473	1177	314
	Lower Beam	364	750	347
	Base of Tower	425	581	372

Comparison of the results with observed damage

There left a trace of friction in the wind shoes which shows the real displacement during the earthquake. The displacement of the main girder is estimated to be 45cm at the maximum, while the calculated displacement is 42 46cm which shows good agreement. The horizontal strength of wind shoes is considered to depends on the strength of set bolts, and the horizontal strength of wind shoes is estimated from the strength of set bolts. Comparison of the calculated strength with the estimated seismic force and damage of the shoes also explains good simulation as shown in table 3. The shear strength of the upper beam web on the end pier was calculated taking buckling into consideration. The result shows that shear stress caused by the earthquake is much larger than the calculated strength which can also explain the seismic damage.

Table 3: Comparison of estimated strength and seismic force at wind shoes

Position	Estimated Seismic Force (tf)	Estimated Lateral strength (tf)	Damage Situation
End Pier (P187)	629	660	*Set bolts were broken.*
Pier (P186)	1065	2100	Gap less than 1mm occurred between the sole plate and the upper shoe.
Tower (p185)	2746	3200	Same as the above.
Tower (p184)	2747	3200	Same as the above.
Pier (p183)	1105	2100	Gap of 1 2mm occurred.
End Pier (P182)	689	660	Gap of 4 14mm occurred.

Estimation of the damage mechanism

This series of damage is considered to be caused successively by horizontal force in the transverse direction larger than the design load. The damage mechanism estimated from the damage investigation and the analysis result is as follows: [1] The seismic response stress of the main girder, towers and cables was at most 75% of the yield stress as the supporting condition of the girder was free in the longitudinal direction and the principal period of the center part of the span was also long (about 5seconds). On the other hand, the principal period of the side part of the girder was 1.0 1.3seconds and large seismic force was supposed to act on the end of the girder. [2] Large transverse force from the girder broke the set bolts of the wind shoes on the end pier (P187) after one or two times of the girder's sway. Losing the support of the wind shoes, large horizontal force in the transverse direction worked on the vane-type oil dampers on the P187 and they were also broken at one blow. It is estimated that the horizontal force acted on the wind shoes during the earthquake was more than its strength of 660tf. [3] After these shoes and dampers damaged, the end of girder on the side P187 became free in the transverse direction, and the connecting eye-plates of pendulum shoes on P187 were deformed to open by repeated transverse force and the pins were taken off. The bridge end was lifted up about 0.4 m losing the tensile restraint of the pendulum shoes. [4] The web of lower beam and base section of column on the P187 buckled by repeated large transverse force. Since there existed a similar type of buckling in the web of lower beam on the end pier P182, it is estimated that a similar level of seismic force occurred on the other end pier. However, some difference in structures and soil conditions caused a little difference in seismic response between both end piers, and as a result it produced large difference in the degree of damage between them.

CONCLUDING REMARKS

The Higashi-Kobe Bridge has the unique supporting system from the point of seismic design in that the main girder is movable in the longitudinal direction. After the earthquake, an analytical study was made using the observed records, and the results were compared with the damage and movement during the earthquake to evaluate the seismic behavior and damage mechanism. Although there was difficulty in estimating spring coefficients and damping factors of soil, the obtained results are considered satisfactory showing good agreement with the structural damage.

As for the damage mechanism, the wind shoe on the end pier (P187) was first broken by horizontal force in the transverse direction larger than the design load, and then the pins of pendulum shoes were taken off. The bridge end was lifted up about 0.4m but it did not lead to fatal damage due to the restraint of other piers. The damage of the Higashi-Kobe Bridge by the earthquake was centered on the shoes of P187, and in the restoration work the wind shoes on the end piers P182 and P187 were replaced by newly manufactured ones which can sustain the horizontal force of 900ft. This design force is based on the analytical result since the damage of the wind shoes is considered to have caused other damage. It is more desirable to control an excessive uplift or to install a fail-safe device against unexpected force.

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