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Earthquake resistant design of a long-period cable-stayed bridge

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ABSTRACT: The Higashi-Kobe Bridge is a long-span cable-stayed bridge in which the unique feature is that 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 lengthening the fundamental period of the bridge to a relatively longer period. By using this supporting method, the effects of the inertial forces due to the superstructure on the bridge towers and the caisson foundations will be greatly reduced, thereby resulting in a more rational and economical bridge design.

1 OUTLINE OF THE HIGASHI-KOBE BRIDGE

The Higashi-Kobe bridge, a steel cable-stayed bridge presently under construction, will be an important link forming the Osaka Bay Route of the Hanshin Expressway Public Corporation. The features of this bridge (Fig. 1) can be summarized as follows:

- center span length — 485m
- width of bridge — 13.5m (3 lanes for upper and lower decks)
- main girder — 9m high Warren truss with no vertical chords
- tower — H-shaped tower with 146.5m high columns and curved cross beams tied at relatively low positions

2 EARTHQUAKE-RESISTANT DESIGN

General concepts of the aseismic design are summarized as follows:

- Keep the bridge structure flexible to a reasonable level in order to reduce seismic inertia, but provide safety devices to suppress excessive deformation.
- Adopt multi-mode response analysis to determine sectional forces due to the design earthquake loads in order to properly model the bridge behavior.

2.1 Selection of the Basic Structural Configuration

In order to determine the ideal support system for the Higashi-Kobe Bridge, the earthquake response of several configurations of structural system were considered and the comparison is shown in Table 1. The advantages and disadvantages of these supporting systems can be summarized as follows:

- The use of one fixing support or two fixing supports, case 1 and case 5 in Table 1, causes large forces in the fixed piers. And, even multiple fixing support system (case 2), cannot disperse earthquake forces.
- The use of elastic supports (case 3) makes it possible to adjust natural period of the bridge adequately enough to reduce earthquake forces in the towers. However, using such system will require

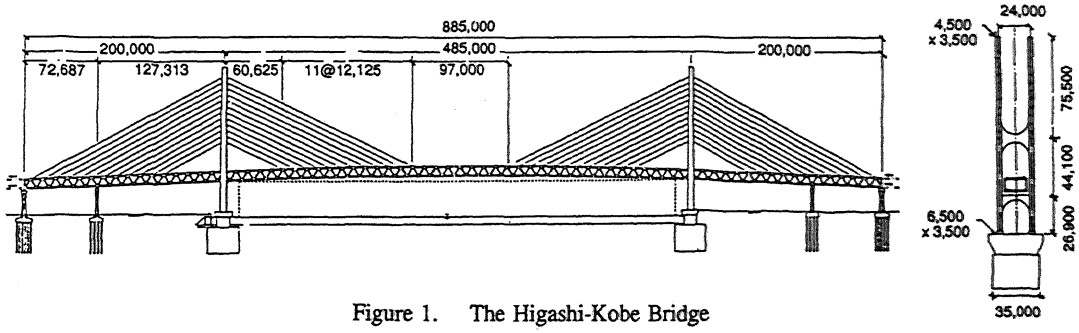

Figure 1. The Higashi-Kobe Bridge

Table 1. Comparative study of different supporting systems

Support Configuration		Natural Period *sec.*	Section Forces per column (at tower base) *kN, kN-m*			Displacement (girder) *cm*
(1) 2 Supports Fixed	*M+F+F+M*	1.42	M=608000	N=90000	S=24000	20
(2) All Supports Fixed	*F+F+F+F*	1.26	M=609000	N=85000	S=24000	18
(3) Spring Supports	*M+S+S+M*	3.01	M=308000	N=88000	S=10000	37
(4) Supports All Free	*M+M+M+M*	8.69	M=155000	N=90000	S= 2000	56
(5) One Support Fixed	*M+F+M+M*	2.21	M=602000	N=97000	S=23000	22

Load combination: $D + L_{EQ} + EQ_L + T_{15}$; Fan pattern cable arrangement assumed; *'M'* movable; *'F'* fixed; *'S'* elastic

careful maintenance.

- The use of all movable supports (case 4) where the main girder is supported by towers via cables (herein refered to as "all free") gives the bridge a rather longer natural period, and therefore reduces earthquake forces of the tower considerably. However, the displacement of the main girder can be quite large.

Acceleration response spectrum for the superstructure design was determined against the most unfavorable case at the construction site especially in the longer period range. Table 2 shows forces categorized according to design loads comparing the all-free and the two-fixed support systems. Adopting the all-free support system, wind loads (rather than earthquake loads) governs in the design of the towers. As a result, the size of the caisson for the all-free system can be made about 10m smaller than that designed for a two-fixed support system making an estimated saving of about 1.8 trillion yen (about 14 million US dollars at the current exchange rate).

Table 2. Forces categorized by design loads

Design Loads	All-Free System		Two-Fixed System	
	S *(kN)*	M *(kN-m)*	S *(kN)*	M *(kN-m)*
Wind W	9200	440000	9300	219000
Seismic EQ	7600	406000	25000	600000
Critical Design load	7200	348000	22900	400000
	(D+W+T)/1.35		(D+L+EQ+T)/1.50	
Plate at Tower Base	36mm		45mm	
Caisson (L×B)	32m × 35m		40m × 35m	

N.B. – Harp pattern cable arrangement is assumed
– Forces are given per column

2.2 Countermeasures for Large Displacements

Having selected the all-free configuration, the following problems should be properly addressed:

- accurate and reliable evaluation of the earthquake response of long-period structure;
- disadvantage due to out-of-plane buckling of the tower column not fixed to the main girder;
- large longitudinal displacement of the main girder due to earthquake and wind.

The latter two problems can be solved by arranging the cables in a harp pattern. Arranging cables in harp pattern has the effect of constraining rotational movement of the towers and also of constraining longitudinal movement of the girder (Table 3). The next section of this paper addresses the first stated problem.

Table 3. Tower displacements for different cable pattern

Cable Pattern	HARP	FAN
WIND	61.3 cm	189.5 cm
SEISMIC	62.3 cm	75.5 cm

2.3 Design Acceleration Response Spectra

The design spectrum was provided with a relatively large safety margin in the long period range. This is because the Higashi-Kobe bridge has an unprecedented long natural period of longitudinal sway mode oscillation. The design spectrum range around the natural period plays a critical role in determining the structure response to earthquakes. The design spectrum was determined by the following procedure (Fig. 2):

1. In formulating the design acceleration response spectrum, a 1000-meter deep bedrock at the construction site was assumed from the fact that a granite layer lies at that depth. By taking the deep bedrock, there occurs a possibility of the seismic wave and bridge oscillation frequencies coinciding and causing amplification in the long period range.
2. The maximum acceleration of seismic waves at the bedrock was assumed to be 160 gal as an expected value during a 100-year return period. For the input waves W1 at the bedrock, three earthquake records including Taft were used as data which were reliable up to the range of 0.07 Hz. These are then converted to their equivalent bedrock motion.

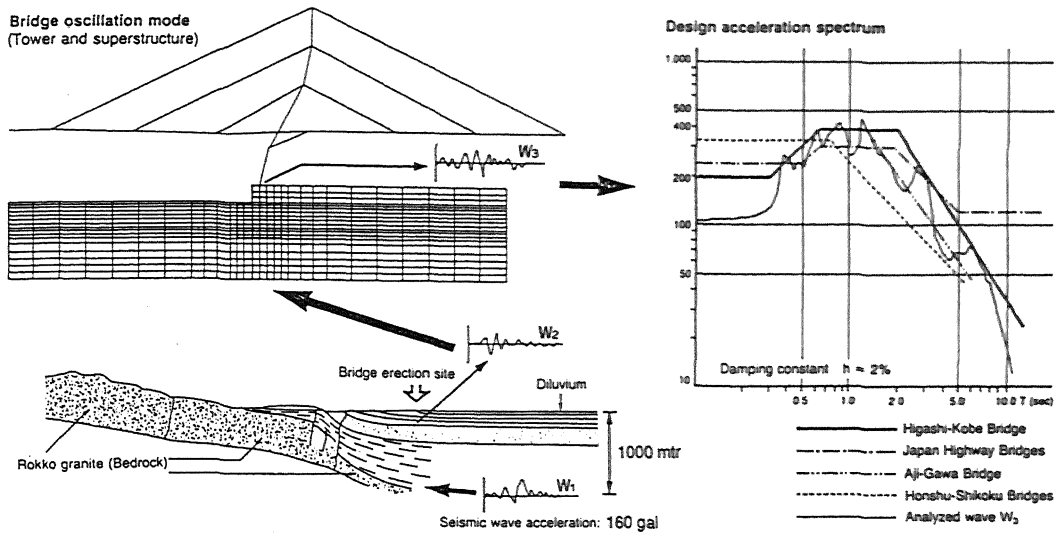

Figure 2. Procedure used in determining design spectrum

3. Waves W2 at the 80m deep design ground base, which will be used for the earthquake response analysis using finite element method, were obtained by multiple wave reflection analysis in which strain dependency of soil properties was considered.
4. The acceleration response spectra for the superstructure were obtained from the tower base waves W3 in the above analysis in which the superstructure, substructure and the surrounding ground are modeled. These spectra have peaks at natural periods of the ground coinciding with that of the bridge.
5. The design spectrum was determined by taking an envelope curve of mean spectrum of these three in longer period range. The shorter and intermediate part of the spectrum was determined by refering to other specifications for highway bridges.

As a result, the design acceleration at 4.4 seconds which corresponds to the natural period of the sway mode oscillation of the structure becomes 120 gal., which is nearly 1/3 of that used in standard bridges in Japan. The sectional forces and displacements using this design criterion have already been shown in Table 2.

The design spectrum was confirmed by another study in which records by Japan Meteorological Agency Seismometer and the same records by SMAC were first compared to find differences in both waves and to find recording error of the SMAC. It was found that the error was caused by the spring which controls the SMAC pen movement. Since the ground moves slowly during a long period earthquake, the spring cannot have the power to pull the pen. The error which should be removed turned out to be the minimum value of the smoothed Fourier Spectrum. An attenuation equation was obtained from the relation between magnitude, epicentral distance and SMAC acceleration data modified by the above method. By supposing the Nankaido Earthquake that will cause the worst possible effect for the bridge site, the expected spectrum on the ground was obtained. As a result, it was found that the design spectrum already determined corresponds to the expected value of the mean plus 1σ (standard deviation) and was judged suitable for design use (Figure 3).

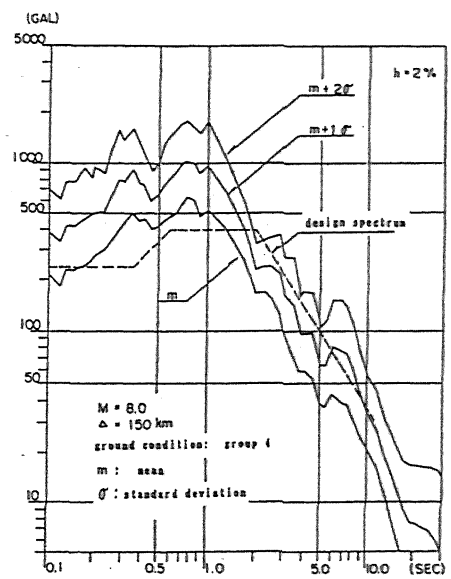

Figure 3. Confirmation of design spectrum

2.4 Earthquake Safety Devices — Development and Design

As there is not much data on earthquake resistant design for a cable-stayed bridge with a long natural period, it was decided to install a vibration-resisting device to prevent excessive girder movement which might result in tower collapse. A case in which a simple stopper is installed on each end pier was analyzed. The result indicated that because of the low rigidity of the piers, the movement of the heavy girders cannot be controlled.

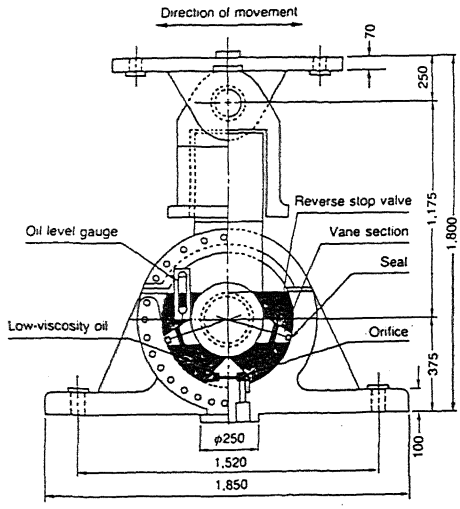

Figure 4. Vane-type oil damper

After considering several devices, the vane-type oil damper was selected. The turbulent flow generated when the oil passes through the orifice due to girder movement produces the power that moderates the movement (Fig. 4). The characteristics of the vane-type oil damper are as follows:

- The damper does not have the problem found in the case of simple stoppers in which small difference in clearance setting often results in big difference in reaction force.
- The effect of the damper increases as girder movement becomes larger and faster. This indicates that the damper is very effective in preventing excessive amplitude.
- The damper increases the safety margin without changing the characteristics of the natural period of the bridge.
- A well-designed damper reduces the girder movement and controls the reaction forces on the end piers.
- The effect of oil temperature on the damper performance is negligible.

To examine the characteristics and applicability of the damper, a 1/2-scale model was fabricated for performance testing. The test was conducted with different movement velocities and orifice sizes. The test results reveals that the turbulent flow resistance characteristic is given by the following expression:

$F = 1840V^2 \times 2$ (in case the orifice angle is 15°)

where V: girder movement velocity; F: resistance which is equivalent to the reaction on the pier.

In designing the damper, an earthquake that is 1.4 times stronger than that considered in the bridge design is assumed. Since the design spectrum corresponds to the expected value of the mean plus 1σ, the value of mean plus 2σ was considered to be suitable for design due to an unexpectedly strong earthquake. Using the characteristics of the vane-type oil damper and this earthquake input, nonlinear time history earthquake response analysis was conducted. The input earthquake used was that of the Izu Peninsula Earthquake.

The results are given in Table 4 and can be summarized as follows:

- If an earthquake 1.4 stronger than the design spectrum occurs on the bridge without the dampers (structural damping ratio of 1% is assumed), the displacement of the girder will be $72 \times 1.4 = 102$ cm. This will be over the critical displacement of 74 cm at which the tower buckles.
- If the dampers are installed, the girder displacement will be reduced to 64 cm, which is in the

Table 4. Effect of Vane-type Damper on Girder Displacement

Earthquake Level	Damping Constant (%)	Relative Girder Displacement (cm)	Maximum Response Velocity (cm/sec)	Damper Reaction (kN)
whole structure	2*	61*	—	—
design level	1	72		—
(1.0EQ)	2	61	98	—
girder stopping device	1	102	231	—
design level (1.4EQ)	1+6.2**	64	99	3620

Remarks:
- Values marked with * were obtained using the design spectrum
- Damping constant marked with ** indicates damping of the whole structure plus damping due to the damper

range of the design displacement of 61 cm. This is below the critical displacement for the tower to buckle. In this case, the equivalent damping constant due to the damper is calculated to be approximately 6% based on the displacement response.

2.5 Verification of Dynamic Behavior of the Bridge by Shaking Table Tests

The appropriateness of the method adopted to evaluate the bridge response to the earthquake and the effectiveness of the damper were confirmed by vibration tests using a three-dimensional 1/100-scale elastic model (Fig. 5). The model was made of steel satisfying the similarity of stiffness and weight. The natural frequency and mode shapes of the model in several low modes agreed well with the analytically predicted ones. The structural damping of the model without the damper is adjusted to 1~2%, reflecting fairly light damping of a large flexible structure. An electromagnetic damper is attached to the girder of the model to substitute the vane-type oil damper.

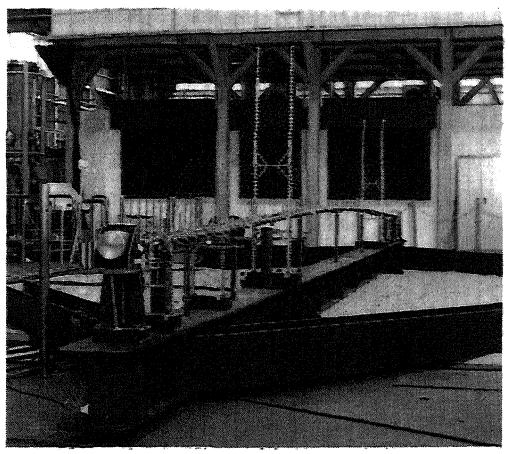

Figure 5. Full model vibration test

Modified long-period predominant and short-period predominant earthquake records were used for input earthquake ground motion. Results are shown in Table 5. All values are converted to the prototype for comparison. Displacement response due to the short period predominant earthquake (El Centro NS, 1940) record is found to be much less than the design values.

Displacement response at the top of the tower due to the long period predominant earthquake motion without the damper is observed to be nearly equal to the design values. However, the energy dissipation of the damper suppressed the response fairly well to the design values. Close agreement of the experimental and numerically calculated results verifies both methods.

To investigate complex 3-D dynamic behavior of the bridge, single component vibration tests in three orthogonal directions and simultaneous three-component vibration tests were conducted and it was confirmed that no interaction between the three directions exists and thus 3-D results can be predicted from the linear superposition of the single component tests.

Table 5. Maximum Displacement Response (cm.) for Cases with and without Damper

Input Earthquake		Input Along Long'l Axis	
		No Damper	With Damper
Long Period	Izu-oki EQ. (synthesized)	64.0	50.0 (53.9)
	Chiba-oki EQ. (corrected)	63.0	52.0 (51.6)
Short Period	El Centro EQ.	18.5	17.0
Design Value by Response Spectra		60.4	—

N.B. – Viscous damping coefficient of 2% is assumed for cases without damper
– Values ( ) are those calculated from time history response analysis

3. VIBRATION TEST OF ACTUAL TOWERS

In earthquake resistant design process of the Higashi-Kobe bridge, damping characteristics of the tower itself is assumed as $h = 0.01 \sim 0.02$ (damping ratio). Vibration experiments of the actual tower of the Higashi-Kobe bridge using a pair of exciters were employed to verify the dynamic property assumed in the design and the efficiency of the vibration suppressing devices used only during construction, namely TMD (tuned mass damper) and the TLCD (tuned liquid column damper) as shown in Fig. 6. The exciters and the dampers were attached to the top of the tower.

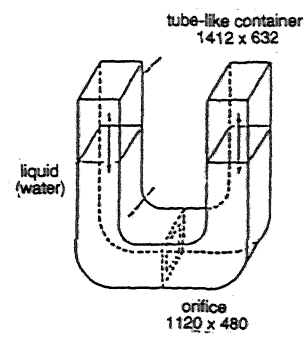

Figure 6. Tuned liquid column damper

The natural frequency and damping of the tower are shown in Table 6. Calculated and measured natural frequency agrees fairly well in the first and second modes. The logarithmic decrement was found to be about 0.03 for the first mode with the dampers.

Table 6. Natural frequencies and damping capacity of the actual towers

mode	Freq. cps	logarithmic decrement		
		tower only	w/ TMD	w/ TLCD
1st	0.256 (0.249)	0.028 ~0.040	0.098	0.110
2nd	1.107 (1.050)	0.094 ~0.099	—	—

N.B. — values in ( ) are obtained by analysis

The frequency response curves of the tower with and without dynamic dampers are shown in Fig. 7. The maximum response of the TMD- and TLCD-suppressed cases are reduced to almost 1/3 of that of non-suppressed case (Fig. 7). It is verified that logarithmic decrements of those devices are more than 0.1 and effective to control wind-induced excitation.

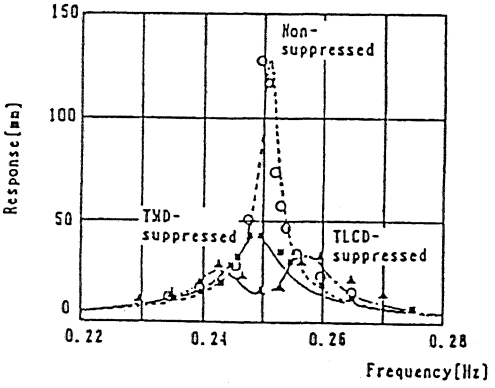

Figure 7. Resonance tests

4. CONCLUSIONS

Main conclusions drawn from this study on aseismic design of a long-period cable-stayed bridge are as follows:

- The main girder is supported by the towers through cables in such a way that the girder is movable in the longitudinal direction. By this supporting method, the fundamental period of the bridge is lengthened to reduce seismic design forces. Consequently, the size of the foundations of the towers was significantly reduced.
- To avoid large displacement due to the all-free girder supports, harp pattern arrangement of the cable was adopted. In addition, harp pattern is considered to be aesthetically favorable due to the absence of visual criss-crossing of the cables when viewed from the sides.
- To prevent failure of the towers due to excessive displacement by exceptionally large earthquake ground motion, vane-type oil damper has been developed and installed between the main girder and the end piers.
- The logarithmic decrement of the tower was found to be 0.03 and that for the tower with TMD or TLCD was found to be more than 0.10 and effective to control wind-induced excitation.

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