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ANALYTICAL STUDY ON SEISMIC BEHAVIOR OF CABLE-STAYED CONCRETE BRIDGE

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SUMMARY

The nonlinear response of a cable-stayed concrete bridge with tall tower was analyzed considering the degrading of the stiffness under severe earthquake motion. The influence of cable vibration on the bridge behavior was also studied numerically. The bridge showed enough strength against the estimated ground motions to the satisfaction of the designer.

INTRODUCTION

In recent years, various types of cable-stayed concrete bridges have been constructed under the various seismic conditions in Japan. It has become important to study the nonlinear behavior and to make rational design of the bridges.

This paper focuses on the nonlinear dynamic analysis of the Tsukuhara Bridge, cable-stayed concrete bridge that was constructed in 1986.

OUTLINE OF TSUKUHARA BRIDGE

The Tsukuhara Bridge shown in Fig. 1 and Fig. 2 was constructed in Hyohgo prefecture in the western part of Japan. It has a tall concrete tower with 89 m and a long concrete girder with 174 m over the valley. The girder is rigidly connected to the tower by the cast-in-place concrete. The girder has laterally movable supports at both ends.

This bridge is of a multi-cable type that fixes diagonal cables to the main girder at intervals of 7.0 m. Diagonal cables are used to shape two semi-harp forms suspended in parallel. The height of girder is suppressed to 1.6 m The employment of a low girder is based on a recent design concept of a long cable-stayed bridge to support the load with diagonal cables instead of bending rigidity of the girder.

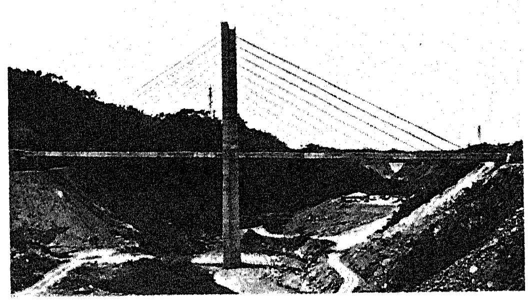


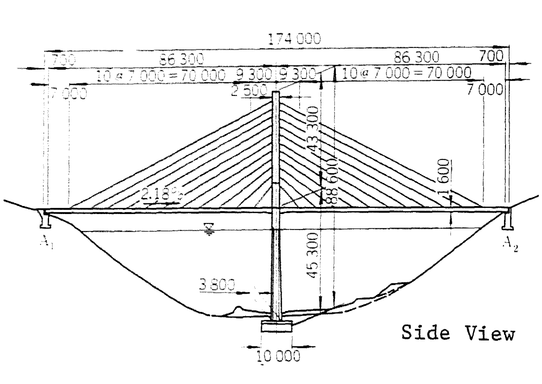
Fig. 1 General View of Tsukuhara Bridge

NUMERICAL ANALYSIS

Design Ground Acceleration Historical earthquakes having a magnitude of more than 5 and a distance less than 150 km were studied as shown in Fig. 3. The empirical acceleration at the site was calculated by the attenuation equation developed by the Public Works Research Institute, Ministry of Construction. The design ground acceleration was decided as 120 gal through the statistical process.

Though the seismic activities around the site was not high, the provable maximum ground motion was assumed to be 500 gal for the earthquake proof check of the bridge in this study as shown in Table 2 on the last page of this paper.

Acceleration Wave Form at the Foundation Two types of acceleration diagram were employed for the analysis. One is Hachinohe NS in 1968 off Tokachi earthquake with magnitude 7.9 as shown in Fig. 4 and the other is El Centro NS in 1940 with magnitude 7.0.



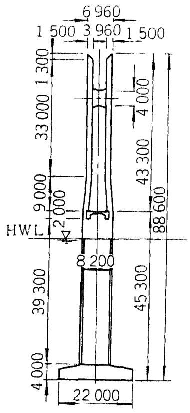


Fig. 2 Dimension of Tsukuhara Bridge

Front View

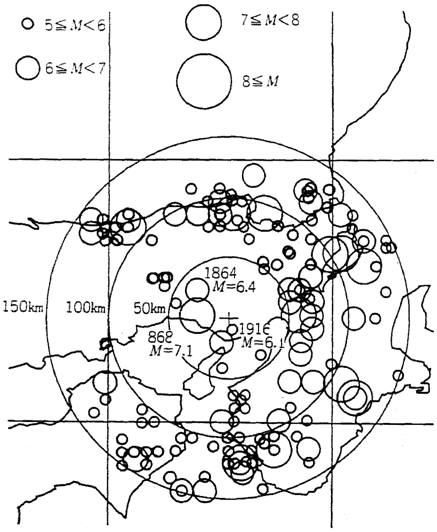


Fig. 3 Locations of the Epicenters : from 700 to 1986

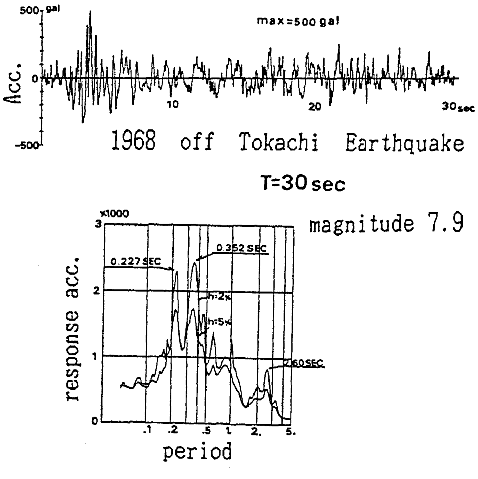


Fig. 4 Input Acceleration Diagram

Numerical Analysis Model A plane model shown in Fig. 5 was employed modeling the tower and girder as concentrated mass and beam elements. As the foundation was placed on the sound rock, the bottom end of the tower was considered as a fixed end. The mass of cables was added to the tower and the girder, respectively, in this study.

Cable Vibration Effect The cable vibration effect was examined by the numerical method. The response by the precise model considering cable vibration was compared with the simplified model. The mass of cables was added to the tower and the girder respectively in the simplified model. The response bending moment of the precise model decreased by 2% compared with the simplified model because of the separate vibration of the cables. The cable vibration has small effect on the tower response because of the small weight ratio of the cables as shown in Table 1 in the case of concrete girder.

Hysterisis Characteristics of Members Fig. 6 shows the skeleton curve and the hysterisis curve applied in the analysis. The well known degrading trilinear model and step-by-step integration procedure was employed.

Table 1 Cable Weight Ratio

	Weight; ton	Ratio; %
Cable weight	33	1.5
Girder weight	2,200	100.0

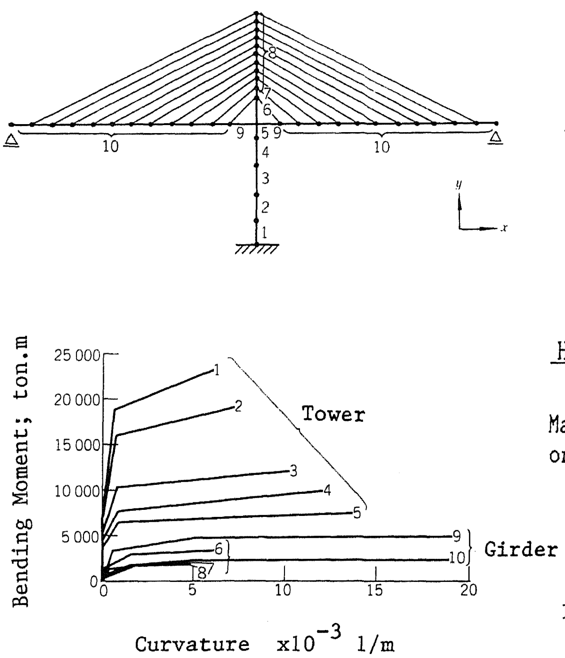


Fig. 5 Numerical Analysis Model

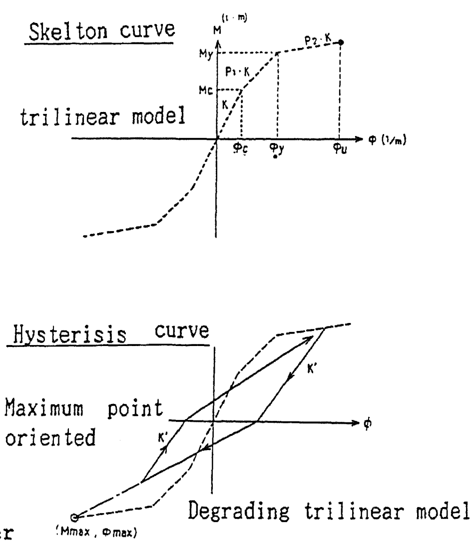


Fig. 6 Hysterisis Characteristics of the Concrete Members

Calculation Procedure The following increment equation was employed in the analysis:

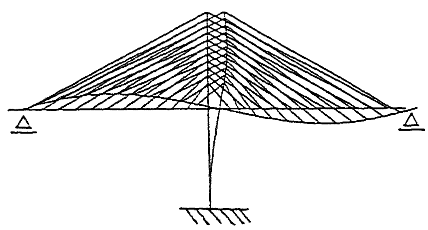
$$M\Delta\ddot{u} + C\Delta\dot{u} + K^{t}\Delta u = \Delta R$$

where, M, C: Mass matrix and damping matrix
k^{t}: Tangent rigidity matrix between time t and $t+\Delta t$
ΔR: External force increment vector
$\Delta\ddot{u}$: Acceleration increment vector
$\Delta\dot{u}$: Velocity increment vector
Δu: Displacement increment vector

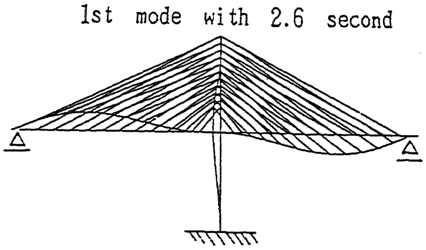
The Newmark's β method and Newton method were used for integration and iteration, respectively.

Vibrational Modes and Natural Period A free vibrational analysis was made to calculate natural period and modes. Fig. 7 shows principal vibration modes in the order of stimulus coefficient. Vibration modes that have a large stimulus coefficient are all anti-symmetric vibration modes. The primary vibration of this bridge has an anti-symmetric mode with natural period of 2.6 seconds. The first mode shows the lateral movement of the girder together with the bending deformation of the tower.

Response Displacement Time History Fig. 8 shows the nonlinear displacement response of the joint compared with the linear response under the same 500 gal ground motion. The predominant period without the degrading of the members is 2.6 seconds and it yields to 4.4 seconds with degrading system. The maximum amplitude without degrading is 50 cm and it decreases to 35 cm with degrading system. The reason of smaller response is mainly upon the energy absorption during the degrading process.

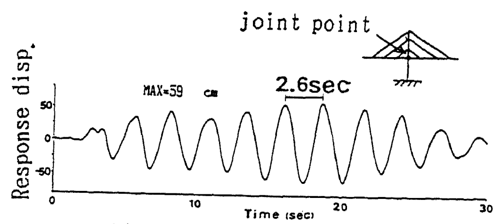


1st mode with 2.6 second

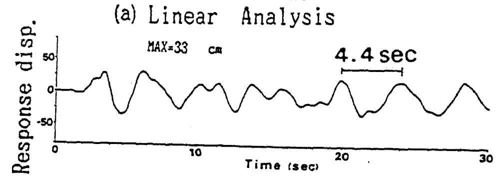


2nd mode with 1.4 second

Fig. 7 Vibrational Modes and Period



(a) Linear Analysis



(b) Nonlinear Analysis

Input acc; Hachinohe 500 gal

Fig. 8 Displacement Time History

Bending Moment Time History The bending moment time history of the bottom end of the tower is shown in Fig. 9. The predominant period in the linear analysis corresponds to the fundamental natural period of the bridge. The predominant period in the nonlinear analysis, which is longer than the original fundamental period, derives from the deterioration of the bending rigidity of the tower.

Maximum Bending Moment The maximum response of the bending moment is shown in Fig. 10 by double circles. The maximum bending moment at the bottom end of the tower in the nonlinear analysis is a little greater than the yield moment that corresponds to yield of the steel bar. The result of linear analysis show much over-estimation from the actual M-ϕ relationship.

Distribution of Maximum Response The maximum acceleration is 1.2 G which is caused by the higher modes of vibration of the girder as shown in Fig. 11. The maximum bending moment of the girder in the nonlinear analysis shows rather uniform distribution than the linear response result due to the degrading of the bending rigidity as shown in Fig. 12.

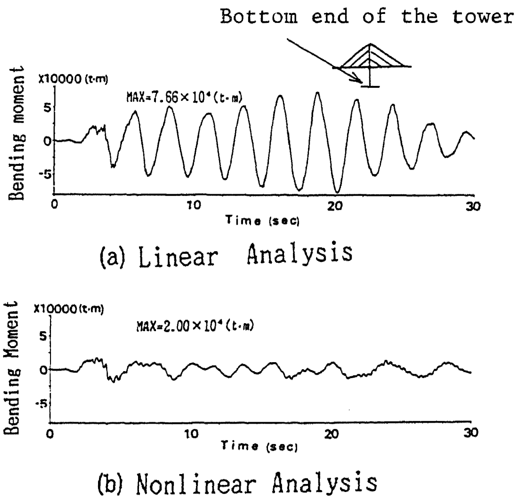


Fig. 9 Bending Moment Time History

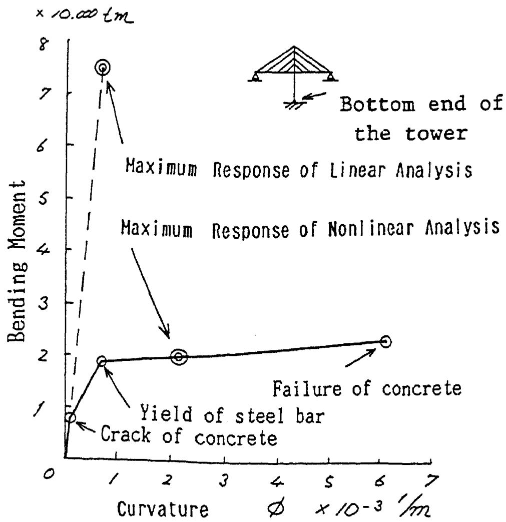


Fig. 10 Maximum Bending Moment

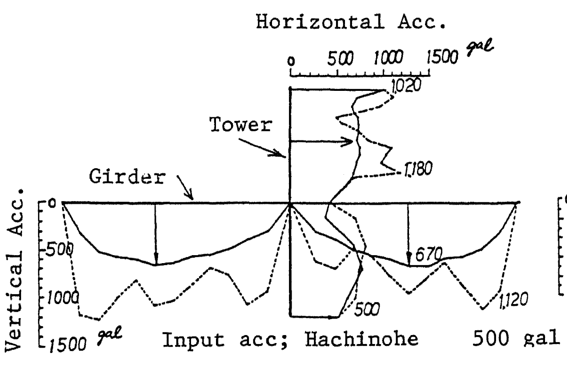


Fig. 11 Maximum Acceleration Distribution

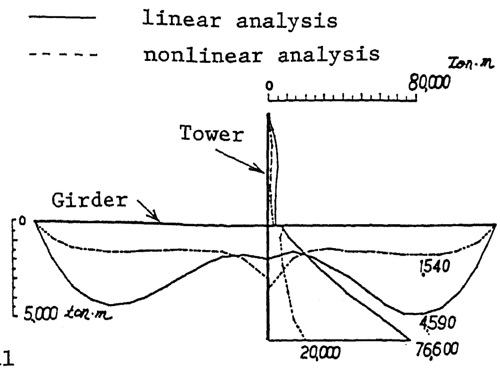


Fig. 12 Maximum Bending Moment Distribution

Crack Extension Fig. 13 shows the extension of the bending crack zone of the concrete in the nonlinear analysis. The initial crack starts from the bottom end and the middle part of the tower, then extends almost the whole tower during the first 3 seconds from the secondary wave arrival. The crack extension of the girder follows after the crack extension of the tower when the maximum ground motion occurred. The yield of the steel bars occurs in the three elements of the tower, but compressive concrete failure does not occur in any element.

CONCLUSION

The dynamic behavior of the cable-stayed bridge with a single tall tower was studied by the nonlinear analysis. The crack of the concrete members and yielding of steel bar were considered as the factors of nonlinearity.

The study revealed the importance of nonlinear dynamic analyses for the rational seismic design.

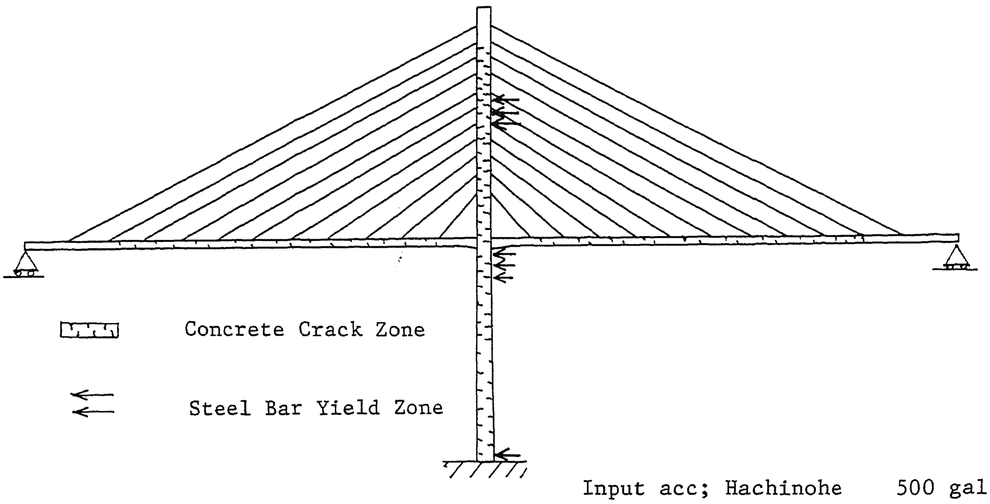


Fig. 13 Extension of Concrete Crack Zone

Table 2 Maximum Input Acc. for the Analysis

	Design ground acc.	Max. acc. for proof check
Linear analysis	1A ; 120 gal	2A ; 500 gal
Nonlinear analysis	1B ; ---	2B ; 500 gal