Earthquake protective design for super-long-span bridges

Earthquake Protective Design for Super-Long-Span Bridges

Koichi YOKOYAMA
Dir., Earthquake Disaster Prevention
Ministry of Construction
Tsukuba City, Japan

Keiichi TAMURA
Head, Ground Vibration Div.
Ministry of Construction
Tsukuba City, Japan

Shigeki UNJOH
Head, Earthquake Eng. Div.
Ministry of Construction
Tsukuba City, Japan

Toshimi MORITANI
Mgr, Third Design Div.
Honshu-Shikoku Bridge Authority
Kobe City, Japan

Summary

Japan is one of the most seismically disastrous countries in the world and has often suffered significant damage from large earthquakes. Therefore, the effect of earthquakes is one of the most critical issues in the design of civil infrastructures. We have studied the feasibility of a super-long-span suspension bridge with a center span length over 2,000m that is to be constructed on the fault zone which may cause large earthquakes with Magnitude 8. This paper presents the research and development on the earthquake protective design and technology for the super-long-span bridge, including seismic design ground motions, innovative seismic design methods for foundations and earthquake response control technology for superstructures.

1. Introduction

The tower-pier foundations and anchorage foundations of a super-long-span suspension bridge with a center span length over 2,000m becomes extreme large and heavy rigid foundations to support the weight of long-span superstructures. When the foundations are subjected to strong ground motions during large earthquakes, the inertia force of the foundation itself is the most critical for the stability of the foundations. It is important to decrease the mass and then the inertia force to improve the seismic performance of the foundations and to develop the stability evaluation methods for the foundations considering the nonlinear effect of surrounding soils. To solve these subjects, research and development on new types of foundation as well as the seismic design methods against large earthquake has been conducted [1].

On the other hand, since superstructures with longer spans over 2,000m becomes relatively flexible and have smaller damping characteristics, therefore, the vibration of the superstructures such as towers and stiffening girders is easily developed by external excitations such as wind and earthquakes. In the design of the superstructures, the effect of wind is generally the most critical design conditions. However, when the bridge with low damping characteristics is subjected to extremely strong earthquakes, the effect of earthquakes also becomes a critical issue to determine the member design. To improve the safety of superstructures against earthquake ground motions, one solution is an application of earthquake protective systems such as installation of damping augmentation devices.

To protect the super-long-span bridges from earthquakes and to achieve economical and practical design, the new design concept is being introduced in the seismic design methods for the super-long-span bridges. Two stage design concept considering moderate earthquake ground motion with high provability to occur and extreme earthquake ground motion with low probability to occur. Against such two types of earthquake ground motions, the acceptable design limit state of the bridge is being studied and proposed.

This paper presents the earthquake protective design and technology for the super-long-span bridges, including seismic design ground motions, innovative design methods for foundations and earthquake response control technology for superstructures.

2. Seismic Design Ground Motions

2.1 Basic Concept of Seismic Design Ground Motions

In Japan we have threat of destructive ground motions caused by both inter-plate earthquakes in the ocean such as the 1923 Kanto Earthquake and inland intra-plate earthquakes such as the 1995 Kobe Earthquake. These earthquakes are very rare to occur, however ground motions generated by them are extremely strong. In addition to these earthquakes, we also have many earthquakes which occur more frequently. Considering the occurrence ratio and influence of various earthquakes, it is rational to classify ground motions into two groups for seismic design of structures as follows:

(1) L1 Ground Motion
L1 Ground Motions represent those generated by earthquakes which have relatively high probability to occur during design life time of structure. This ground motion may be determined as ground motion with return period of 150 years. A bridges shall be designed so that it is not damaged against this ground motion.

(2) L2 Ground Motion
L2 Ground Motions correspond to those generated by earthquakes which unlikely to strike a structure during its life time. Ground motions due to huge inter-plate earthquakes which have their hypocenters in the ocean and earthquakes by inland faults should be incorporated into L2 Ground Motions. A bridge shall be designed so that it is not fatally damaged when this ground motion strikes.

2.2 Establishment of Seismic Design Motions

(1) L1 Ground Motion
This ground motion can be determined by risk analysis [2]. The relationship between earthquake magnitude and occurrence ratio, e.g., Gutenberg-Richter equation, and the attenuation equation of ground motions may be incorporated into this analysis. The deviation associated with attenuation equation and so forth should be carefully considered in this process.

(2) L2 Ground Motion
In case of developing L2 Ground Motion, it is necessary to consider the spread of fault plane and location of fault, because near field ground motions should be incorporated into this seismic design ground motion. As an example, fault plane models proposed for the 1923 Kanto Earthquake are illustrated in **Fig.1**. The following methods are employed to establish L2 ground motion. Note that each method has its own characteristics and obtained result should be compared each other when a seismic design ground motion is proposed.

<u>Attenuation Equation</u>
Estimate the peak ground motion and response spectra from earthquake magnitude and distance from epicenter or fault by attenuation equations. This method has been widely used in engineering practice. Note that the near field ground motion data are usually limited and this equation gives average ground motion, therefore suitable correction for near filed and the deviation from inferred result should be considered.

<u>Fault Rupture Process Model</u>
Divide a fault into subfaults, simulate the rupture process and generate a ground motion.

Vigorous studies have been made in this area mainly from a viewpoint to reproduce ground motions of past events [3-5]. This method requires various information about a fault and its rupture, however, note that available information is generally limited and the estimated result strongly depends on assumed conditions.

Recorded Strong Ground Motion
Use ground motions recorded at near field of large earthquakes as design ground motions. Ground motions reflect various conditions including earthquake magnitude, distance from fault and ground condition, and these conditions should be carefully considered when they are employed as seismic design ground motions. With deployment of strong motion monitoring networks, near field ground motions have been accumulated, and these ground motions can be effectively used with results inferred by other methods.

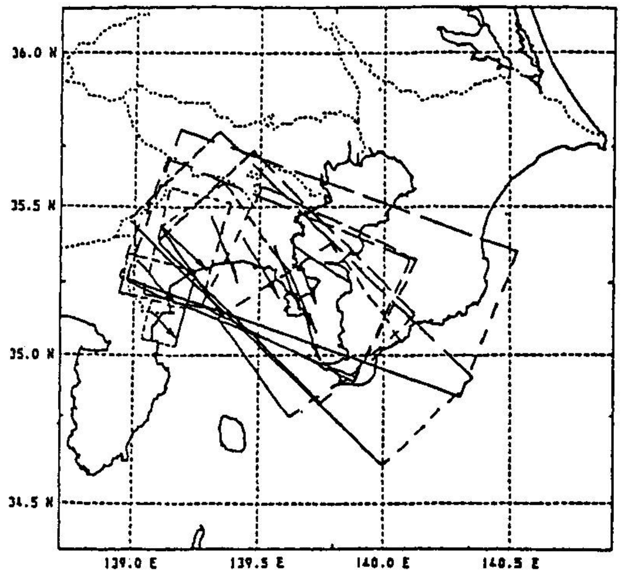


Fig.1 Fault Plane Models Proposed for 1923 Kanto Earthquake

3. Seismic Design Methods for Foundations

3.1 Design Methods against L1 Ground Motion

Fig.2 shows an analytical idealization of a rigid foundation using a two-degree-of-freedom model. The earthquake response against Ll Ground Motion is analyzed by the response spectrum analysis method considering the effects of soil-foundation-interaction and the strain dependence of soil stiffness. Acceleration response spectrum for the seismic design is given for each foundation obtained based on the fundamental acceleration spectrum which was given as L1 Ground Motion in the preceding section and the ground condition. To determine the design acceleration response spectrum, the effect of soil-foundation-interaction is included in the effective earthquake ground motion.

Analytical model of the rigid foundation is two-degree-of-freedom system with a horizontal mass consisted of the weight of foundation itself and additional mass including dynamic water pressure, and rotational inertia mass. Soil stiffness is obtained based on the soil-foundation-interaction. Dynamic soil stiffness is a reaction force to develop unit displacement to the foundation. The complex soil stiffness, which gives soil stiffness by a real part and damping by an imaginary part, is used in the analysis. Complex soil stiffness is given as Eq.(1) which is depending on a frequency.

$$K(\omega) = k(\omega) + i\,\omega\, c(\omega) \quad (1)$$

where, $K(\omega)$: dynamic soil stiffness, $k(\omega)$: spring function, $c(\omega)$: damping function, ω : circular frequency, and i : imaginary number.

Complex soil stiffness, K^*, considering material damping, D, is approximately given by Eq.(2).

$$K^* = k(a_0) + i\,\{a_0 c(a_0) + 2\,Dk(a_0)\} \quad (2)$$

where, $a_0 = \omega\, a/ Vs$, a : diameter of foundation, and Vs : shear velocity of soil.

Separation between the ground and the bottom of the foundation is considered by the Eq.(3). **Fig.3** shows the relation between the moment and rotation angle of the foundation. The energy equal concept is used to estimate the nonlinear response of the foundation.

$$\left.\begin{array}{ll}\theta_N = \theta_L & (M_L/M_0<1.5)\\ \theta_L/\theta_0 = \{6(\theta_N/\theta_0) - 40/3(\theta_N/\theta_0)^{3/10} + 25/3)\}^{1/2} & (M_L/M_0 \geqq 1.5)\\ \theta_0 = M_0/k_R & \end{array}\right\} \quad (3)$$

where, θ_N : nonlinear rotation angle of foundation, θ_L : rotation angle obtained from linear analysis, M_N : moment at the bottom of foundation considering separation between foundation and ground, M_L : moment obtained from linear analysis, M_0 : moment when separation between foundation and ground is initiated, and k_R : rotation stiffness.

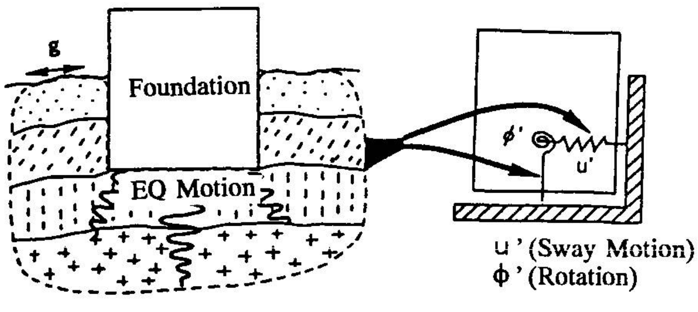


Fig.2 Analytical Idealization of Rigid Foundations

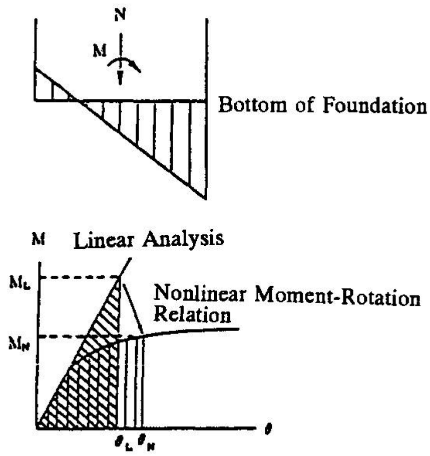


Fig.3 Evaluation of Nonlinear Response of Foundation considering the Separation between Foundation and Ground

Strain dependence of the soil stiffness is also considered in the design. The shear strain is obtained based on the horizontal response displacement and rotation angle of the foundation.

$$\gamma = \alpha \cdot \theta + \beta \cdot u_B/B \quad (4)$$

where, γ : shear strain, u_B and θ : response displacement and rotation angle of foundation, B : width of foundation, and α, β : coefficients. In the response analysis, the shear strain is given based on the relation between the shear strain and shear stiffness, G, or material damping, D, obtained by the material tests. The iteration analysis is made to achieve enough convergence of the shear strain.

The stability of the foundations is checked by :

1) Foundations should be supported by the stable stiff ground. Safety on bearing capacity and sliding capacity of ground should be checked and the displacement developed at the foundations should be within an acceptable limit so that the displacement of the foundations does not affect the superstructures.
2) Vertical soil reaction at the bottom of foundations should be within an acceptable limit of vertical bearing capacity.
3) Horizontal force at the bottom of foundation should be within an acceptable shear strength.

3.2 Design Methods against L2 Ground Motion

The check of the stability of the foundations against L2 Ground Motion is made by the elasto-plastic finite element analysis methods (FEM). The soil-foundation system is idealized as two dimensional finite element model and the foundation is modeled as the rigid mass with the concentrated mass at the center of the foundation or the elastic body with the distributed mass. The boundary condition of the FEM model of the ground is assumed as a viscous boundary at the bottom and the both sides. The nonlinearity of the soil is modeled as the modified model of the Ramberg-Osgood model. Integration for the response analysis is made in the time domain to consider the separation between foundation and ground and the nonlinear stress-strain relation of the soil. Through the nonlinear response analyses, the strain distribution and the stability of the soil element considering the failure criteria and the residual displacement are obtained. The stability of the foundation is checked for the most critical condition as follows:

1) Vertical soil reaction at the bottom of foundations should be within an acceptable limit of vertical bearing capacity.
2) Horizontal force at the bottom of foundation should be within an acceptable shear strength.

Proposed design methods in the above is based on the in-situ tests on the dynamic characteristics of soils and the field loading tests using large soil specimens. The analysis methods was used to check the safety of the Akashi Strait Bridge against the Hyogo-ken Nanbu Earthquake of 1995 [6].

5. Earthquake Response Control for Superstructures

5.1 Effect of Earthquakes on Member Forces

In the design of the superstructures, the effect of wind is generally the most critical design conditions. However, when the bridge with low damping characteristics are subjected to extremely strong earthquakes, the effect of earthquakes also becomes a critical issue to determine the structural member.

The effect of earthquakes on the member forces is studied through the dynamic analysis of a long-span suspension bridge with a center span length of about 2,000m. The analytical model is made based on the Akashi Strait Bridge. The earthquake response is computed by the response spectrum analysis using a linear three dimensional beam-mass model. The member forces obtained through the analysis is compared with those against wind loads. The damping ratio is assumed as 0.005 and the L2 Ground Motion which was given for the trial analysis in our study is used [1]. Wind velocity is assumed as 46m/s as a fundamental design wind velocity based on the Wind Resistant Design Manual for Akashi Strait Bridge.

Computed member forces and displacement is shown in **Table 1**. In the longitudinal direction, displacement of the stiffening girder developed during earthquakes is almost the same or slightly greater than that against the wind load. The member forces developed at the bottom of tower and mid-height section of tower during earthquakes is about two to three times those developed by the wind load. Therefore, they can be decreased by applying the damping augmentation devices. The member forces developed at the stiffening girder and the cable during earthquakes is enough small comparing these developed against the dead load and live load.

5.2 Effect of Damping Augmentation Devices

In the analysis above mentioned, the damping ratio is assumed as 0.005. The damping ratio for predominant modes is increased by the damping augmentation devices and the effectiveness to decrease the member forces is analyzed. The increased damping ratio is assumed as 0.055 and **Table 1** shows the effectiveness to decrease the member forces and displacement. If the damping ratio is increased form 0.005 to 0.055, the response can be decreased up to about half. The damping augmentation devices are designed so that the damping ratio is increased from 0.005 to 0.055. The damping coefficient is assumed as about 29,400kN•s/m and **Fig.4** shows the design examples of the devices using elasto-plastic effect of lead.

5. Conclusions

This paper presents the earthquake protective design and technology for the super-long-span bridges, including seismic design ground motions, innovative seismic design methods for foundations and earthquake response control technology for superstructures. These are our on-going study and the further study is being made including the development on evaluation methods of earthquake ground motions based on the active fault model, the feasibility of reinforced concrete tower against strong earthquake ground motions.

Table 1 Comparison of Member Forces between Earthquake and Wind Loads and the Effectivenss of Damping Augmentation Devices

	Location	Wind Load	Earthquake Load without Damper (A)	Earthquake Load with Damper (B) (B/A)
Displacement of Stiffening Girder	Side Span (Anchorage)	2.16 m	2.03 m	1.20 m (B/A=0.56)
	Center of Center Span	1.00 m	0.88 m	0.60 m (B/A=0.60)
Bending Moment of Tower	Bottom	1.89×10^6 kN•m	1.03×10^6 kN•m	1.23×10^6 kN•m (B/A=0.65)
	Mid-Height	7.25×10^5 kN•m	2.40×10^5 kN•m	not Analyzed

Acknowledgements

This study has been conducted under the guidance of the Investigation Committee on Strait Crossing Highway Projects in Japan (Chairman : Iwao YOSHIDA, President of Honshu-Shikoku Bridge Engineering Co. Ltd.). We sincerely thank Dr. Yoshida and all of the committee members for their encouraging guidance.

References

[1] Public Works Research Institute : Report of the Investigation Committee on Strait Crossing Highway Projects in Japan, Technical Memorandum of PWRI, No.3479, March 1986 (in Japanese)

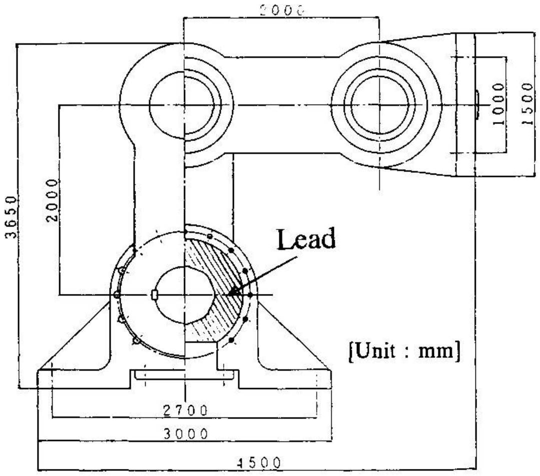
[2] Arakawa, T. et al. : Determination of Input Ground Motions for Dynamic Analysis Based on Probabilistic Methods, Technical Memorandum of PWRI, No.1992, 1983 (in Japanese)

[3] Hartzell, S. H. : Earthquake Aftershocks as Green's Function, Geophysical Research Letters, Vol.5, No.1, 1978

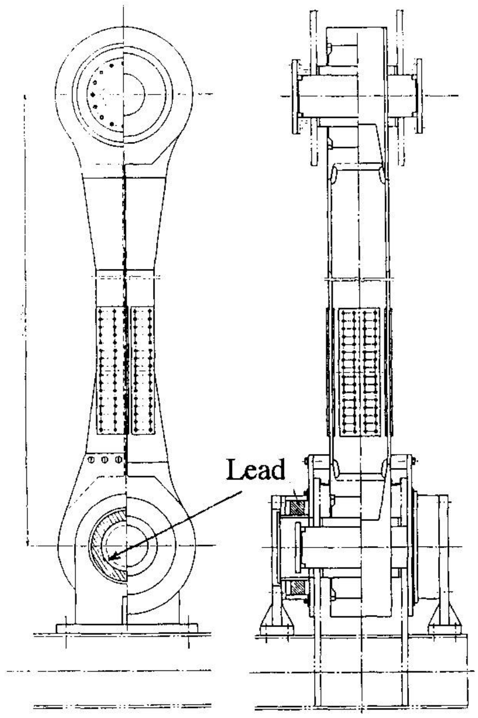
[4] Irikura, K. : Semi-Empirical Estimation of Strong Ground Motions during Large Earthquakes, Bulletin of Disaster Prevention Research Institute, Kyoto University, Vol.33, Part 2, No.298, 1983

[5] Midorikawa, S. and Kobayashi H., A : Semi-Empirical Method for Estimating Response Spectra of Near-Filed Ground Motions with Regard to Fault Rupture, Proc. of Seventh European Conference on Earthquake Engineering, 1982

[6] Honshu-shikoku Bridge Authority : Report of Committee on Seismic Design and Foundation Design, March 1997 (in Japanese)



(a) Rotational Lead Extrusion Damper



(2) Tower Link Rotational Lead Damper

Fig.4 Design Examples of Damping Augmentation Devices at Tower and Pier Connection