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SEISMIC RISK ANALYSIS OF PIER STRUCTURES AND PILE FOUNDATIONS OF EXISTING HIGHWAY BRIDGES

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SUMMARY

In this study, a load combination analysis and seismic risk analysis of rigid-frame pier and pile foundation system supporting three-span continuous box girder bridge is performed using the extended level 2 reliability method. Four typical types of pier structures with pile foundations are selected out of the existing actual bridge structures constructed on the Hanshin (Osaka-Kobe area) Expressway Network and are so modeled that they are amenable to analysis. Four actual load components (dead, live, temperature, and earthquake load) are considered. In numerical calculations, the safety indices of pier structures and pile foundations are calculated and compared.Further, some shortcomings inherent in the current ASDM are revealed.

INTRODUCTION

Nowadays, it is a world-wide tendency to introduce a Load Factor Design Method (LFDM) based on the reliability theory into the structural design standards instead of the current Allowable Stress Design Method (ASDM). In Japan, practical investigations have been started several years ago to introduce the LFDM into the design standards for highway bridges (Refs. 1, 2, 3).

In this study, with this situation in mind, a seismic risk analysis of pier structures and pile foundations of existing highway bridges is performed to reveal some shortcomings inherent in the ASDM. Four typical types of rigid-frame pier and pile foundation system supporting three-span continuous box girder bridges constructed on the Hanshin Expressway Network are selected and are so modeled that they are amenable to analysis. Based on the historical data on earthquakes in Hanshin (Osaka-Kobe) area, the actual earthquake load (E) is modeled by the limiting spike type of Borges-Castanheta (B-C) load model. In the same way, based on the observation data on the Hanshin Expressway Network, the actual dead load (D), the actual live load (L) and the actual temperature load (T) are modeled by the mixed type of B-C load model. For various load combination cases contained the earthquake load, the safety indices are calculated for these models of rigid-frame pier and pile foundation system using the extended level 2 reliability method. And some shortcomings inherent in the current ASDM are revealed.

MODELING OF PIER AND FOUNDATION SYSTEMS

As was stated above, the four typical types of rigid-frame pier and pile foundation systems were selected out of the existing systems on the Hanshin Expressway Network. These four systems were selected by considering the combination of two basic parameters, i.e. the total height of pier H=10, 20m and the total width of pier W=20, 30m. The principal dimensions and the configuration of the systems are listed in Tables 1 and 2 and demonstrated in Figs.1 through 3. These systems were designed by the conventional allowable design formats shown in Table 3 (Ref. 4). In Table 3, D_n, L_n, T_n, E_n = the nominal value of each load component D, L, T and E, respectively; α_D, α_L, α_T, α_E = factors which convert each load component into corresponding stress level; and ϕ = the augmentation factor of the allowable stress.

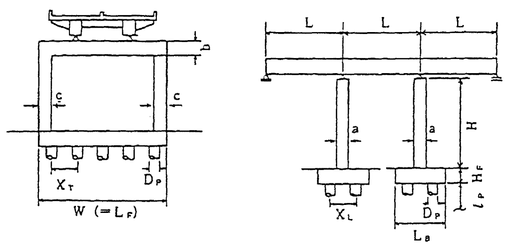


Fig. 1 Rigid-Frame Pier and Pile Foundation System Supporting Three-Span Continuous Box Girder Bridge

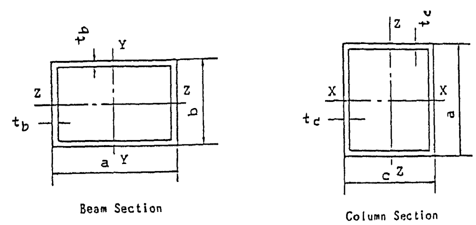


Fig. 2 Cross Section of Rigid-Frame Pier

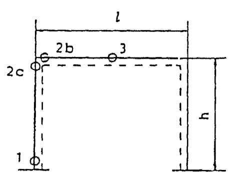


Fig. 3 Analytical Model of Rigid-Frame Pier

Table 1 Four Models for Pier Structure

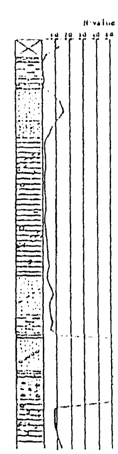
Model No.	L (m)	H (m)	W (m)	h (m)	l (m)	a (m)	b (m)	c (m)	t_b (mm)	t_c (mm)
1	40.0	10.0	20.0	9.17	18.5	2.00	1.67	1.5	22.5	28.8
2	40.0	10.0	30.0	8.75	28.0	2.00	2.50	2.0	29.0	39.3
3	40.0	20.0	20.0	19.17	18.0	2.00	1.67	2.0	20.9	2.30
4	40.0	20.0	30.0	18.75	27.5	2.00	2.50	2.5	24.0	26.6

Table 2 Four Models for Pile Foundation

Model No.	dimensions of footing $L_F \times L_B \times H_F$ (m×m×m)	dimensions of piles			c. to c. distance between piles		number of piles	vertical limit bearing capacity (t)	natural period of pier structure	
		D_P (mm)	t_P (mm)	l_P (m)	x_T (m)	x_T (m)			longitudinal (sec)	transverse (sec)
1	20.00×7.52×5.00	752	14.0	33.6	2.50	1.88	8×4=32	455.1	0.5	0.5
2	30.00×9.45×7.50	756	14.0	31.1	2.50	1.89	12×5=60	475.5	0.5	0.5
3	20.00×10.55×5.00	844	14.0	36.6	2.50	2.11	8×5=40	510.7	0.7	1.0
4	30.00×12.75×7.50	850	14.0	31.1	2.50	2.13	12×6=72	514.4	1.0	1.0

Table 3 Current Design Fomulas

Code	Current Design Formulas	ϕ
1	$\alpha_D \cdot D_n + \alpha_L \cdot L_n \leq \phi \cdot \sigma_a$	1.00
2	$\alpha_D \cdot D_n + \alpha_L \cdot L_n + \alpha_T \cdot T_n \leq \phi \cdot \sigma_a$	1.15
3	$\alpha_D \cdot D_n + \alpha_E \cdot E_n \leq \phi \cdot \sigma_a$	1.50
4	$\alpha_D \cdot D_n + \alpha_T \cdot T_n + \alpha_E \cdot E_n \leq \phi \cdot \sigma_a$	1.70



design value			
depth of stratum (m)	N-value	τ (t/m³)	f (t/m²)
2.40	12	1.4	0
4.00	0	1.6	0
6.30	10	1.7	0
12.00	2	1.5	2.0
4.70	4	1.6	4.0
8.20	6	1.8	1.2
8.00	50	2.1	10.0
5.80	8	1.6	16.0

Fig. 4 Boring Log Foundation

The characteristics of steel material used for pier structure are as follows; the grade of steel = SM50Y; the allowable stress $\sigma_a = 2100\ kgf/cm^2$; the yield stress $\sigma_y = 3600 kgf/cm^2$; the Young's modulus $E_o = 2.1\times10^6 kgf/cm^2$; the linear coefficient of expansion $\alpha = 1.2\times10^{-5}/^{o}C$; and the unit weight $\rho = 7.85\times10^{-3} kgf/cm^3$. The characteristics of steel pile are as follows; the grade of steel = STK-41; and the allowable stress $\sigma_a = 1400 kgf/cm^2$. The boring log of foundation is shown in Fig. 4.

MODELING OF ACTUAL LOAD COMPONENTS

Based onthe historical data on earthquakes in Hanshin area, the actual earthquake load E is modeled by the limiting spike type of Borges-Castanheta (B-C) load model. In the same manner the actual dead load D, the actual live load L and the actual temperature load T are modeled by the mixed type of B-C load model. Based an extensive investigation on actual conditions of various loads acting on urban expressway bridges (Ref. 1), the characteristics of the B-C process of each load component are determined as follows:

Earthquake Load, E   Actual earthquake load is modeled as $E=S_A/g$, where S_A= linear acceleration response spectrum; and g= acceleration of gravity. The cumulative distribution function (CDF) of S_A is expressed as

for natural frequency of structure =0.5sec

$$F_{S_A}(x) = 1 - \exp[-\{(x-41.28)/34.24\}^{0.913}] \quad (41.28 < x)$$

for natural frequency of structure =0.7sec

$$F_{S_A}(x) = 1 - \exp[-\{(x-25.88)/26.12\}^{0.879}] \quad (25.88 < x) \qquad (1)$$

for natural frequency of structure =1.0sec

$$F_{S_A}(x) = 1 - \exp[-\{(x-17.19)/18.05\}^{0.850}] \quad (17.91 < x)$$

In evaluation of Eq.(1), the occurence of earthquake is assumed to be Poisson process, and its average return period is considered to be greater than 2 years. Furthermore, it is assumed that the magnitude is greater than 5.0, the ground condition is Grade 2 and the damping ratio of structure is 0.05. The uncertainty of attenuation law is not considered.

Dead Load, D   As dead load D, only the own weight of structure is considered and it is assumed to be deterministic. To take its variability into consideration, however, the design value of D is calculated by the formula $D=D'(1+\delta)$, where D'=actual weight of the structure calculated on the basis of the unit weight of the material and the volume of the members; and δ =0.05 for the superstructure, 0.10 for pier structure and 0.0 for pile foundation.

Live Load, L   The actual live load is modeled as the support reaction on the piers by using the Monte-Carlo simulation technique. The probability of occurence, p, and the basic time intervals, τ_L, are taken as 0.75 and 6 hours, respectively.Given that the load occurs the CDF of its amplitude, $F_L^*(x)$, is expressed as

$$F_L^*(x) = 1 - \exp[-(x/56.49)^{2.342}] \quad (x > 0\ ;\ \text{unit:ton}) \qquad (2)$$

This CDF is evaluated for two supports on the pier.

Temperature Load, T   Actual temperature load is modeled as the temperature difference such that actual temperature of structure minus $15^{o}C$. The parameters p and τ_T are taken as 0.75 and 6 houres, respectively. The CDF of the temperature difference, $F_T^*(x)$, is expressed as

$$F_T^*(x)=0.5+0.5\,\Phi\{(x-13.2)/4.4\} \qquad (x>0\ ;\ \mathrm{unit:}^\circ\mathrm{C}) \tag{3}$$

LOAD COMBINATION ANALYSIS AND SEISMIC RISK ANALYSIS

In load combination analysis and seismic risk analysis of the model rigid-frame pier and pile foundation systems under combined action of the earthquake, dead, live and temperature load components, the Turkstra's rule in connection with the B-C processes (Ref. 5) and the extended level 2 reliability method based on Rackwitz-Fiessler's procedure (Ref. 6) are used.

Evaluation of Reliabilty of Pier Structure   Assuming that the stress σ^* in the ultimate limit state of member is the yield stress σ_y=3600(kgf/cm^2), the safety index β is evaluated as

$$\beta=(\sigma^*-\sigma_D-\sum_{i=1}^{3}C_{X_i}\cdot\mu_{X_i}')/\sum_{i=1}^{3}C_{X_i}\cdot\sigma_{X_i}'\cdot\alpha_i \tag{4}$$

$$\left.\begin{aligned}
&\alpha_i=C_{X_i}\cdot\sigma_{X_i}'/\kappa\\
&\kappa=\Big(\sum_{i=1}^{3}C_{X_i}{}^2\cdot\sigma_{X_i}'^2\Big)^{1/2}\\
&\mu_{X_i}'=x_i^*-\Phi^{-1}\{F_{X_i}(x_i^*)\}\cdot\sigma_{X_i}'\\
&\sigma_{X_i}'=\frac{\phi[\Phi^{-1}\{F_{X_i}(x_i^*)\}]}{f_{X_i}(x_i^*)}\\
&x_i^*=F_{X_i}{}^{-1}\{\Phi(\beta\cdot\alpha_i)\}
\end{aligned}\right\} \tag{5}$$

where X_1, X_2 and X_3= live,temperature and earthquake load, respectively; x_i^*= X_i-coordinate of the design point; F_{X_i} and f_{X_i} = the CDF and the PDF of X_i , respectively; $\Phi(\cdot)$ and $\phi(\cdot)$= the standard normal distribution function and density function, respectively; C_{X_i}= the factor which converts the load X_i into the stress level; and σ_D = the deterministic stress for the dead load. By solving the Eqs.(4) and (5) iteratively for β , the safety index β for the four checking points 1, 2c, 2b and 3 (see Fig. 3), can be determined. In this study , the minimum value among the safety indices for these four checking points is considered to be the safety index of the pier structures.

Evaluation of Reliability of Pile Foundation   In this study, the failure event of pile foundation is defined by $R_u-P_N<0$; where P_N = vertical force acting on the pile top; and R_u= vertical ultimate bearing capacity of pile. Herein, R_u is formulated as $R_u=\alpha_R\cdot R_n$, where R_n= vertical ultimate bearing capacity evaluated by design formula; and α_R= factor which considers the uncertainty of R_n.Based on the data on pile loading test (Ref. 7), α_R is assumed to be log-normal random variable with the mean value μ_α=0.99 and the coefficient of variation V_α=0.5. According to the definition of failure event mentioned above, the safety index β is evaluated as

$$\beta=(Ag+Bg\cdot\mu_{x2}'-R_n\cdot\mu_{x1}')/(R_n\cdot\sigma_{x1}'\cdot\alpha_1-Bg\cdot\sigma_{x2}'\cdot\alpha_2) \tag{6}$$

$$\left.\begin{aligned} &\alpha_1 = -R_n \cdot \sigma_{x1}'/k \quad , \quad \alpha_2 = Bg \cdot \sigma_{x2}'/k \\ &k = (R_n^2 \cdot \sigma_{x1}'^2 + Bg \cdot \sigma_{x2}'^2)^{1/2} \end{aligned}\right\} \qquad (7)$$

where X_1 and X_2 = α_R and earthquake load, respectively; μ_{Xi}' and σ_{Xi}' = the same expressions in Eq.(5); and A_g and B_g = the factors which evaluate the vertical force acting on the pile top. In evaluation of β for pile foundation, the live load L and temperature load T are not considered.

NUMERICAL EXAMPLES

Reliability of Pier Structures The safety index β is calculated for each of the four model pier structures shown in Table 1. The results obtained in transverse direction are shown in Fig. 5. As it is seen from this result, the safety index β considerably differs from each other, depending on the model type of pier structure. For the models No.3, β takes comparatively small value, while for the model No.2, β is very large. The reason for this difference lies in the augmentation factor ϕ =1.50 or 1.70 (see Table 3) of allowable stress which is used when earthquake is to be considered. According to the current design code which is based on the ASDM, the allowable stress is uniformly augmented regardless of the type of structure. On the other hand, as the model No.3 is affected significantly by actual load effects due to earthquake, the β value becomes smaller than those of other models. The current design code does not insure consistent level of safety for different type of pier structures subjected to earthquake load.

Reliability of Pile foundations The safety index β is calculated in longitudinal direction as well as transverse direction. The results are shown in Fig.6. Fig.6 shows that the β values in longitudinal direction are smaller than those in transverse direction. The safety indices of the four model foundations do not so much differ from each other compared with those of the corresponding pier structures shown in Fig.5. This is due to the fact that for pile foundations the load effect due to dead load is considerably large compared with that of earthquake load. Further, it is seen from comparing Fig.5 with Fig.6 that the pile foundations have small values of safety indices compared with those of the corresponding pier structures. The reason lies in the fact that the ultimate bearing capacity of pile has a considerably large coefficient of variation, while the ultimate strength of pier structure is considered to be deterministic in this study.

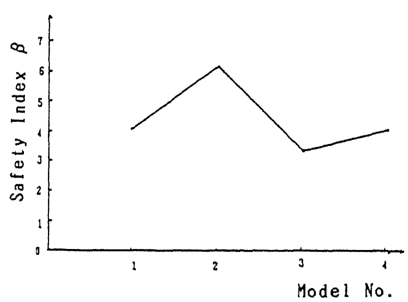


Fig. 5 Safety Index β of Pier Structures

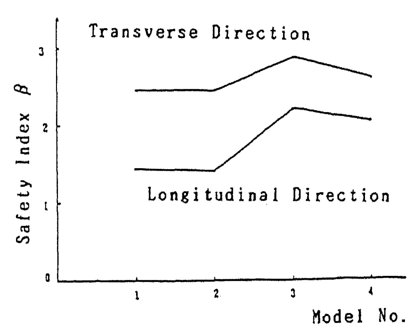


Fig. 6 Safety Index β of Pile Structures

SUMMARY AND CONCLUSIONS

A load combination and seismic risk analysis of steel rigid-frame pier and pile foundation systems of existing highway bridges was performed, and the safety indices were evaluated. Furthermore, some shortcomings inherent in the current ASDM were revealed. The main results are as followes;

1) The safety indices of pier structures considerably differ from each other, depending on the model type of pier structures.
2) The pier structures have large values of safety indices compared with those of the corresponding pile foundation.
3) The current design code dose not insure consistent level of safety for different type of structures subjected to earthquake load.

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