Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.VI)

10-2-17

EVALUATION OF GUIDELINES FOR SEISMIC DESIGN OF HIGHWAY BRIDGES

Michel GHOSN[1], Shuichi NISHIHASHI[2] and
Masanobu SHINOZUKA[3]

[1]Deparment of Civil Engineering, The City College of New York, New York, N.Y. 10031
[2]Taisei Corporation, Tokyo, Japan
[3]Department of Civil Engineering, Princeton University, Princeton, NJ. 08544

SUMMARY

This paper presents a preliminary review of the adequacy of the recommended AASHTO specifications for the design and analysis of highway bridges. Bridge models designed according to AASHTO's recommendations are analyzed using time step dynamic analyses for different earthquake records. The safety inherent in the recommended design procedure is analyzed using a simple second order reliability model. The deterministic and the reliability analyses indicate that the AASHTO procedures provide adequate safety for the seismic design of bridge columns.

INTRODUCTION

The significant damage exhibited by highway bridges due to the San Fernando earthquake led to increased interest in studying the behavior of highway bridges under seismic loads (Ref. l). The first implementation of these studies in design codes was in the CalTrans revisions to AASHTO's specifications relating to seismic design (Ref. 2). Subsequently, the Federal Highway Administration in association with Applied Technology Council developed new seismic design guidelines applicable to all regions of the United States (Ref. 3). These guidelines were finally adopted by AASHTO in 1983 (Ref. 4).

This paper reviews the AASHTO recommendations by analyzing a typical bridge designed according to the guidelines and subjected to various simulated earthquakes. A simple reliability model is used to study the safety of the recommended AASHTO design methodology. The analysis is based on the statistical information about the behavior of bridge structures under earthquake loading obtained from the bridge model analysis. The limit state probability is used as a measure of the safety of the bridge piers under earthquake load. This can be defined as:

$$P_f = Pr\left(\frac{M_R}{M_T} < 1\right) \qquad (1)$$

Where: P_f = Probability of failure, M_R = The moment resisting capacity of a bridge column, M_T = The applied total moment under earthquake loading. If both M_R and M_T are lognormally distributed, then:

$$P_f = \Phi\left[\frac{-\ln\left(\frac{\widetilde{M}_R}{\widetilde{M}_T}\right)}{\left(\sigma^2_{\ln M_R} + \sigma^2_{\ln M_T}\right)^{1/2}}\right] \qquad (2)$$

Where: $\widetilde{M}_R$ = median value of M_R, $\sigma_{\ln MR}$ = standard deviation of ln M_R , and Φ = cumulative normal distribution function.

The safety index β is defined such that:

$$P_f = \Phi(-\beta) \tag{3}$$

or

$$\beta = -\left[\frac{\ln\left(\frac{\overline{M}_T}{\overline{M}_R}\frac{\sqrt{V_{M_R}^2+1}}{\sqrt{V_{M_T}^2+1}}\right)}{\sqrt{\ln\left(\left(V_{M_R}^2+1\right)\left(V_{M_T}^2+1\right)\right)}}\right] \tag{4}$$

where: V_{MR} = coeficient of variation of M_R and $\overline{M}_R$ = mean of M_R.

RELIABILITY MODEL

For the bridge under earthquake loading, the resistance M_R is the capacity of a bridge column to sustain the moment due to the earthquake load. The bridge columns are designed to resist a moment such that:

$$M_R = \frac{\gamma(M_E + M_D)}{\varphi} \tag{5}$$

where M_E is the moment due to the earthquake load and M_D is the dead load moment. γ is the load combination factor and is equal to 1.0 (Ref. 4). ϕ is the strength reduction factor and varies from 0.5 to 0.9 depending on the axial load. (Ref. 4).

AASHTO stipulates that the moment capacity is to be calculated from an elastic analysis and to account for member "ductility" and "risk" by dividing the elastic moment by a response modification factor R. If M_{el} is the moment applied on the member assuming elastic behavior under the earthquake design spectra and assuming that the columns are fixed at their bases and the bridge deck is rigid, then:

$$M_E = \frac{M_{el}}{R} = \frac{P_e hL}{2R} \tag{6}$$

where: P_e = equivalent static load per unit length, h = the column height, L = the bridge length, and:

$$P_e = WC_s = \frac{W\,1.2\,AS}{T^{2/3}} \tag{7}$$

where: $C_S = \frac{1.2\,AS}{T^{2/3}}$ is a dimensionless elastic seismic response coefficient, W = weight per unit length, A = acceleration coefficient , S = dimensionless coefficient for soil profile, and T = period of the bridge.

$$M_R = \frac{\left(M_D + \frac{W\,1.2\,AS}{T^{2/3}}\cdot\frac{hL}{2}\cdot\frac{1}{R}\right)}{\varphi} \tag{8}$$

Shinozuka et al (Ref. 5) assume that the general format of equation (7) for the equivalent static load is correct and that all the variables of equation (7) follow lognormal distributions. Accordingly, the median of load P becomes:

$$\tilde{P} = \frac{1.2\,\tilde{S}\,\tilde{W}}{\tilde{R}\,(\tilde{T})^{2/3}}\,\tilde{A} \tag{9}$$

and,

$$\sigma_{\ln P} = \left[\sigma^2_{\ln S} + \sigma^2_{\ln W} + \sigma^2_{\ln R} + \left(\tfrac{2}{3}\right)^2 \sigma^2_{\ln T} + \sigma^2_{\ln A}\right]^{1/2} \tag{10}$$

where: $\sigma_{\ln P}$ = standard deviation of log of P = $\sqrt{\ln (V^2_p + 1)}$, $\tilde{P}$ = median of P = $\bar{P}$ exp. $(-1/2\ \sigma_{\ln P})$, $\bar{P}$ = mean of P, V_P = coefficient of variation of P. Shinozuka et al. (Ref. 5) establish a relationship between the median values of the random variables of equation (7) and the design values as given in ATC. They suggested to use $\tilde{T}$ = 0.91 T and $\sigma_{\ln T}$ = 0.35 based on the recommendation of Haviland (Ref. 6). $\tilde{W}$ = 1.05 W and $\sigma_{\ln W}$ = 0.10 are used based on the work of Ellingwood et al. (Ref. 7). They also used $\tilde{S}$= S and $\sigma_{\ln S}$ = 0.3 for soil profile coefficient. The statistics of R can be obtained by performing an elasto-plastic analysis of typical bridge columns and comparing the results to the results of the elastic analysis. The acceleration coefficient A is a function of the site location and will be treated as a variable in this paper.

EVALUATION OF RESPONSE MODIFICATION FACTOR R

To obtain an estimate of the actual relationship between the assumed elastic forces and the elasto-plastic forces of bridge columns, a bridge designed according to AASHTO'S specifications is analyzed. The bridge dimensions are given in Figure 1. The model is based on the bridge given in the worked example of the AASHTO guide. The computer program, ERDARCS (Ref. 8) was used for the elasto-plastic analysis.

The model was first subjected to a gound motion input based on the El-Centro earthquake with a maximum acceleration = 0.40 g acting in the axial direction. Analysis of the elasto-plastic structure with the material properties given in Figure 1 yielded a maximum bending moment ≈ 16,200 per bent (or about 5400 per column). The analysis assuming elastic behavior gives a maximum moment of 38,000 kip-ft per bent. The total axial force on each column yields a stress larger than 0.2 f'_c and the strength reduction factor ϕ recommended by AASHTO is 0.50. The unfactored response modification factor obtained based on the computer analysis is R = 2.3. However, if the strength reduction factor ϕ is applied then the required design section capacity that would produce an ultimate capacity per column = 5400- kip-ft is 2700 kip-ft. When comparing the calculated elastic response to the design elasto-plastic capacity the response modification factor R* becomes equal to 4.7. This value compares well to the AASHTO recommended value (R=5) for the design of bridge columns of multiple-column bents.

To analyze the sensitivity of the results to the type of earthquake input used, several artificial earthquakes were generated based on a Monte-Carlo simulation of the Kanai-Tajimi earthquake spectrum as developed by Shinozuka, et al. (Ref. 9). A total of 30 artificial earthquakes were generated such that their average maximum accleration is 0.4 g. For each earthquake an elastic analysis of the basic bridge model is performed, and the results are compared to those of the elasto-plastic response. The ratio of the maximum moment obtained assuming purely elastic behavior to the maximum elasto-plastic moment is calculated for each generated earthquake. The average R ratio calculated was found to be 1.93 with a standard deviation of 0.386 which corresponds to a coefficient of variation V_R= 20%.

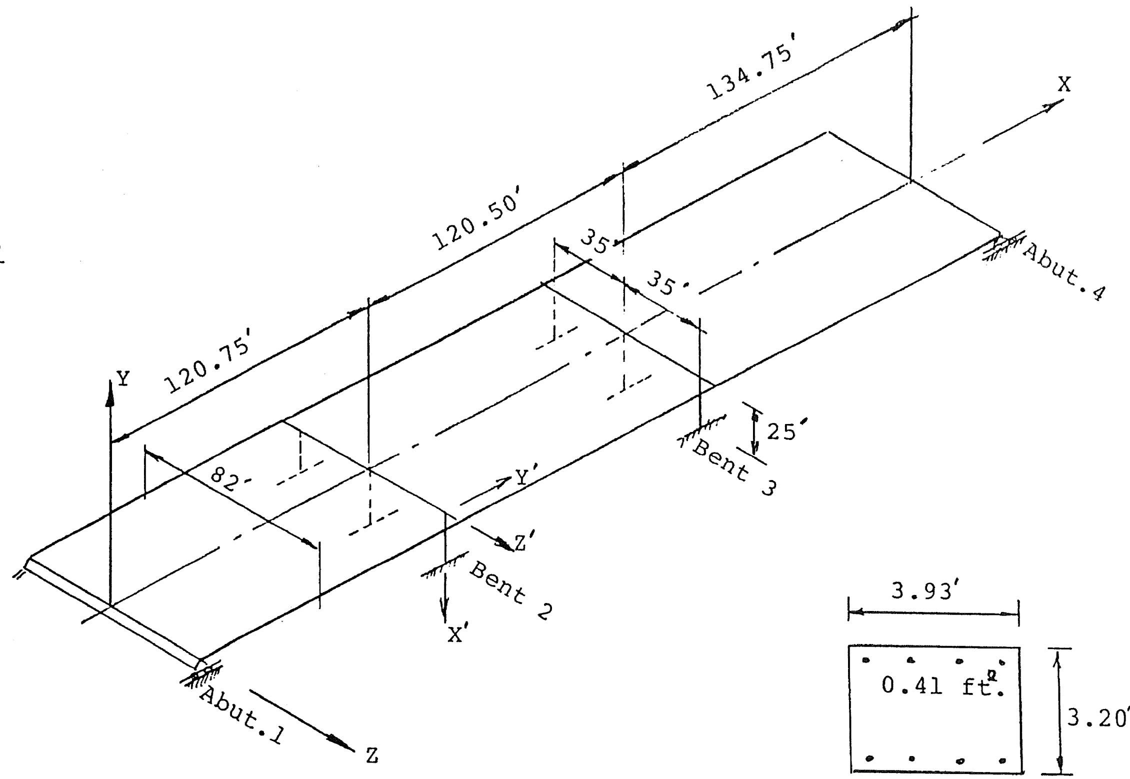
Results of dynamic analysis Several additional models with different geometries and assumed boundary conditions were also analyzed. The results of the dynamic analysis as given in reference 10 can be summarized as follows: a) The analysis and design recommendations for multiple-bent bridge column design

SUPERSTRUCTURE

$L = 376$ ft.
$A_x = 123$ ft^2.
$I_x = 117$ ft^4.
$I_y = 65550$ ft^4.
$I_z = 527$ ft^4.
$f_c = 3250$ psi
$E_c = 3{,}000{,}000$ psi

SUBSTRUCTURE

$H = 25$ ft.
$A = 13$ ft^2.
$I_x = 26$ ft^4.
$I_y = 13$ ft^4.
$I_z = 13$ ft^4.
$f_c = 3250$ psi
$E_c = 3{,}000{,}000$ psi



Column Section

Figure 1- Dimensions of Bridge Model

as given by AASHTO are adequate when compared to an elasto-plastic analysis of a typical 25 ft. high bridge model under El-Centro earthquake record scaled to produce 0.4 g maximum acceleration. The AASHTO recommended R value seems to be slightly higher than the computed values when design safety factors ϕ are included in the calculation. b) The assumption that the maximum displacement due to elastic and elasto-plastic analysis are equal is not always correct: The error is on the order of 10% -25%. c) The results are very sensitive to the variation in the earthquake input, in fact, due to 30 artificial earthquakes, the average R value is equal to 2.0 and the coefficient of variation of R is 20%. d) The results are sensitive to the assumed end conditions and the average value of R and its C.O.V. vary considerably as a function of the assumed flexibility of the column foundation.

RELIABILITY ANALYSIS

The reliability model developed above was used with the results of the dynamic analysis to evaluate the inherent safety of the AASHTO guidelines for seismic design. Several values of $\tilde{A}$ and $\sigma_{\ln A}$ are used and the results are given in Fig. 2. For $\tilde{A} = 0.01$, β varies between 5.62 and 3.10 depending on $\sigma_{\ln A}$. This value of A however corresponds to regions of little seismic activity. For $\tilde{A} = 0.1$, which respresents regions of high risk, β ranges between 2.36 and 1.34 depending on $\sigma_{\ln A}$. These calculations are executed forR=2.0 and $\sigma_{\ln R} = 0.2$. $\sigma_{\ln R} = 0.2$ however, accounts for the uncertainty in R due to the characteristics of the earthquake input. To account for the uncertainty in material and modeling, a higher $\sigma_{\ln R}$ should be used . When $\sigma_{\ln R}$ varies from 0.2 to 0.4, β drops to range between 1.95 and 1.25 for $\tilde{A} = 0.1$. Figure 3 shows the variation of the safety index β for the case where $\tilde{A} = 0.1$ and different $\sigma_{\ln R}$ values.

It is to be noted here, that most design codes stipulate a safety index in the range between 2.5 and 3.5 for main members with usually higher values for connections. Slightly lower β values are usually accepted for extreme environmental loads, since providing high safety levels for these rare events is uneconomical. From this limited analysis one can observe that the current AASHTO procedure seems to provide acceptable safety levels for seismic regions where the median of the annual extreme peak ground acceleration is equal to or lower than 0.1 g ($\tilde{A} \leq 0.1$). This is based on the assumption that the safety index should not reach a value less than 1.5.

ACKNOWLEDGEMENT

This work was partially supported by sub-contracts nos. NCEER 873020 and 871006 under the auspices of the National Center for Earthquake Engineering Research (Master NSF contract No. ECE 8607591). The authors are also grateful to Taisei Corporation, Tokyo, Japan for its support of this work.

REFERENCES

1. Barenberg, M. and Foutch, D., "Inelastic Response and Damage Assessment of Reinforced Concrete Highway Bridges Subjected the Seismic Loadings", Civil Engineering Studies No. 528, University of Illinois at Urbana-Champaign, 1986.
2. "Standard Specifications of Highway Bridges Relating to Seismic Design" AASHTO 1977 with revisions by Caltrans, July, 1984.
3. "Seismic Design Guidelines for Highway Bridges", ATC-65, Applied Technology Council, Berkeley, CA, October, 1981.
4. "Guide Specifications for Seismic Design of Highway Bridges", AASHTO, 1983.
5. Shinozuka M., Hwang, H. and Ushiba, H., "Natural Hazard Mitigation on Low-Rise Buildings". Proc. of U.S.-Korea Seminar on Critical Engineering Systems, Seoul, Korea, May, 1987.

6. Haviland, R., "A Study of the uncertainties in the Fundamental Translational Periods and Damping Values of Real Building", M.I.T., Cambridge, Massachusetts, 1976.
7. Ellingwood, B., et al, "Development of Probability-Based Load Criterion for American National Standard A58", NBS SP 577, National Bureau of Standards, Washington, D.C.
8. Shinozuka, M., and Chung, Y.S., "ERDARCS computer program for Earthquake Response and Damage Analysis of Reinforced Concrete Structures", Columbia University, 1987.
9. Shinozuka, M., Yamazaki, F. and Mizutani, M., "Elasto-plastic Response of Multi-Degree of Freedom Systems", ASCE National Convention, Seattle, WA, 1986.
10. Ghosn, M., Nishihashi, S. and Shinozuka, M. "Safety Analysis of Highway Bridge Under Earthquake Loads", Report to NCEER, in progress.

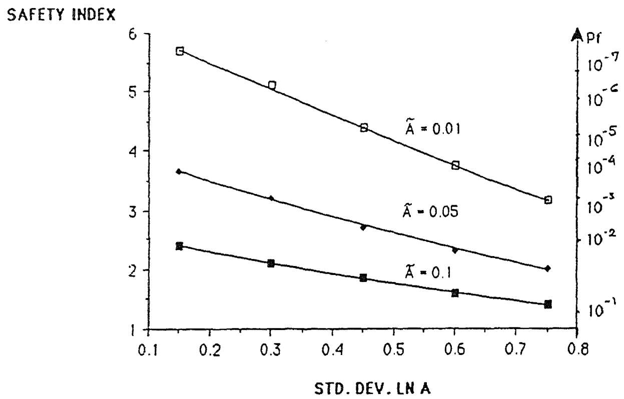


Figure 2 Sensitivity of Safety Index to Site Intensity

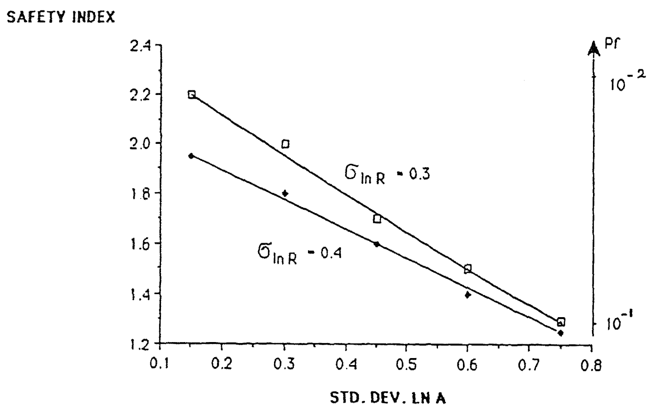


Figure 3 Sensitivity of Safety Index to Uncertainty in Reduction Factor