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

10-2-16

ANALYSIS AND DESIGN OF HIGHWAY BRIDGES

A. GHOBARAH

Department of Civil Engineering and Engineering Mechanics
McMaster University, Hamilton, Canada

SUMMARY

A study was conducted to establish general guidelines for the applicability of simplified design procedures and code type formulas in the analysis and design of base-isolated highway bridges. Several single and multiple span bridges were analyzed when subjected to strong motion earthquake records. The solutions obtained using complex dynamic analyses are compared with simplified design and code formulas.

It is concluded that simplified code type design approach is adequate as long as the bridge system can be approximately represented by a single degree of freedom model. This is particularly true in the case of a symmetrical bridge configuration with rigid deck which represent the majority of highway bridges.

INTRODUCTION

Bridges provide the essential link in the lifeline transportation system for fire fighting, emergency rescue, and evacuation operations following a major disaster such as earthquakes. It is critical that bridges remain operational during and following a devastating earthquake. During recent earthquakes, bridges in Japan and the United States suffered heavy damage. Okubo and Iwasaki (Ref. 1) summarized the general features of seismic damage to bridges in Japan. They noted that most of the damage was caused by weakness of supports and failure of the substructure or surrounding soil. Following the 1971 San Fernando earthquake, Gates (Ref. 2) noted that significant damage was due to vibrational effects of the superstructure. Some damage was also sustained by the supporting structures. Seismic forces on the bridge structural components exceeded those anticipated in the design. The design procedures that were in use before the San Fernando earthquake proved inadequate. Design codes are now being constantly revised to improve the seismic performance of bridges.

With the increased seismic loads, the design of highway bridges to remain elastic during major earthquakes may be uneconomical. One way to achieve elastic behaviour of bridge components such as piers is to use base-isolation techniques. The principal advantages of these devices is to shift the fundmental period of the structure away from the high energy periods of the earthquake. With period shift to the long period range, the structure will be subjected to less acceleration and less seismic forces. In such a case, the shear forces transmitted to the piers and abutments are reduced. Under these conditions the substructure can be designed to remain elastic. In addition to the period shift advantage, some base-isolation systems provide energy dissipating mechanisms through the hysteresis behaviour of the isolator. Some of the base-isolation systems and examples of design for bridges are reviewed by Blakely (Ref. 3). One of the efficient base-isolation systems is the lead-rubber bearing shown in Fig. 1.

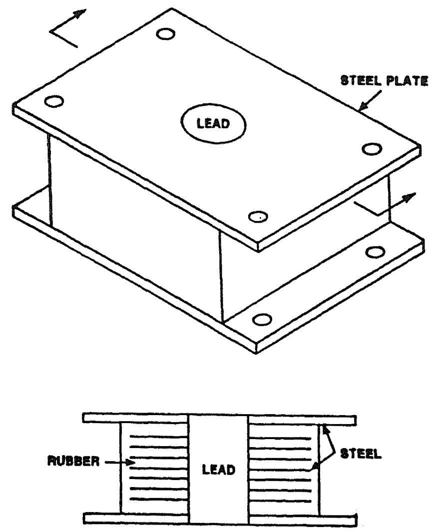

Fig1. Lead-rubber bearing

The use of base-isolation gives the designer some freedom in trading off reduced shear forces on the substructure with increased deck displacements. In addition, base-isolation provides a degree of flexibility in controlling the magnitude of forces transmitted to piers and abutments. This feature is advantageous in retrofitting existing bridges to withstand higher seismic design loads. In many cases, it is possible for abutments to carry additional forces while it is desirable not to increase the forces transmitted to the piers. In the design of base-isolated highway bridges, the quantities of interest are the fundamental period, the seismic forces transmitted to piers and abutments and the maximum bridge displacements. Available analysis and design procedures vary from the complex dynamic analysis to the simplified and code type approaches (Refs. 4 and 5). At present, no guidelines are established as to when various simplifying assumptions are applicable and the implications of parameter variations on the resulting design.

The objective of this study is to establish general guidelines for the applicability of simplified design procedures and code type formulas in the analysis and design of base-isolated highway bridges.

BRIDGE MODEL

The bridge structure is modelled by a series of beam elements. The earthquake ground motion is taken to act in either the longitudinal or the transverse directions. The vertical earthquake component is not taken into account. Because of the ground motion assumptions, the displacements in the transverse and longitudinal directions are assumed uncoupled. Each node in the model is allowed two degrees of freedom. One of the degrees of freedom is displacement in the horizontal direction, either longitudinal or transverse, and the second is rotation about the vertical axis. The bridge deck is assumed to be continuous over the piers. It is also assumed that there is no significant axial shortening of the pier. The piers and abutments are taken to rest on rock or stiff soil.

The bridge base-isolation system is taken to be the lead-rubber bearing. The behaviour of the bearing is represented by a bilinear hysteretic model. The equations of motion (Ref. 6) are numerically evaluated using the Wilson-θ method. The method is a step-by-step integration procedure that applies the linear acceleration assumption over extended computation time interval.

In the analysis of the bridge behaviour, artificially generated time histories are used as input ground motion. The time histories are generated based on actual strong motion records. The response spectrum of the actual earthquake is modified to match the elastic spectrum used by the California Department of Transportation (Caltrans) with 0.6 g maximum acceleration on rock site (Ref. 2). The ordinates of the computed spectrum are increased or decreased to match the target Caltrans spectrum. This process is performed iteratively by modifying the Fourier coefficients. The use of actual earthquake records takes into account the uncertainty in real earthquake characteristics. Some of the actual earthquake records used include El Centro 1940 S90W, Taft 1952 S69E and San Fernando 1971 S74W components. The generated time histories are then normalized to give maximum acceleration of 0.6 g or 5.886 m/s^2. The use of artificially generated time histories based on actual earthquake records is not essential, however, it serves the purpose of comparing bridge responses to various earthquake inputs that have the same given response spectrum. Normalizing the maximum acceleration to 0.6 g is arbitrary and does not affect the conclusions of the study.

DECK RIGIDITY

The lateral vibration of the bridge deck as a beam supported on hinged or roller supports can be evaluated by using simplified methods such as the single mode spectral analysis (Ref. 7). This approach is adequate for most highway bridges. However, for the case of a bridge deck supported on base-isolation bearing system, another degree of freedom is introduced. The deck of a typical two or four lane overpass is fairly rigid in its own plane with a high frequency of free vibration. With base-isolation the fundamental frequency of free vibration of the coupled deck, bearing system becomes much lower. The coupled system respresents the vibration of the bridge deck as a beam in its own plane and the vibration of the deck as the rigid body with the bearing pads acting as springs. If there is a wide frequency spread between the two modes of vibration, the system can be uncoupled. Decoupling the system will simplify the analysis procedure. In this case, the bridge deck-support bearing system can be represented by a single degree of freedom model by assuming the deck to be rigid in its own plane. Rigid deck assumption becomes an important consideration in simplifying design procedures for base-isolated highway bridges.

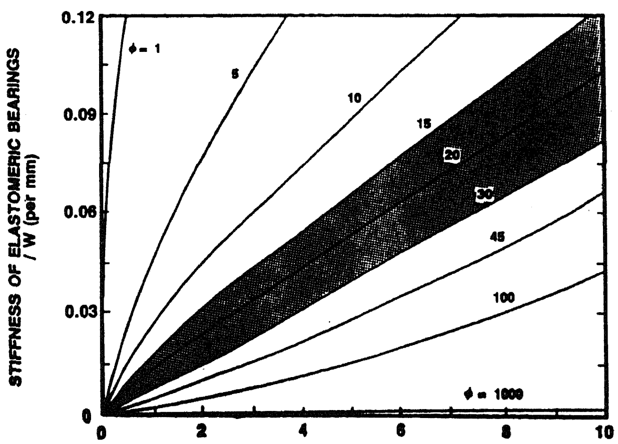

Fig. 2 Error in fundamental period due to rigid deck assumption

A typical highway bridge configuration is analyzed using the full model of beam elements and the translation massless springs for the bearings (Ref. 8). The same structure is analyzed again assuming rigid deck. In order to assess the effect of neglecting the stiffness of the deck on the dynamic response of the bridge, the error in the undamped fundamental period of the structure is plotted in Fig. 2 (Ref. 8). The error percentage is plotted against the total stiffness of the elastomeric bearing normalized to the weight of the superstructure. The error in the period is calculated as:

$$\text{Error}\,\% = 100\,(T_1 - T_2)/T_1 \tag{1}$$

where T_1 is the period of free vibration in the transverse direction assuming the deck to be flexible. T_2 is the period of the structure using the rigid deck assumption.

A positive error in Eq. 1 represents an underestimation of the period by the approach using rigid deck assumption. The deck flexibility ϕ is given by the expression

$$\phi = K_c / K_d \tag{2}$$

where K_c is the stiffness of the pier as a cantilever and K_d is the stiffness of an equivalent spring at pier location considering the deck as a simply supported beam in the lateral direction.

If the pier stiffness K_c in Eq. 2 is held constant, large ϕ represents flexible deck.

For a given ϕ value, Fig. 2 shows that the error in the fundamental period of free vibration increases for stiffer bearing design. This error is on the conservative side since smaller period corresponds to higher acceleration and higher design forces being in the long period zone of the response spectrum. The shaded area of the curve represents the practical range for ϕ for short and medium span highway bridges. For bearing stiffness design of 0.05 of the total bridge weight, the error in the period calculation corresponding to the rigid deck assumption does not exceed about 6%.

PIER STIFFNESS

Most simple design procedures do not take into consideration the variation in pier stiffness. The effect of this parameter is investigated by analyzing the response of bridges with pier stiffness varying between 8 and 550 MN/m. These values cover the range representing single column piers of 1.0 m diameter and heights of about 2.5 to 10 meters. The pier is assumed to be fixed at the foundation with a free top. The pier stiffness is calculated using the expression

$$K_c = 3\,EI / H^3 \tag{3}$$

where E is the modulus of elasticity of concrete, I is the cross section moment of inertia and H is the pier height.

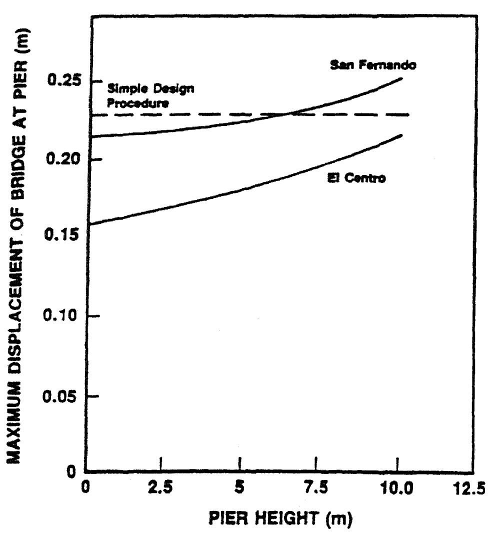

Fig. 3 Effect of pier stiffness on bridge displacement

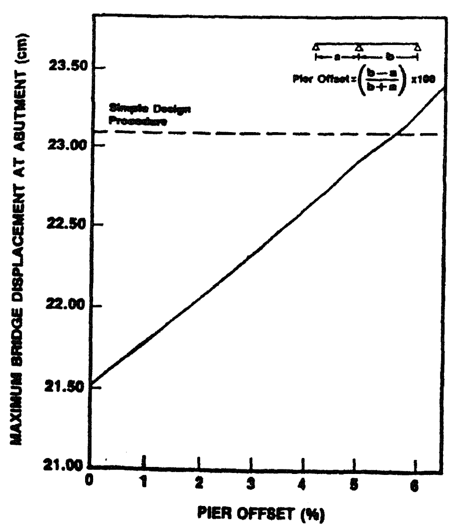

Fig. 4 Maximum displacement of two unequal span bridge

The maximum deck displacement variation with pier stiffness is shown in Fig. 3. For simplicity of illustration the pier stiffness used in the analysis is related to the pier height by using Eq. 3.

In general, the increase in the pier stiffness (i.e. shorter height) is accompanied by a decrease in the maximum displacement response. When the maximum displacement response obtained from the time history analysis of the flexible bridge model is compared with that obtained using simple design procedures (0.23 m), it is evident that after a certain pier stiffness the simple procedures become inadequate and begin to underestimate the response. For the San Fernando record, this occurs after pier height of about 5 m which corresponds to pier stiffness of 70 MN/m.

The effect of the pier stiffness variation on the forces transmitted to the pier and abutments is quite significant. The shear transmitted to a 10 m high pier is about 60% of the shear transmitted to a stiff pier. On the other hand, forces on abutments are increased by almost 30% over the stiff pier case. Similar effects can be obtained by the proper choice of the isolator stiffness. This observation is significant in the case of retrofitting existing bridges when additional forces on the pier are undesirable. Instead, the increased design forces due to the application of stricter codes are mostly taken by the abutments.

UNEQUAL SPAN BRIDGE

Two span highway bridges with near equal spans are fairly common structures. As the bridge configuraton departs from symmetry, the errors resulting from the application of simple design procedures will increase. For an unequal two span bridge, the maximum transverse displacement and shear force were found to occur at the abutment adjacent to the longer span. The maximum displacement and shear values increase with the increase of pier off-set. The pier off-set is defined as the difference between two spans divided by the sum of the two spans. The maximum bridge displacement variation with the pier off-set is shown in Fig. 4 (Ref. 6). For a symmetrical bridge, the simple design procedures overestimate the forces and displacements due to the rigid deck assumptions. As the pier off-set increases, the actual forces and displacements in the bridge increase while most simple design procedures are unable to take this effect into account. The simple design procedures cease to provide conservative results at a pier off-set of about 5.5 to 6%. For practical purposes, the limit of applicability of the simple design procedures is taken to be 5% off-set of the pier. For a pier off-set of less than 5%, the bridge spans are considered to be almost equal and the simple design procedures can be applied.

CONCLUSIONS

The applicability of simplified design procedures for base-isolated bridges was investigated. It is concluded that the simplified procedures are adequate as long as the bridge system can be approximately represented by a single degree of freedom dynamic model. This is particularly true for the case of symmetrical bridge configuration of single span or continuous over rigid piers. The bridge is also assumed to have a rigid deck which represents the majority of highway bridges.

The assumption of rigid deck can be evaluated for any specific case by using Fig. 2. However, for the majority of highway bridges, the error in underestimating the fundamental frequency was found to be less than 6%. As the pier stiffness is reduced, deviation from the response of simple methods is observed. A limit of pier stiffness of 70 MN/m is suggested for the applicability of simplified methods. This limit corresponds to a pier of 1.0 m diameter and 5 m in height. Bridges with two unequal spans represent another departure from the assumptions used in simplified analyses. A limit of the pier off-set of 5% is suggested for considering the spans to be almost equal without loss of accuracy from using simplified design procedures.

ACKNOWLEDGEMENTS

The author wishes to acknowledge the support of the Natural Sciences and Engineering Research Council of Canada. This work was carried out under an NSERC grant for Earthquake Engineering to McMaster University. The author also wishes to acknowledge the contributions of Mr. K.W. Wong and Mr. H.M. Ali to the calculations presented.

REFERENCES

1. Okubo, T. and Iwasaki, T., Summary of experimental and analytical seismic research recently performed on highway bridges. Proceedings of a workshop on Earthquake Resistance of Highway Bridges. Applied Technology Council, ATC 6-1, 567-601, Palo Alto, CA. 1979.

2. Gates, J.H., Factors considered in the development of the California seismic design criteria for bridges. Proceeedings of a workshop on Earthquake Resistance of Highway Bridges. Applied Technology Council Report, ATC 6-1, 142-162, Palo Alto, CA 1979.

3. Blakeley, R.W.G., Analysis and design of bridges incorporating mechanical energy dissipating devices for earthquake resistance. Proceedings of a workshop on Earthquake Resistance of Highway Bridges. Applied Technology Council, Palo Alto, CA, Report ATC 6-1, pp. 314-341, 1979.

4. Dynamic Isolation Systems Inc., Seismic base-isolation using lead-rubber bearings - design procedures for bridges, Berkeley, CA, 1984.

5. Ghobarah, A. and Ali, H.H., Design of base-isolated highway bridges. Proceedings of the Ninth World Conference on Earthquake Engineering. Tokyo-Kyoto, Japan, 1988.

6. Ghobarah, A., Seismic behaviour of highway bridges with base-isolation. The Canadian Journal of Civil Engineering, Vol. 15, No. 1, pp. 72-78, 1988.

7. ATC, Seismic design guidelines for highway bridges. Applied Technology Council, Palo Alto, CA, Report ATC-6, 1981.

8. Ghoborah, A. and Ali, H.M., Seismic performance of highway bridges. Engineering Structures Journal, (in press).