# NONLINEAR SEISMIC RESPONSE OF SHORT REINFORCED CONCRETE HIGHWAY BRIDGES

Mehdi Saiidi (I)
James D. Hart (II)
Presenting Author: M. Saiidi

## SUMMARY

Nonlinear modeling of short highway bridges subjected to static lateral loads and free-vibration excitation is discussed. The behavior of inelastic bridge components, namely, foundation elements, elastomeric bearing pads, and pier elements is reviewed in view of the available experimental data. The calculated and measured static and free-vibration response of a five-span bridge (the Rose Creek Bridge) which was tested as part of the project are presented, and difficulties in nonlinear modeling of the components are discussed.

## INTRODUCTION

One of the design philosophies common to many seismic codes is that a structure must not collapse, though may be severely damaged, in the event of the maximum credible earthquake. Allowance for considerable damage is necessary to keep the design economical. In ductile structures, damage under strong earthquakes is associated with significant nonlinear effects. Even moderate earthquakes can cause pronounced nonlinear action in some parts of the structure. Any analysis of structures for moderate to severe ground motions, therefore, should account for inelastic behavior of structural components and the structural unit.

This paper describes an analytical model which takes into account the inelastic behavior of all nonlinear components to predict the lateral response of short bridge systems. The model was developed in conjunction with experimental testing of the Rose Creek bridge by Douglas et al., near Winnemucca, Nevada (Ref. 1).

## THE NONLINEAR MODEL

A computer program was developed for the analysis of short to intermediate bridges with single-column piers. A schematic view of the bridge model is shown in Fig. 1. At the abutments, three springs were assumed, one translational and two rotational. The springs in translation and rotation with respect to the vertical axis were assumed to be nonlinear, while the other spring was treated as a linear system, the reason being its relatively small contribution to the response of the bridge. Other possible degrees of freedom were restrained because they were not expected to be significant. Deck and pier elements were idealized as line

(1) Associate Professor, University of Nevada, Reno, Nevada, USA

(II) Graduate Research Assistant, University of Nevada, Reno, Nevada, USA

members with nonlinearity allowed only at the base of pier elements. The pier foundation effect was represented by two nonlinear springs at the base of each pier. Only translation in the transverse direction and rotation with respect to an axis parallel to the longitudinal axis of the bridge were accounted for. Other possible degrees of freedom were fixed because, under lateral loads, their effect was believed to be negligible.

Structural Component

Idealization of different components of the bridge was based on the available experimental data and judgement.

Piers. In bridges with single-column piers the maximum moment usually occurs at the bottom of the pier.The nonlinear behavior, therefore, was assumed to be concentrated at the base of the pier over the height where the section is "weak". The shear resistance was assumed to be sufficiently large to prevent shear failure. To determine stiffness variations upon unloading and reloading after load reversal, a hysteresis model was developed which is a modified version of the Q-hyst model (Ref. 2), in that the primary curve consists of three linear segments as opposed to a bilinear primary curve used in the original model. The new version called "TQ-hyst", is described in Ref. 3.

Foundation. It is known that most soil types exhibit inelastic behavior even at very small strains. In lateral loading of soil samples, typically a curved relationship with gradual decrease in stiffness is obtained. Studies on the cyclic behavior of foundations have generally revealed that the hysteresis relationships include substantial deterioration of strength and stiffness with a trend similar to what is assumed in the TQ-hyst model.

Bearing Pads. Cyclic testings of neoprene bearing pads in shear have shown that a nonlinear effect is present even at small load amplitudes (Ref. 4). As loading continues a reduction in stiffness is observed until slippage occurs. Upon unloading and load reversal stiffness changes. Cyclic loadings of pads for vertical loads, on the other hand, have shown some stiffening effects as loading progresses. In bridges, due to the relatively large vertical gravity forces acting on the pads, variations in the vertical force due to the rotation are not expected to lead to any significant nonlinear effects. As a result, the vertical behavior of the pads was assumed to be linear. For the lateral behavior of the pads, the Ramberg-Osgood model (Ref. 5) was adopted to represent the cyclic response (Fig. 3). This was decided after a qualitative study of the available experimental data. The model was used in both translation, and rotation about the longitudinal axis of the bridge.

Deck Element. Design of bridge decks is usually controlled by gravity loads. The relatively large width of bridge deck results in large stiffness and large cracking moment and shear resistance unlikely to be exceeded as a consequence of lateral loading. The deck element, therefore, was assumed to remain elastic.

Total Bridge Structure. A computer model was developed for static and free-vibration analysis of highway bridges subjected to lateral loads. In the static analysis part of the model, horizontal forces can be applied at pier-deck intersections, abutments, and pier bases. For each load increment the status of the nonlinear elements is checked and their stiffnesses are updated as necessary. To allow for close monitoring of force-deformation variations, the loads are applied in small increments. For each load increment, lateral displacements, rotation, and all the internal forces are computed.

The dynamic analysis portion of the computer model determines the free-vibration response of the bridge with initial displacements being those caused by the static forces. The initial stiffness and the status or hysteresis curves for different components used at the start of the free-vibration analysis are those determined at the end of the last static load increment. The differential equation of motion is formulated in an incremental form and integrated using small time intervals. The acceleration and lateral displacement are calculated and stored for plotting the response histories.

## THE ROSE CREEK BRIDGE

The Rose Creek bridge is a five-span reinforced concrete multi-cell box girder bridge with a total length of 120m, located on highway I-80 near Winnemucca, Nevada (Fig. 4). The substructure consists of four single piers (Fig. 5) and the abutments, all of which are supported by pile foundations. The deck is continuous with no intermediate expansion joints, and is supported by five elastomeric bearing pads at each abutment. The bridge was designed based on the 1965 AASHTO code.

The reinforcement distribution in the piers is shown in Fig. 6. The connection to the footing is a hinged connection in the longitudinal direction of the bridge, but is rigid in the transverse direction. Because no yielding of the reinforcement in the deck was expected, the deck steel did not enter the analysis and is not shown.

## EXPERIMENTAL STUDIES

The Rose Creek bridge was subjected to static and dynamic loads (Ref. 1). The static loads were applied in the transverse direction of the bridge at the intersection of the piers and the deck by four hydraulic rams acting at an angle of $45^{\circ}$. The rams were loaded manually at low rate. Temporary reaction foundations were built to support the rams. The bridge was loaded to several amplitudes and the ram loads were simultaneously released to allow for free-vibration testing of the bridge.

## ANALYTICAL AND EXPERIMENTAL RESULTS

The Rose Creek Bridge was analyzed using the analytical model described in the previous sections. The basic properties of pier and deck elements were determined based on the geometry and reinforcement distribution, using the average of measured strength for 28-day concrete samples and the specified yield strength of steel. To determine the basic backbone curve for the abutment springs the available guidelines prepared by the manufacturer were initially used, but the results appeared to lead to unreasonably "soft" abutments. Estimates of initial stiffness of the pads for ambient level of vibration were available in a report by Gates and Smith (Ref. 6). These values were used as the slope of tangent to the back-bone curves.

Attempts were made to determine the primary curves for pier foundation springs based on the soil profile and properties of the piles and the pile caps. Information was needed on the lateral and rotational behavior of pile groups. No procedure for finding these properties could be found in the available literature. The only information was the lateral stiffness of the pile groups (Ref. 7). These stiffnesses, modified to exclude cyclic effects, were used for the translational springs. The rotational springs were assumed to be fixed due to the fact that no procedure for calculating pile group rotations could be found.

The lateral displacement of the bridge is shown in Fig. 7. It can be seen that the experimental data exhibited a slight degree of nonlinearity. No visible nonlinear behavior could be observed in the analytical result. This is, in part, due to the fact that for small loads, the only source of nonlinearity in the model is the elastomeric bearing system, which does not affect the response at the center to any great extent. The response at the bearing pads, on the other hand, showed a slight but visible nonlinear effect (Fig. 8). It can be seen in Figs. 7 through 9 that the analytical results were smaller than the measured values. This is attributed to the fact that the pier foundations were assumed to be fixed against rotation.

The measured and calculated free-vibration acceleration histories for three locations of the deck are shown in Fig. 10. Reasonably good correlation was observed during the first four seconds in the response for pier 1 and pier 2. The modeling of the nonlinear behavior cannot be evaluated in detail based on these response histories because of the fact that only a limited degree of nonlinearity was present in both the calculated and measured response and that the basic properties of the elastomeric bearing pads and foundation springs were approximate. Nevertheless, the relatively good agreement observed in the large-amplitude part of the responses can be an indication that the assumptions and idealizations made in developing the analytical model were somewhat realistic.

## CONCLUSIONS

The study presented in this paper showed that the nonlinear effects should be taken into account if a reasonable estimate of seismic response is to be obtained. Unlike building structures, where the response is usually dominated by the superstructure, bridge response is significantly affected by the foundation and abutments (Ref. 1). It was found that for the Rose Creek Bridge the lateral displacement was underestimated at the deck center by more than 30 percent by restraining the pier foundations against rotation. The available literature dealing with the seismic aspects of geotechnical engineering do not provide adequate guidelines to determine basic nonlinear force-deformation properties of pile groups for translation and rotation, although a considerable amount of data is available from testing of single piles. Another parameter for which information is inadequate is the nonlinear force-deformation characteristics of elasto-meric bearing pads, and their cyclic behavior. This information is needed for a realistic modeling of the nonlinear response of bridges. Nonetheless, based on the correlation between the analytical and experimental data, it can be concluded that the proposed model is a reasonably good start for nonlinear modeling of dynamic response in bridges.

ACKNOWLEDGMENTS

The study leading to this paper was part of a continuing project supported by the U.S. National Science Foundation under Grant CEE-8108124. The support is gratefully acknowleged. Appreciations are due to Dr. Bruce Douglas, the project director, for his many helpful comments and advice. Dr. Gary Norris is specially thanked for providing the stiffness properties of the pile foundations.

REFERENCES

1. B. DOUGLAS, M. SAIIDI, J. RICHARDSON, and J.D. HART 1983 Proceedings of the Fifteenth Joint Meeting of U.S./Japan Panel on Wind and Seismic Effects, UJNR. Results from High-Amplitude Dynamic Tests and Iimplications for Seismic Design.

2. M. SAIIDI 1982 ASCE, Journal of the Structural Division, Vol. 108, No. ST5, 1077-1087. Hysteresis models for reinforced concrete.

3. M. SAIIDI, J.D. HART and B. DOUGLAS 1983 Proceedings of the Second International Conference on Recent Advances in Structural Dynamics, The University of Southampton, England. Inelastic Analysis of Short Highway Bridges Subjected to Strong Ground Motions.

4. W.B. NACHTRAB and R.L. DAVIDSON 1965 National Academy of Sciences-National Research Council, Highway Research Record No. 76, 83-101. Behavior of elastomeric bearing pads under simultaneous compression and shear loads.

5. W. RAMBERG and W.T. OSGOOD 1943 NACA Technical Note 902. Description of stress-strain curves by three parameters.

6. J.H. GATES and M.J. SMITH 1982 California Department of Transportation, Report No. FHWA/CA/SD-82/07. Verification of dynamic modeling methods by prototype excitation.

7. B. DOUGLAS and G.M. NORRIS 1983 ASCE Journal of Technical Topics in Civil Engineering. Bridge dynamic tests/implications for seismic design.

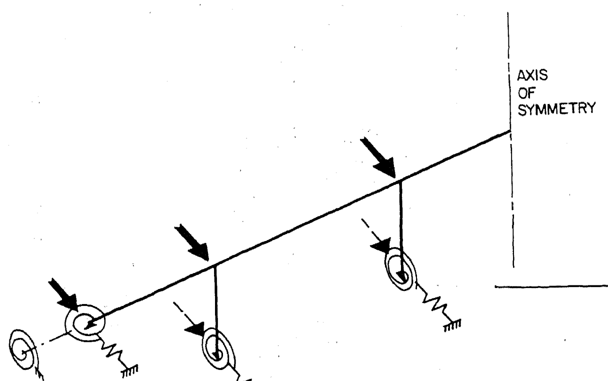


Fig. 1 Idealized Bridge Structure

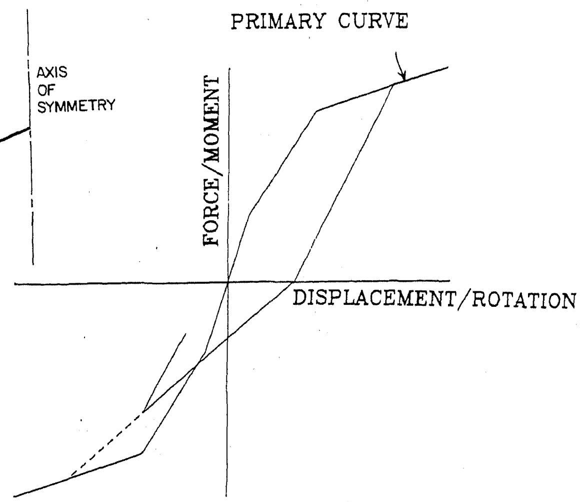


Fig. 2 The Trilinear Hysteresis Model (TQ-Hyst)

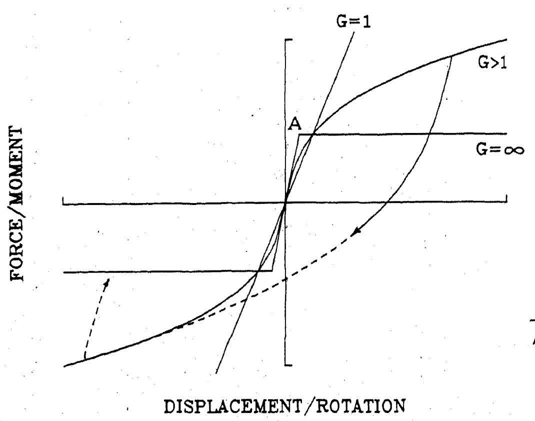


Fig. 3 The Ramberg-Osgood Model

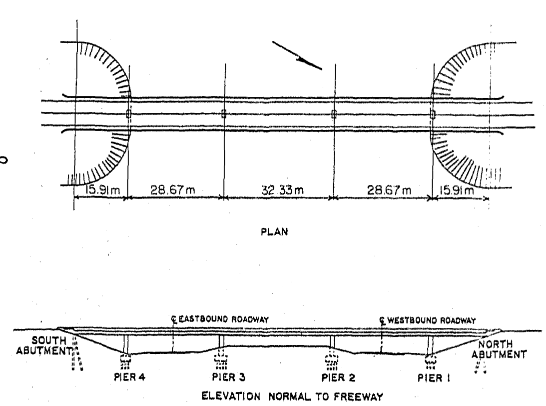


Fig. 4 The Rose Creek Bridge

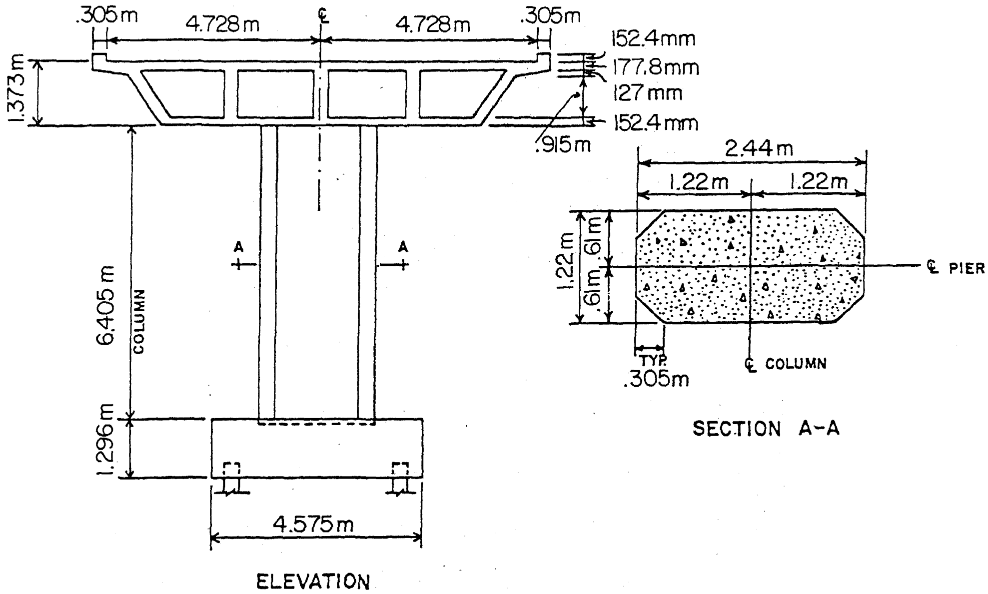


Fig. 5 Pier and Deck Dimensions

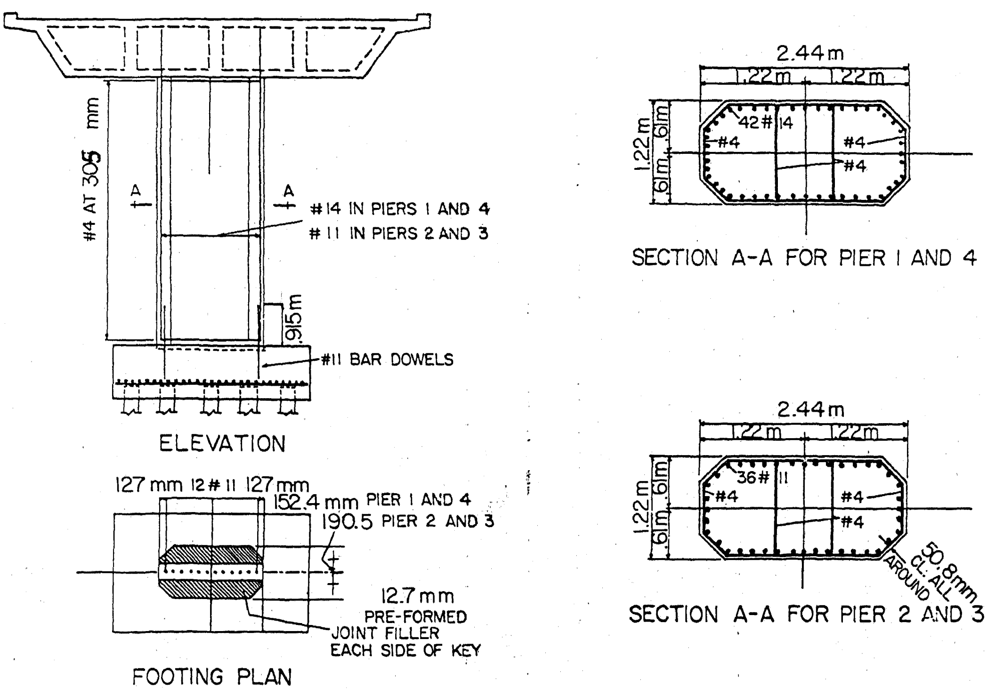


Fig. 6 Detail of Pier Reinforcement

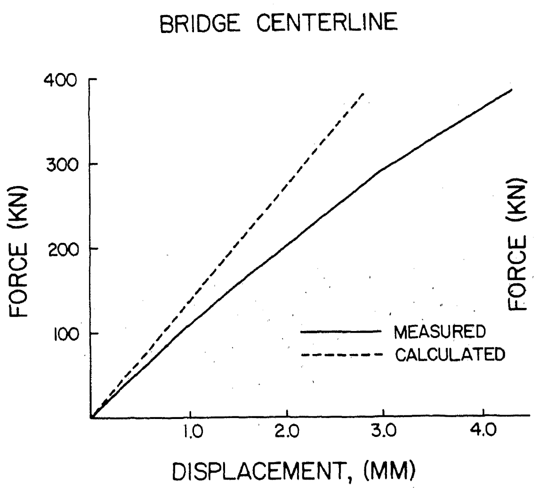


Fig. 7 Static Response at the Deck Center

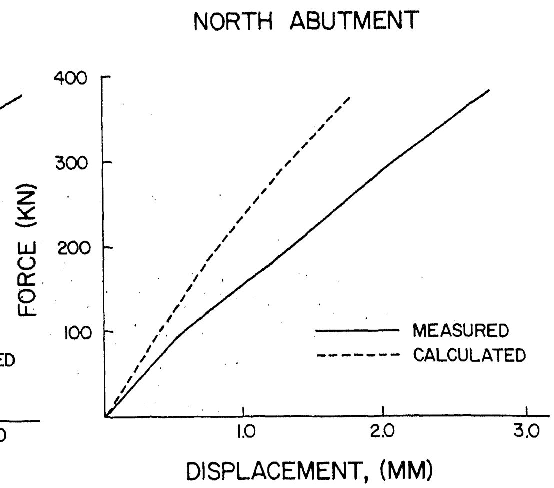


Fig. 8 Static Response at the North Abutment

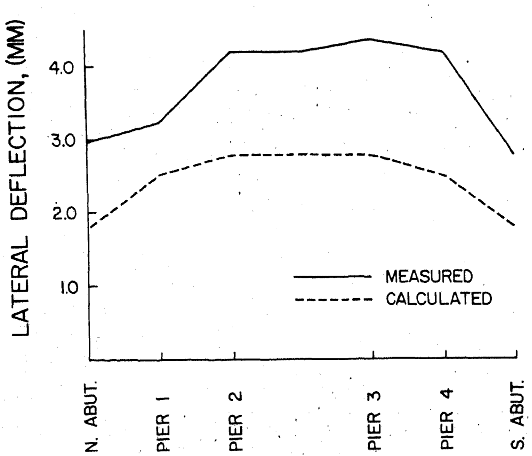


Fig. 9 Plan View of Deflection for the Maximum Load

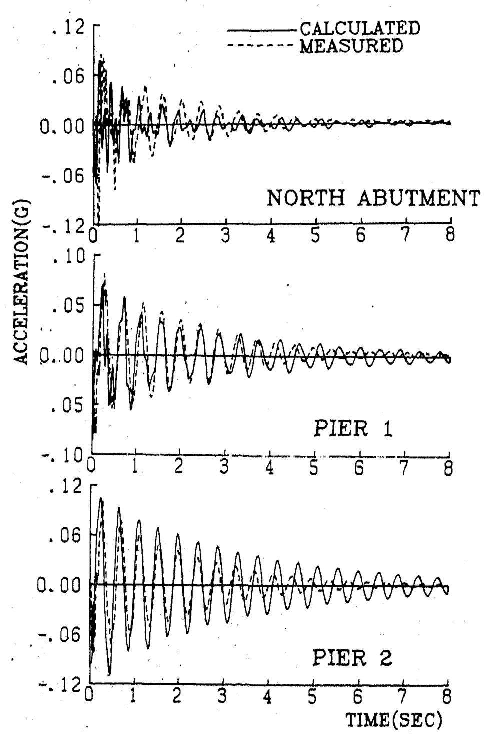


Fig. 10 Acceleration Histories