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Effect of base isolation on the seismic response of multi-column bridges

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Abstract. A nonlinear model for time-step analysis of bridges subjected to two orthogonal horizontal components of earthquake motions was developed. The focus of the study was on elastomeric isolators with or without lead cores. The hysteretic behavior of the isolators, the columns, abutments, and shear keys was taken into account. The nonlinear analysis showed that, contrary to linear theory prediction, the use of isolators does not necessarily increase the displacement of the superstructure. Furthermore, it was shown that properly designed isolators can reduce the ductility demand in RC bridge columns substantially.

Key words: abutments; base isolation; bridges; columns; displacement; earthquakes; elastomeric; force; seismic.

1. Introduction

Seismic isolation has been considered to be a viable mean of reducing earthquake forces and ductility demands on bridge substructures (Blakely 1982, Buckle and Mayes 1989) and is beginning to receive special attention as a seismic retrofit alternative (Sultan 1998). The general premise in seismic isolation has been that, by lengthening the vibration period of the structure, the dynamic forces will decrease but displacement will increase due to the period shift. While this statement is true for linear systems, because the response of an isolated structure during strong earthquakes may be highly nonlinear, an increase in displacements does not always occur. A study by Saiidi *et al.* (1992) showed that, by proper design of isolators, displacements can be kept relatively small while reducing dynamic forces. That study was limited to two-span bridges supported on a single column pier. The response of two-span base-isolated bridges supported on single-column piers was also studied by Foutch and Chen (1994). This paper presents results from a study of a six-span bridge supported on multi-column bents. The structural properties are based on a six-span base-isolated bridge located on Interstate 80 west of Reno, Nevada.

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2. Analytical modeling

2.1. General

A computer program was developed for nonlinear seismic analysis of isolated bridges subjected to two simultaneous horizontal components of earthquake motions applied in the orthogonal directions. The program carries out a time-step analysis of the structure with the tangent stiffness used over each time step. Details of the program are presented in (Griffin *et al.* 1995). Some of the main features of the program are:

1) Foundation flexibility is incorporated by a series of uncoupled linear springs;
2) The superstructure is represented by linear beam elements;
3) Isolators are modeled by uncoupled bilinear hysteretic elements;
4) The nonlinear response of the bridge columns is modeled by the four-spring biaxial hysteresis element (Jiang and Saiidi 1990);
5) Abutment and shear keys are modeled by a bilinear hysteretic element which is activated only when the abutment gap or the shear key gap is closed;
6) Newmark's Beta method is used to solve the differential equation of motion. Only straight, non-skewed bridges are allowed in the analytical model.

2.2. Modeling of column footings

Of the six possible degrees-of-freedom (DOF) at column bases, only the vertical displacement and rotation about the vertical axis are ignored because they are believed to have negligible effect on the response. A linear soil spring is assigned to each of the remaining four DOF's.

2.3. Modeling of isolators

Only elastomeric type of isolators, with or without a lead core, was incorporated in the analytical model. The nonlinear behavior of the isolators in each direction was represented by an uncoupled translational spring. Recent research has indicated that the coupling effect can be significant in some cases (Fenves *et al.* 1998). However, experimental data on the biaxial dynamic response of isolators are limited, and analytical models that are calibrated with extensive test data are yet to be developed. Nonetheless, an analytical study was conducted to examine the effect of isolator coupling on the response of a single DOF structure before proceeding with the investigation on the six-span structure.

The Bouc-Wen model which was introduced by Wen (1980) using a unidirectional hysteresis model developed by Bouc (1971) was adopted in this study.

The properties of the SDOF model were based on the isolators and the bridge mass of structure G-772 located on Interstate 80 west of Reno, Nevada. The input earthquake was the NS component of El Centro 1940. The SDOF model was analyzed with coupled and uncoupled isolator elements. Each model was analyzed for a unidirectional earthquake and a bidirectional earthquake applied horizontally. In the case of bidirectional motion, the El Centro-NS record was applied in both directions. Fig. 1 shows the response for the unidirectional earthquake. It can be seen that coupling did not affect the response significantly. The amplitudes and frequency content for both case were very close. The difference was more pronounced when bidirectional motion was applied (Fig. 2). However, the large amplitude waves for both the coupled and uncoupled

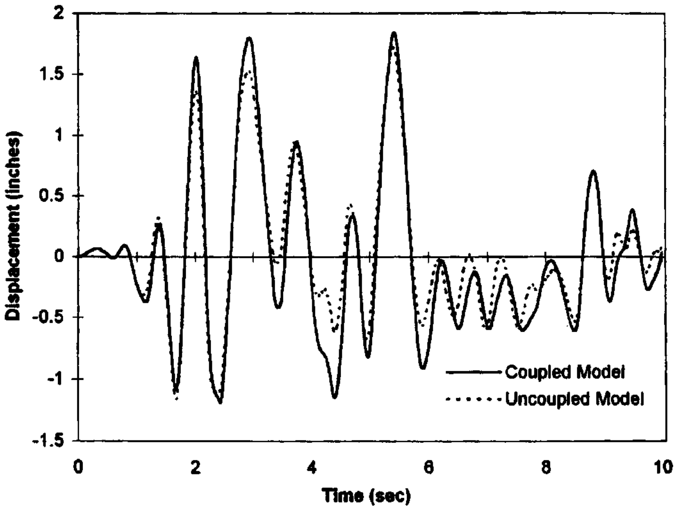

Fig. 1 Effect of isolator coupling on the unidirectional response

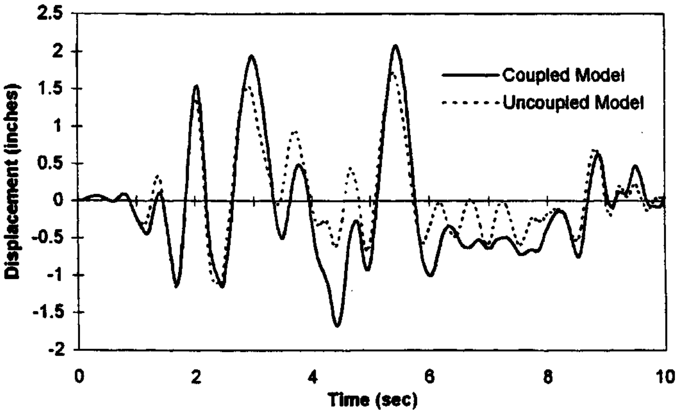

Fig. 2 Effect of isolator coupling on the bidirectional response

models were similar with respect to the peaks and frequency. Considering that the uncoupled hysteresis model is considerably simpler than the coupled element and the fact the bridge response is also affected by the response other bridge components, in addition to a lack of extensive experimental data to verify the coupled model, it was decided to adopt the uncoupled model in the analysis of the six-span bridge.

2.4. Modeling of piers

Only bridges with reinforced concrete piers were considered in the study. The biaxial bending behavior of the columns was modeled by the 4-spring element (4-SE) that had been developed by Yang and Saiidi (1990). The plastic hinge in this model is represented by four nonlinear springs that, when in tension, account for steel tensile forces and when in compression they account for the combined forces in concrete and steel. The hysteretic behavior of the springs is represented by a simple model that is a modified version of the Q-Hyst model (Saiidi 1982). The 4-SE has been

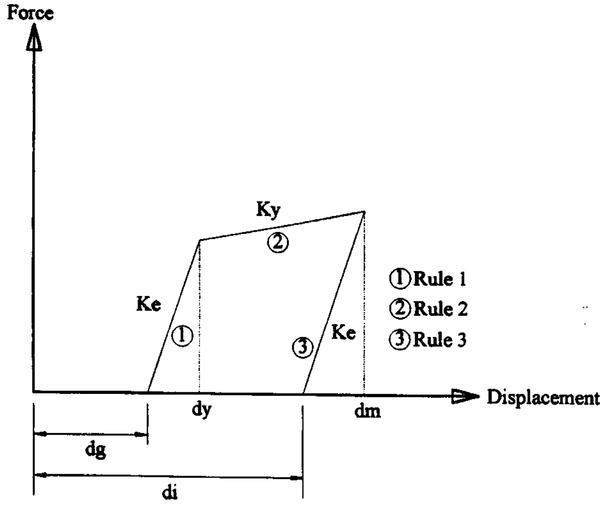

Fig. 3 Primary curve for abutment and shear key springs

evaluated against test data for reinforced concrete columns subjected to biaxial loading and has shown that it represents the response well (Yang and Saiidi 1990).

2.5. Modeling of abutments and shear keys

Unidirectional nonlinear springs were used to model the nonlinear response of the abutments and shear keys. The basic load-displacement relationship for the springs is shown in Fig. 3. In this figure, d_g indicates the gap between the superstructure and the abutment or shear key. Once this gap is closed the spring is activated. Note that the spring acts in compression only. Upon yielding and after unloading, the permanent displacement is added to the initial gap and a new gap is determined. For the spring to be reactivated, the new gap has to close first.

3. Example bridge

A six-span bridge located in Verdi at approximately 10 miles west of Reno, Nevada was used as the basis of the model analyzed in this study. Fig. 4 presents a sketch of the computer model of the bridge. The section properties and the dimensions of the bridge model were close to those of the Verdi Bridge. This bridge was recently retrofitted with elastomeric seismic isolators with lead cores. The superstructure is a continuous multi-girder element supported on isolators at the abutments and the piers. All the columns were assumed to have 3.5' by 3.5' square sections, although the actual bridge has rectangular columns with plan view dimension of 3.5' by 4' in two of the bents. A continuous superstructure was assumed in the analysis. The superstructure is of steel and concrete composite type. The footings were assumed to provide full fixity at the base of the columns to maximize the demand on the bridge elements. The damping ratio was assumed to be five percent. The concrete compressive strength was taken equal to 4 ksi and the steel yield stress was 60 ksi.

Two models of the structure were analyzed, an as-built bridge and a retrofitted bridge with

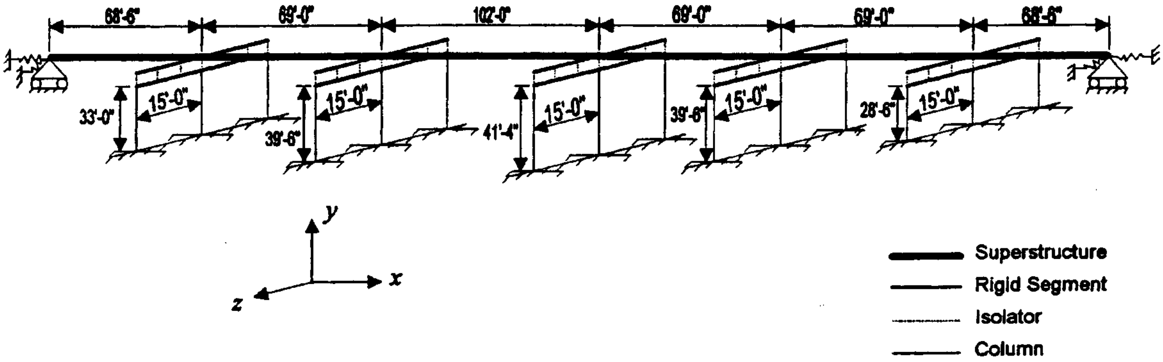

Fig. 4 The bridge model

isolators. No other elements of the bridge were assumed to be retrofitted. The as-built structure was assumed to be pinned at Bents 2 and 3, and on rollers elsewhere. In the retrofitted bridge, isolators were assumed at all superstructure bearings. The number, type and the size of the isolators in the bridge model were the same as those in the actual retrofitted bridge. Seven isolators are used at each of the abutments and 14 at each bent. At Bents 2 and 3 the isolators are 12 in. by 12 in. by 7.38 in., while all the other isolators are 11 in. by 11 in. by 7.38 in. The structural properties of the isolators were obtained from the design documents provided by the supplier of the isolators.

The as-built and retrofitted bridge models were analyzed for the north-south component of the 1941 El Centro earthquake record applied simultaneously in both the transverse and the longitudinal directions. The peak ground acceleration was 0.35 g. More information about the bridge is presented in (Griffin *et al.* 1995).

4. Results of the analysis

Fig. 5 shows the displacement response history at the top of the center pier for both the as-built and the isolated structures. Base isolation changed the dynamic characteristics of the bridge considerably as reflected in major changes in the response. It can be noted that seismic isolation reduced the peak displacement in the transverse direction by more than 40. The reduction in the longitudinal response (not shown) was in the same order. The lower maximum displacements are contrary to the expectation that base isolation always increases the displacements, and indicates that the variable stiffness of nonlinear system during an earthquake does not allow for estimation of the response from linear response spectra. The primary effect of the lower displacements is the reduction in (a) the abutment forces, and (b) the column drifts and ductility demands. This, of course, means less damage in the bridge substructure and the abutments.

The most critical bents in the as-built bridge are Bents 2 and 3 because their connection to the superstructure is pinned, while other bents provided a roller support for the superstructure. Of these bents, columns of Bent 2 are more critical because they are shorter. The longitudinal and transverse force-displacement responses of the central column in Bent 2 for the as-built bridge are shown in Figs. 6 and 7, respectively. It can be observed that significant yielding occurred in the column. The estimated yield displacement for the longitudinal and transverse directions for this column was 1.05 in. and 0.64 in., respectively. The yield displacements were found using cracked

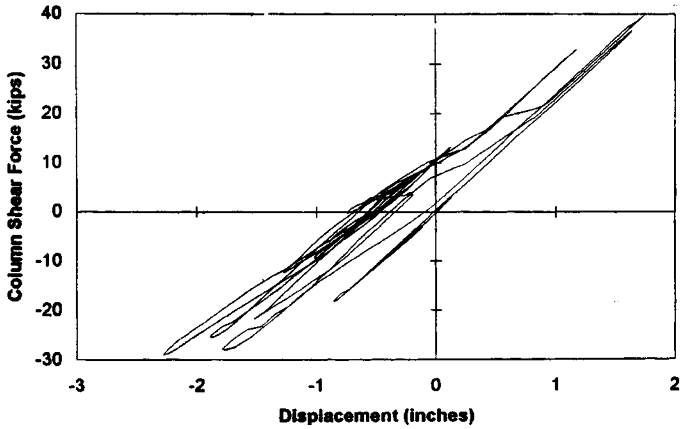

Fig. 8 Longitudinal force-displacement response of the middle column in bent 2 of the retrofitted bridge

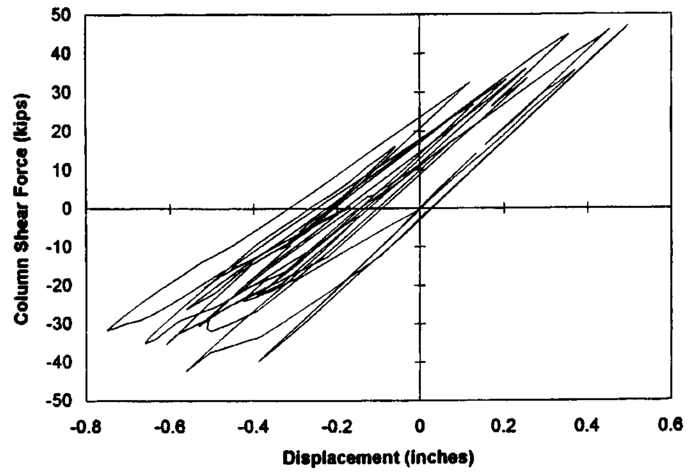

Fig. 9 Transverse force-displacement response of the middle column in bent 2 of the retrofitted bridge

section properties and taking into account the flexural and bond slip deformations. The peak displacement ductility was hence 6.8 in the longitudinal direction and 4.3 in the transverse direction. Due to a lack of adequate confinement steel, the as-built columns in the actual bridge would not be able to develop these ductilities and would fail. The response of the same column in the base-isolated bridge is shown in Figs. 8 and 9. Note that the displacements were considerably lower and that the extent of yielding was limited. The maximum ductility demands in the longitudinal and transverse directions were 2.2 and 1.2, respectively, which indicate a level of damage that a pre-1971 bridge column design could sustain.

The primary source of energy dissipation in the isolated bridge was the hysteretic damping provided by the isolators. Figs. 10 and 11 show the hysteresis curves for the longitudinal and transverse directions of the isolators at top of Bent 2. It is evident that considerable yielding occurred in the isolators and that the isolators dissipated a great deal of energy. Other isolators in the bridge also indicated large amount of energy dissipation.

The aforementioned results indicate that seismic isolation reduced the extent of column yielding

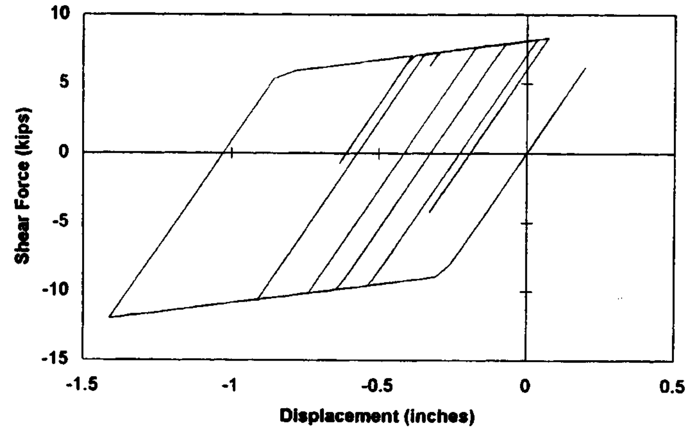

Fig. 10 Hysteretic response of isolators on bent 2 in the longitudinal direction

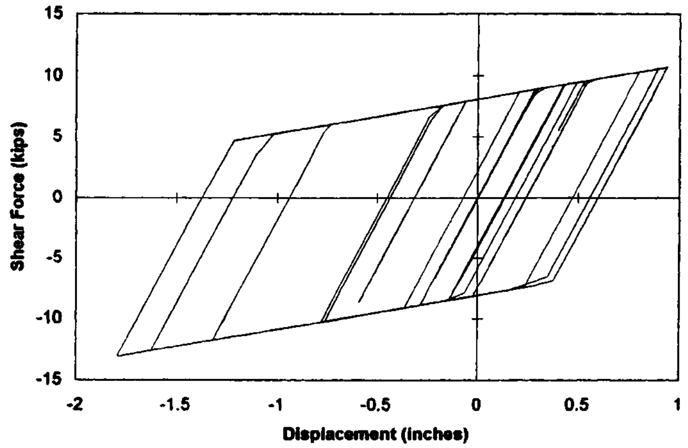

Fig. 11 Hysteretic response of isolators on bent 2 in the transverse direction

substantially. The results also showed that energy dissipation capability of the isolators was extensively utilized during the earthquake. More detailed description of the analysis and the response at different locations of the bridge may be found in (Griffin *et al.* 1995).

5. Conclusions

The effectiveness of seismic isolators in reducing the force and displacements on the superstructure of a six-span bridge was demonstrated. It was also shown that, contrary to the linear theory prediction, the use of isolators does not necessarily increase the displacement of the superstructure. In the example bridge used in this study, it was found that the use of isolators reduced the peak longitudinal and transverse displacements by 60 and 40 percent, respectively. Seismic isolation also reduced the maximum column ductility demand from 6.8 to 2.2, a ductility level that existing columns with inadequate confinement steel are capable of providing. The results of the study also indicated that a reasonable estimate of the isolator response can be obtained even when the coupling effect in modeling of the isolators is ignored.

Acknowledgements

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