SEISMIC RETROFIT OF BRIDGES UTILIZING DUCTILE BASE ISOLATION CONCEPTS

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ABSTRACT

Recent developments in New Zealand on the use of base isolation and energy dissipating mechanisms for the design of new bridges offer exciting possibilities for the retrofit of existing bridges. The concepts provide retrofit solutions for columns and piers with inadequate strength and for superstructure girders with inadequate support lengths. The base isolation and energy dissipating lead/rubber bearings developed in New Zealand significantly decrease the superstructure displacements during seismic excitation and they provide a force-limiting mechanism for the supporting substructure. The paper presents the application of the lead/rubber bearings on two typical existing bridges. Comparisons are provided for the forces resulting in both the as-built structure and the structure incorporating the bearings when the bridge is subjected to CalTrans design ground motions for a site close to a major fault.

1 INTRODUCTION

The collapse of a highway bridge during an earthquake will in many cases sever vital transportation routes at a time when they are most needed to provide emergency services to or facilitate evacuation from a stricken area. The loss of the bridge as a transportation link may potentially result in a greater loss of life than the immediate effects of collapse. The San Fernando Earthquake of 1971 taught engineers a great deal about the seismic resistance of bridge structures. This earthquake also demonstrated the potential inadequacy of past design procedures in providing seismically resistant bridges. Since most existing bridges in service today were designed using pre-1971 design procedures, it follows that many of the nation's highway bridges in seismically active areas may have insufficient strength to resist seismic loading.

To address this problem, the U.S. Federal Highway Administration (FHWA) has funded two major projects that are being performed by Applied Technology Council (ATC). The first, which has been completed, consisted of the development of ATC-6 Seismic Design Guidelines [1] for new highway bridges. The second, which will be completed in mid-1983, consists of the development of seismic retrofit guidelines for existing bridges. An overview of the retrofit guidelines is presented in Section 2.

Recent developments in New Zealand on the use of base isolation and energy dissipating mechanisms for the design of new bridges offer exciting possibilities for the retrofit of existing bridges. The concepts provide, as discussed in Sections 3 and 4, solutions for columns and piers with inadequate strength and for superstructure girders with inadequate support lengths. The application of the lead/rubber bearings to the retrofit of two bridges is presented in Sections 5 to 9. The two bridges have identical dimensions, the only difference being that one has individual cantilevered columns and the other has a more typical column bent.

2 ATC-6-2 RETROFIT GUIDELINES

An ATC Project Engineering Panel is being used to develop the seismic retrofit guidelines for highway bridges. The retrofit guidelines will recommend procedures for evaluating and upgrading the seismic resistance of existing highway bridges. Methods of evaluation will assist engineers in identifying and assessing bridges which could be hazardous to life safety during earthquakes. Methods of retrofitting various vulnerable bridge components will also be presented in the guidelines. Since seismic retrofitting is a relatively new concept, only a few retrofitting schemes have been tried in practice. At present, seismic retrofitting is an art requiring considerable engineering judgment. Although the guidelines will present accepted retrofitting techniques, they are not intended to restrict innovative designs.

The retrofit process involves identification of the bridges which pose the greatest threat to life safety due to earthquakes; a procedure for the detailed evaluation of individual bridges so identified; determination of the need for retrofitting; identification of appropriate retrofit measures; an economic assessment of the benefits of retrofitting; and a decision to retrofit or not to retrofit. The guidelines are intended for use throughout this retrofitting process.

The detailed seismic evaluation of a bridge is recommended to be performed in two phases. The first phase is a quantitative evaluation of individual bridge components using the results from the analysis procedures developed for the ATC-6 Guidelines [1]. The resulting elastic force and displacement results, which are referred to as demands, will be compared with the ultimate capacities of each of the components that resist these forces and displacements. The ability of columns to resist post elastic deformations is also to be considered. A capacity/demand ratio will be calculated for each potential mode of failure in the critical components. This ratio is designed to represent the portion of the design earthquake that each of the components is capable of resisting.

The second phase of evaluation is an assessment of the consequences of failure in each of the components with insufficient capacity to resist the design earthquake. Consideration will be given to retrofitting substandard components if their failure results in a bridge collapse. In the case of essential bridges, its loss of function may also warrant consideration of retrofitting. There are four areas where local failure has a high potential of occurring and where component capacity/demand ratios will be calculated. These are: bearings and expansion joints; columns, piers and footings; abutments; and liquefaction potential. Aspects of the evaluation process relating to each of these areas will be presented in the guidelines.

3 BASE ISOLATION CONCEPTS

The isolation of structures from the damaging effects of earthquakes is not a new idea. An historical review [2] suggests that the first patent for a base isolation scheme was taken out as early as 1909, and since that time, several similar proposals have been made. Nevertheless, until very recently very few structures have been built which use these principles. However, new impetus was given to the concept with the successful development of mechanical energy dissipators by the New Zealand Department of Scientific and Industrial Research [3]. When used in combination with a flexible isolation device, an energy dissipator can control the response of the structure by limiting displacements and forces and thereby improving seismic performance. As a consequence, several structures have now been designed and built in New Zealand which incorporate base isolation and energy dissipators [4, 5]. Most are bridges, but two buildings and a free-standing chimney have also been protected in this way. Many of the features of base isolation can be used to improve the seismic performance in existing structures. The inclusion of base isolators and energy dissipators when upgrading a structure can significantly enhance its seismic performance.

3.1 Principles of Base Isolation

There are two basic elements in New Zealand base isolation systems. These are:

1. A flexible mounting so that the periods of vibration of the total system are lengthened (shifted) sufficiently to reduce acceleration response; and

2. A damper or energy dissipator so that the relative deflections across the flexible element can be controlled.

Bridge structures have for a number of years been supported on elastomeric bearings and as a consequence have already been designed with the flexible mount. Increasing the thickness of the bearing is a natural step to ensure adequate flexibility and period shift. However, large superstructure displacements will be generated unless steps are taken to control these relative motions. Energy dissipators have been proposed for this purpose since they provide both additional elastic stiffness and hysteretic damping.

4 CHARACTERISTICS OF LEAD/RUBBER BEARINGS

A lead/rubber bearing [6] comprises alternate layers of rubber vulcanized or cemented to thin steel shims, typically with a lead plug placed in the center (Fig. 2). The rubber in the bearing carries the weight of the structure and the lead plug provides energy dissipation by plastic deformation. Outer steel shims with dowel holes are provided for transfer of lateral force from the structure above to the bearing. Vertical stiffness of the bearing (K_v) and thus the vertical load capacity is inversely proportional to the thickness of individual rubber layers. Hence, bearings have multiple thin rubber layers rather than a single rubber layer. The shear stiffness of the bearing is similarly inversely proportional to the overall thickness of rubber and increases with increased bonded surface area.

Typical behavior of the lead/rubber bearing under cyclic lateral loading illustrates its effective use as a base isolating device (Fig. 3). The bearing is designed to resist low levels of shear, such as wind loads, elastically (K_u), as determined by the yield parameter Q_d. The force level, Q_d, is a function of the diameter of the lead plug(s). The post yield stiffness K_d is kept at a minimum to ensure good energy dissipation and low overall structure stiffness during more severe seismic loading. The parameters that are therefore most important to the designer are Q_d, K_d and also K_v (the vertical stiffness).

5 DESCRIPTION OF BRIDGES

The bridge used for the evaluation is a reasonably large existing highway bridge. The overall layout of the bridge is shown in Fig. 1. The average bridge length is 615 feet with a width of 117 feet. The bridge is on a skew of approximately 60 degrees with respect to the highway traffic lanes. The bridge has a concrete deck slab supported on a steel superstructure with concrete pier columns 3'-0" diameter and approximately 25 feet high. There are a total of 27 columns, generally arranged in a bent formation of 4 columns each. At each end of the bridge due to the severe skew there are only 2 or 3 columns in the bent formation. The foundations are formed of individual pads beneath each column. The steel superstructure consists of a set of longitudinal girders connected to major transverse girders that bear across each column line. The longitudinal girders are fixed at one end and are free to move longitudinally at the other end. The concrete bridge deck has expansion joints over each pier and at each abutment.

For the purpose of the retrofit evaluation two configurations of this bridge were assessed. The first was such that each column was pin-connected to the superstructure transverse girder so that the columns acted as cantilevers under lateral loading. The second was such that each set of columns was connected by a pier cap so the columns act as part of a moment resisting frame.

6 RETROFIT EVALUATION CRITERIA

The two major problems that exist with the existing bridge are the strength and ductility capacity of the columns and the support lengths provided for the longitudinal girders. The reinforcement details of the existing reinforced concrete columns are such that they are unable to withstand any significant inelastic deformation. The ATC-6-2 guidelines recommend that for poorly detailed columns the maximum elastic moment capacity that can be resisted is 1.6 times the nominal moment capacity calculated using a Φ factor of 1. For the reinforcement details of the existing columns, this corresponds to a column shear force of 86 kips for the pin-connected columns, and twice this value for the columns with a pier cap.

For the purpose of retrofit evaluation, the existing bearings at the column and abutment support locations were replaced by lead/rubber bearings to determine if both the column forces and superstructure displacements could be reduced to acceptable levels so the bridge would respond essentially elastically. The following section describes the analyses that were performed to evaluate the performance of the two bridge configurations that incorporate the lead/rubber bearings.

7 ANALYTICAL MODEL AND SEISMIC INPUT

The bridge was modelled using the general purpose, 3-dimensional non-linear finite element program, ANSR-II. The basic characteristics of the analytical models are as follows: Each bent of (typically) four columns is modelled by a single element which is constrained to be elastic. This is the basic philosophy adopted for this retrofit evaluation because of the inadequacy of the columns to respond in a significantly ductile manner. The columns are assumed to have full fixity at their bases. The deck is modelled by a flexural beam, vertically pinned at one end between bents. This allows shear transfer between bents, and permits explicit modelling of the transverse breathing of the deck. For all analyses performed, the superstructure was free to move both longitudinally and transversely within the constraints of the boundary conditions for each model. The connections at the columns and abutments were dependent on the structure analyzed. For the as-built structure with cantilever columns, the columns were connected to the superstructure via pinned connections in both directions. This ensured cantilever action at the piers. The connection detail at the abutments allowed longitudinal movement of the superstructure, but transverse movement was not permitted. For the as-built structure with pier cap, the details for this structure were essentially identical to those of the cantilevered column as-built structure. The transverse stiffness at the piers was modelled by incorporating a pier cap that effectively constrained the piers to deflect transversely in double curvature. For the isolated structure, lead/rubber bearing elements were inserted at each of the piers and at both abutments. Details were such that no moment was transferred to the top of the piers. The bi-linear hysteretic characteristics of the bearings were explicitly modelled. Longitudinal and transverse movements were permitted at both the piers and abutments.

CalTrans use a set of design ground spectra for various soil conditions and various levels of shaking. The spectrum chosen for this study was one scaled to a ZPA of 0.6g, and for underlying conditions of 0' to 10' of alluvium. The spectra is applicable for bridges located 7 to 10 miles from a major California fault.

Because of the non-linear nature of the response of a base-isolated system, a time history analysis is required. The effect of two simultaneous horizontal components of ground motion was included in the study. Two representative earthquakes, one recorded, one artifically generated, were scaled to give the target ZPA. The earthquakes used were firstly, the two horizontal components of the 1940 El Centro record, both components being scaled by 1.71; and secondly, 1.60 times the artificial CalTech B1 earthquake with 1.0 times B2 as the orthogonal component. A comparison of 5% response spectra are shown in Figs. 4 and 5.

The optimum arrangement for the lead/rubber bearings was evaluated during an extensive parameter study and consisted of a 18-inch-square by 12-inch-high bearing with a 5-inch-diameter lead plug over each column. lead plug. At the abutments each longitudinal girder was placed on an elastomeric bearing with four of the fourteen girders placed on a lead/rubber bearing with a 4-inch-diameter plug. This bearing arrangement ensures that a majority of the energy is dissipated over the columns and consequently puts minimum demands on the superstructure to transfer inertia forces to the abutments.

8 RESULTS OF ANALYSES

The results of the analyses are presented in Tables 1 and 2. The column shear results of Table 1 illustrate the dramatic reduction obtained in the elastic shear demand on the columns when the lead/rubber bearings are incorporated. The two ratios on the right-hand side of the table correspond to the maximum individual column shears divided by the nominal shear capacity of 54 kips and by the overstrength shear capacity of 86 kips, respectively. The column shear results indicate that the bridge configurations in the as-built condition are capable of resisting less than 25% of the ground motions used in the analyses. Using the lead/rubber bearings as a retrofit scheme, the columns are able to respond elastically to the ground motions. In addition, they reduce elastic column shears by a factor of 5 and 10 for the two column configurations, respectively.

The displacements corresponding to the analysis of the as-built structures can only be used for comparative purposes since they are elastic displacements. Clearly, at the level of column shear forces resulting from the analyses, the columns would have suffered signifiant inelastic deformation and in all likelihood would have collapsed. The displacements for the isolated cases correspond primarly to bearing displacements. At the abutments the displacements are totally bearing displacements and at the piers approximately two inches of the total displacement is due to the column displacement for the cantilever columns and approximately 0.75 inches for the pier cap case.

9 CONCLUSIONS

Clearly, the seismic safety of existing bridges is of major concern and recent efforts have been directed towards providing guidelines to solving the problem. Failures in past earthquakes have indicated two major problem areas with existing bridges. The first is inadequate support lengths for superstructure girders and the second is inadequate strength and ductility capacity of supporting substructures.

The paper has presented the application of a recently developed lead/rubber bearing for the retrofit evaluation of two bridges. The bearings provide both base isolation and energy dissipation characteristics that decrease superstructure displacements and provide a force-limiting mechanism for the supporting substructure. The two bridges evaluated are of similar dimensions, but have different supporting substructures. In their as-built condition the bridges are capable of resisting less than 25% of the CalTrans design ground motion for a site close to a major fault. Incorporation of the lead/rubber bearings between the superstructure and the columns and abutments enables the bridge columns to respond elastically to the design ground motion. The elastic column shears are reduced by a factor of 5 for the bridge supported on cantilever columns and by a factor of 10 when it is supported on columns connected with a pier cap.

The results of the analyses presented indicate that the base isolation and energy dissipation characteristics of the lead/rubber bearings offer an attractive solution to the major problems encountered in the seismic retrofit of a highway bridge.

10 REFERENCES

[1] Applied Technology Council, "Seismic Design Guidelines for Highway Bridges," Palo Alto, CA., October, 1981.

[2] Kelly, J.M., "Aseismic Base Isolation: Its History and Prospects," Proceedings World Congress on Joints and Bearings, ACI Publication SP-70.

[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, Report No. ATC-6-1, Nov. 1979.

[4] Blakeley, R.W.G., et al., "Recommendations for the Design and Construction of Base Isolated Structures," Bulletin New Zealand National Society for Earthquake Engineering, Vol. 12, No. 2, 1979.

[5] Buckle, I.G., "The Use of Energy Dissipators and Base Isolation in the Design and Retrofit of Bridges and Buildings," Proceedings of 51st Structural Engineers Association of California Convention, October, 1982.

[6] Robinson, W.H., "Lead-Rubber Hysteretic Bearings Suitable for Protecting Structures During Earthquakes," Journal of Earthquake Engineering and Structural Dynamics, Vol. 10, 1982.

TABLE 1
Column Shear Results

Column Detail	Base Isolation	Earth-quake	Maximum Column Shears Individual	Individual Nominal	Individual Overstrength
Cantilever Columns	No	1	327	6.06	3.80
	No	2	376	6.96	4.37
	Yes	1	55	1.02	0.64
	Yes	2	67	1.22	0.78
Transverse Pier Cap	No	1	692	6.41	4.02
	No	2	457	4.23	2.77
	Yes	1	56	0.52	0.33
	Yes	2	74	0.69	0.43

TABLE 2
Superstructure Displacement Results

Column Detail	Base Isolation	Earth quake	Maximum Displacement (Inches) Long'l.	Transverse Abutment	Transverse Center Pier
Cantilever Columns	No	1	4.8	-	11.0
	No	2	4.7	-	12.9
	Yes	1	6.2	6.4	9.6
	Yes	2	3.4	8.6	13.3
Transverse Pier Cap	No	1	4.8	-	9.1
	No	2	4.9	-	7.3
	Yes	1	6.3	5.6	8.2
	Yes	2	3.3	8.5	13.4

<u>Notes</u>:

1. Earthquake 1 is 1.7 times the NS and EW El Centro recorded ground motion. The NS component is applied in the transverse direction.

2. Earthquake 2 is 1.6 times the artificial CalTech B1 earthquake applied in the transverse direction and 1.0 times the B2 earthquake in the longitudinal direction.

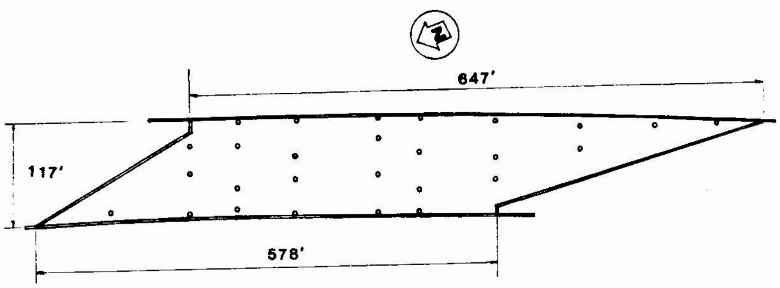

Figure 1 : BRIDGE PLAN

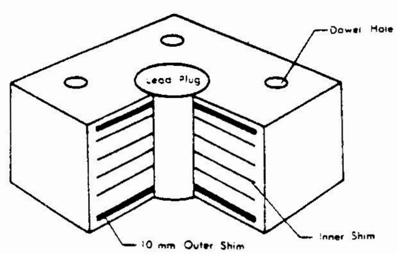

Figure 2 : LEAD RUBBER BEARING

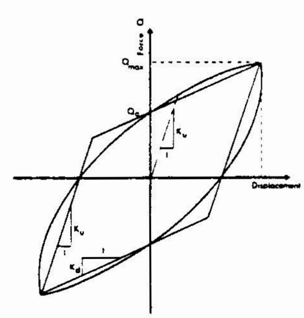

Figure 3 : HYSTERESIS LOOP

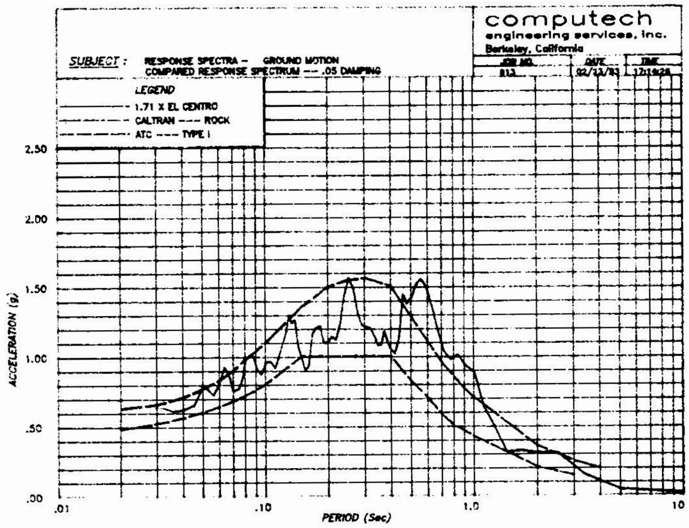

Figure 4 : EL CENTRO SPECTRUM

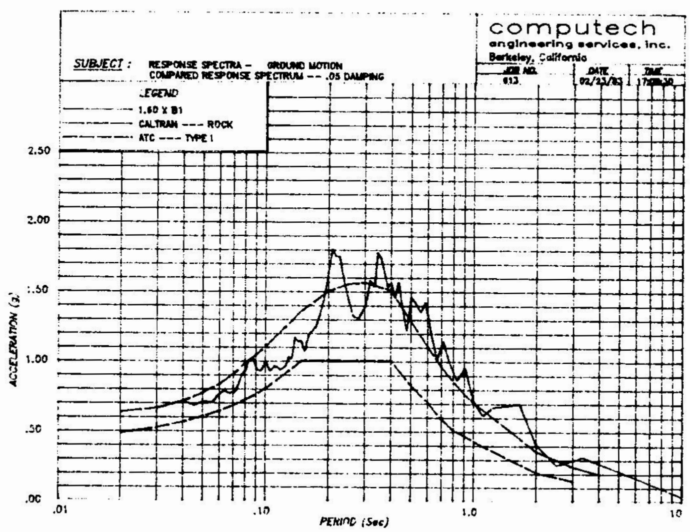

Figure 5 : B1 SPECTRUM