SEISMIC RETROFIT OF BRIDGES UTILIZING DUCTILE BASE ISOLATION CONCEPTS

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SUMMARY

This paper describes the retrofit of an existing freeway overpass to improve earthquake performance by the installation of lead-rubber bearings between the superstructure and the supporting columns. In its present configuration the columns of the bridge are capable of resisting approximately one-quarter the design site earthquake. The base isolation and energy absorption characteristics of the lead-rubber bearings improve this performance such that the columns remain essentially elastic for the full design earthquake.

INTRODUCTION

The collapse of a highway bridge during an earthquake will sever vital transportation routes at a time when they are most needed to provide emergency services. This loss of function may result in a greater loss of life than the immediate effects of collapse. The San Fernando earthquake of 1971 demonstrated the potential inadequacy of past design procedures in providing seismically resistant bridges and many bridges in seismically active areas may have insufficient strength to resist seismic loading.

Recent developments on the use of base isolation and energy dissipating mechanisms for the design of new bridges offer potential solutions for overcoming deficiencies in pier and column strength and girder support lengths in existing bridges. A bridge on the current California retrofit program had severe deficiencies in these areas and was capable of resisting less than one-quarter the design earthquake. Increasing column strengths would be expensive and difficult because of site constraints. The concept of reducing forces through base isolation was investigated and demonstrated the dramatic improvement in seismic performance that could be attained. This paper describes the steps in designing, evaluating and implementing a base isolation scheme into this bridge.

RETROFIT CONSIDERATIONS

The California Department of Transportation (CalTrans) has an ongoing program of retrofitting bridges to improve seismic performance. This program primarily involves the provision of positive connections between the superstructure and supporting substructure. This solution partially remedies the lack of sufficient support lengths for girders but at this stage no attempt

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has been made by CalTrans to strengthen potentially inadequate supporting substructures such as columns and piers because of the expense of conventional means of achieving this. More detailed Applied Technology Council (ATC) guidelines for new bridges and retrofit (Ref. 1) provide means of assessing the elastic force and displacment demands on existing structures and comparing the demands to the ultimate capacities of resisting elements. The evaluation includes assessments of the ability of columns to resist post elastic deformations, the portion of the design earthquake which each component is capable of resisting, the consequences of failure of each component and the importance of the structure and consequences of loss of function.

BASE ISOLATION CONCEPTS

The isolation of structures from the damaging effects of earthquakes is not a new idea. The first patent for a base isolation scheme was taken out as early as 1909 but until 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 (Ref. 2). 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 (Refs. 3, 4). 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.

Principles of Base Isolation

There are two basic elements in practical base isolation systems, as follows:

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.

An elastomeric bearing with a lead plug inserted into a preformed hole is an efficient means of providing both these elements in a single unit (Ref. 5). Bridge structures have for a number of years been supported on elastomeric bearings and thus the concept of the flexible mount is not new. 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.

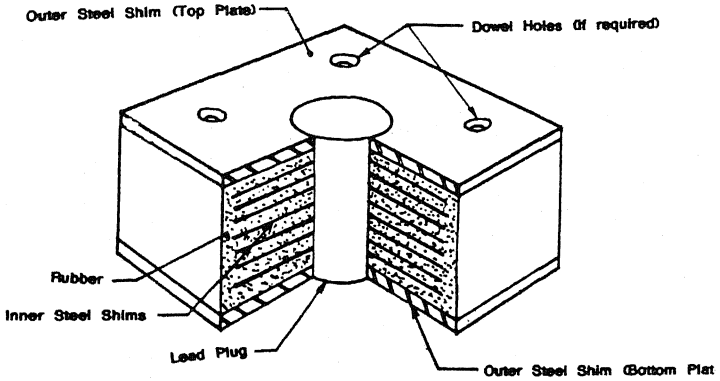
CHARACTERISTICS OF LEAD-RUBBER BEARINGS

A lead-rubber bearing comprises alternate layers of rubber vulcanized or cemented to thin steel shims, typically with a lead plug placed in the center (Fig. 1). The internal steel shims plus the externally applied vertical load provide confinement for the plug. The rubber in the bearing carries the weight

of the structure and the confined 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.

The bearing is designed to resist low levels of shear, such as wind loads, elastically with a high initial stiffness (K_u) until a yield level determined by the characteristic strength, Q_d, is reached. 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 bi-linear hysteresis curve formed by these two stiffnesses has a stable form and large enclosed area, demonstrating the energy absorption properties.

FIGURE 1 : TYPICAL LEAD-RUBBER BEARING CONSTRUCTION



BRIDGE RETROFIT

The bridge to be retrofitted is a freeway overpass which crosses a secondary road and railroad tracks. The plan dimensions of the bridge, shown in Figure 2, are 615 feet long overall and 117 feet wide. The bridge is on a skew of approximately 60 degrees with respect to the highway traffic lanes. The construction is concrete deck slab 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 bents of 4 columns, reducing to two or three column bents at the ends of the bridge due to the skew. The foundations are individual pads beneath each column bearing on the underlying sandstone. The steel superstructure consists of a set of longitudinal girders connected to main transverse girders across each column bent. The concrete deck has expansion joints over each pier and at the abutments and at these positions the main girders on one side are free to move longitudinally.

The bridge was selected for retrofit in accordance with the CalTrans program. The structural form of the substructure, cantilever columns, was such that the bridge had a very low seismic capacity, less than one-quarter the site design value of 0.6g maximum ground acceleration, and column detailing was such

that no significant ducility could be relied upon. Conventional strengthening measures were considered but proved very expensive and almost impossible to implement due to the presence of railroad tracks adjacent to the columns. An alternative to designing for the maximum forces, that of reducing the forces transmitted into the columns by the maximum earthquake, was then investigated, leading to the decision to assess the practicality of isolating the superstructure.

Bearing Design Procedures

The form of base isolation was constrained both by physical restrictions imposed by the present bridge bearings and by performance requirements. Bearings were developed to replace existing steel bearings and to meet constraints on height (10" and 11") and plan dimension (less than 19" square). To obtain high vertical load capacity 3/8" rubber layers were adopted. This was a compromise between load capacity and lateral flexibility. Thicker layers have greater lateral flexibility for the same total height but a lower non-seismic vertical design load. Thinner layers increase the vertical non-seismic design load but with the penalty of higher lateral stiffness for the same total height, i.e. more of the height is taken up by shims.

These considerations provided two bearing constructions, each 18" square with 1/8" shim plates and 3/8" rubber layers with 3/4" outer shim plates. One bearing was composed of 15 layers for a total height of 8.875" and the other of 17 layers providing 9.875" total height. These bearings had a maximum vertical load capacity of 482 kips, less than the maximum load of 678 kips at some positions. Therefore 22 inch diameter bearings were also designed, using the same construction as the 18 inch bearings. The circular plan shape allowed bearings of larger size to remain partially within the perimeter of the 19" square existing base plates. A total of 21 bearing locations had total loads (DL + LL) less than 482 kips and therefore were suited for 18" square bearings based on these non-seismic vertical design loads. The remaining 12 bearing locations required 22" circular bearings.

Once the basic bearing construction was determined the diameter of the lead plugs and distribution was determined to provide the optimum yield level of 5% of the superstructure weight. The abutment bearings were designed with 6" plugs, the maximum size for their plan dimension. The remainder of the 5% of the weight was achieved by using 3" plugs in all 18" pier bearings and 4" plugs in all 22" bearings. Two abutment bearings had low axial loads, insufficient to adequately confine the lead, and these bearings were specified as elastomeric only.

Prior to a detailed analysis an approximate single-degree-of-freedom estimate of the response quantities was made using procedures provided in Reference 6. The effective period was computed to be 1.76 seconds, and for this period the non-linear spectrum from Reference 6 provided a maximum acceleration of approximately 0.20 g and a maximum displacement of approximately 7.5" inches. These predictions proved accurate as the mean maximum displacement from the 5 time history analyses was 7.0 inches and the mean shear force was 0.20g. Note that this simple check provided only overall response and did not provide detailed checks on the shear force transmitted into each pier column.

Analytical Model

The bridge was analyzed using the ANSR-II computer program (Ref. 7) for

the three-dimensional analysis of non-linear structures. This program permits the bi-linear hysteresis curve of the lead-rubber bearings to be included. The model provided the response of the bridge to both longitudinal and transverse earthquake motions. Each column and each bearing was modelled as a three-dimensional beam element, with the columns remaining elastic and the bearing elements having bending properties and yield moments equivalent to the shear stiffness and characteristic strength of the lead-rubber bearings. The earthquake input appropriate for the site was the CalTrans 0.6g maximum acceleration on a rock site. Time histories were obtained for the analysis by frequency scaling a suite of recorded and artifical time histories to closely match the "target" spectrum. The starting histories were selected as El Centro, 1940 N-S, Taft, 1952 S69E, Olympia 1949 NO4W and Caltech artificial records A1 and A2.

Results of Analysis

The results from the series of 5 time history analyses are presented in Table 1. Tabulated values include strains, forces and force ratios for all 5 records plus the mean value of each quantity. Figure 3 illustrates the time history of the superstructure response for the analysis using the record based on El Centro 1940, N-S.

TABLE 1 : OVERALL RESULTS

RESPONSE QUANTITY	EARTHQUAKE (Scaled to 0.6g CalTrans)					MEAN VALUE
	El Centro	Taft	Olympia	Caltech A1	Caltech A2	
Maximum Displacement (inches)	6.44	9.73	7.20	3.71	7.77	6.97
Maximum Bearing Strain	1.14	1.73	1.28	0.66	1.38	1.24
Total Shear Force (kips)	1728	2388	1763	1116	1870	1773
V/W	0.20g	0.27g	0.20g	0.13g	0.21g	0.20g
Maximum Column Shear (kips)	53.7	74.2	56.0	34.13	58.9	55.4
V_{max}/V_n	1.06	1.49	1.10	0.69	1.15	1.10
V_{max}/V_o	0.65	0.92	0.68	0.42	0.71	0.68

The results of two time histories were inconsistent with those from the other analyses : Taft results were higher than all others and the Caltech A1 values were lower. This illustrates the deficiency of a response spectrum as a means to characterize input for non-linear systems in that all records produce similar elastic response spectra. In the evaluation of the results more weight has been placed on the 3 records which produced consistent results.

The maximum shear forces in each column were divided by the nominal capacity, V_n, and the overstrength capacity, V_o. The latter quantity is based on the ATC-6 retrofit studies which conclude that columns will perform adequately provided the elastic moment to which they are subjected does not exceed 1.625 times the nominal moment capacity. The mean nominal force ratio was 1.10 and the mean overstrength ratio was 0.68. All records produced overstrength ratios less than unity. Maximum bearing strains were 114%, 128% and 138% for the El Centro, Olympia and Calteach A2 based time histories respectively. The upper

limit used for new design is generally accepted to be 100% but values up to 150% are accepted for retrofit for a limited number of cycles. In this application a maximum of 4 cycles exceeded 100% strain.

The results of the analyses demonstrated that the bearing layout and construction selected provided satisfactory performance for a retrofit situation. Detailing for the installation of the bearings into the structure was then performed. Figure 4 shows a typical detail for installing the lead-rubber bearing beneath the main girders in place of existing steel bearings.

CONCLUSIONS

This paper has presented the retrofit of an existing bridge with lead-rubber bearings to provide base isolation and energy dissipation. In its present configuration the bridge has serious seismic resistance deficiencies and is able to resist less than one-quarter the design earthquake. Installation of lead-rubber bearings reduces forces transmitted into the weak columns to such an extent that the bridge will respond elastically to the maximum design earthquake for the site.

ACKNOWLEDGMENTS

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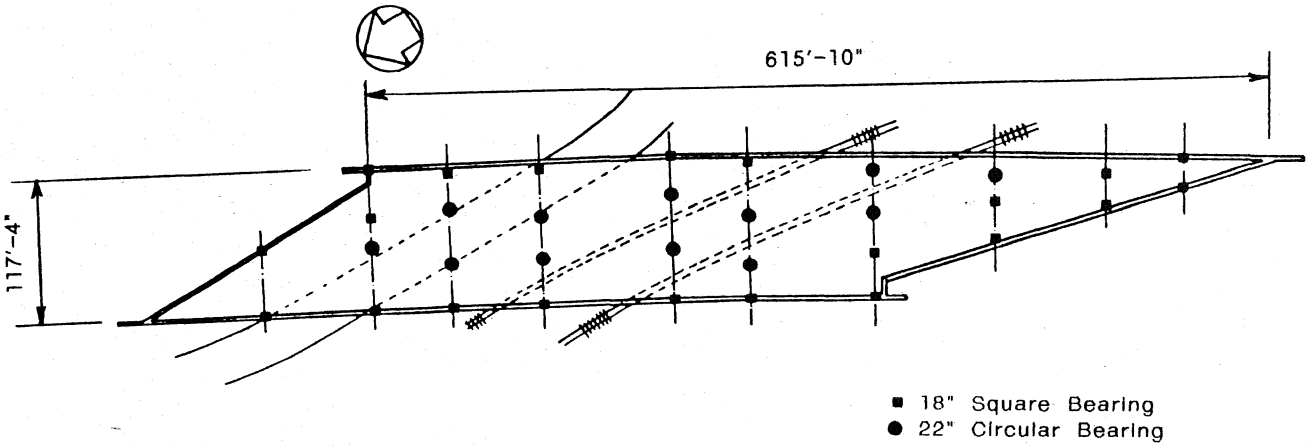

FIGURE 2 : PLAN VIEW OF BRIDGE

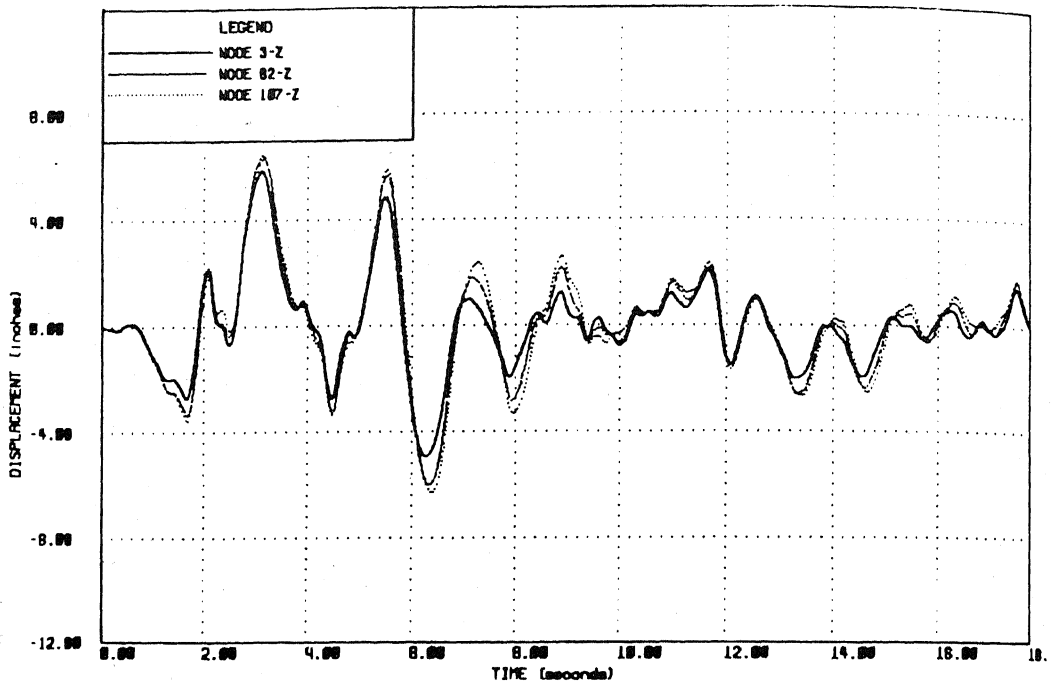

FIGURE 3 : DISPLACEMENT RESPONSE : EL CENTRO BASED RECORD

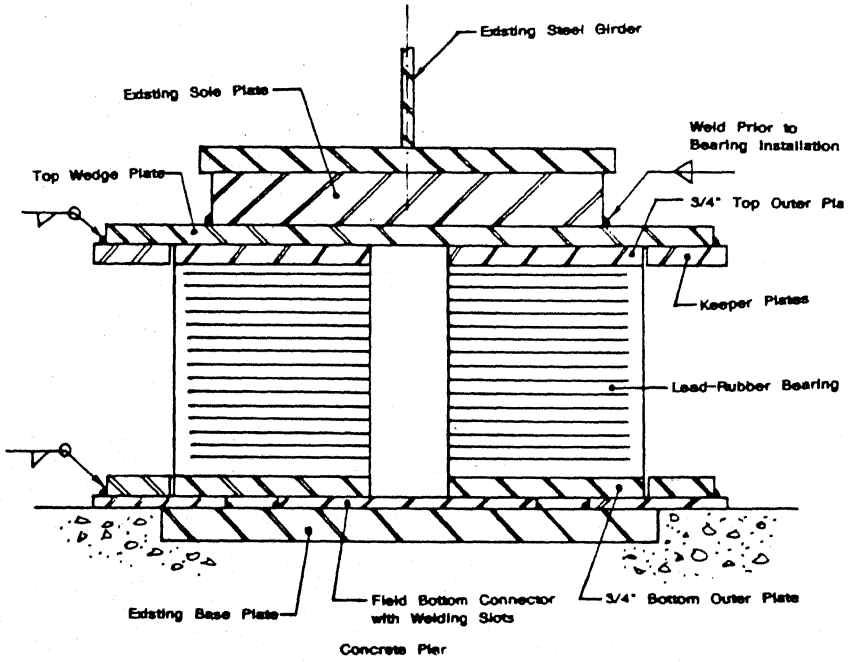

FIGURE 4 : TYPICAL INSTALLATION DETAIL