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EVALUATION OF A SEISMIC RETROFIT CONCEPT FOR DOUBLE-DECK REINFORCED CONCRETE VIADUCTS

FOUED ZAYATI and STEPHEN A. MAHIN

Department of Civil Engineering
Earthquake Engineering Research Center
University of California, Berkeley
Berkeley, CA 94706

ABSTRACT

A third-scale model of a retrofit bent (Bent B-8) from the Alemany Freeway (I280-101 interchange) was built and tested at the structures laboratory at the University of California at Berkeley. After the original tests were completed the test structure was repaired and retested. The retrofit structure performed well up to an overall displacement ductility of 4.3. Damage was concentrated in the lower column where plastic hinging occurred. The performance of the repaired structure was satisfactory. The repaired structure regained about 85% of its original stiffness and was able to achieve a higher base shear capacity than the original structure. Multiple non-linear time history analyses using various lateral load intensities resulted in displacements at the lower level that are significantly higher than those predicted from elastic analyses. The analyses also showed that the peak structural displacement under bi-directional input motion increased, on average, by 19% compared to the displacement resulting from uni-directional input motion.

KEYWORDS

Double-Deck Viaduct, Seismic Retrofit, Reinforced Concrete Viaduct, Bridges, Experimental Study, Dynamic Response, Time History Analysis, Soft Story Response.

INTRODUCTION

Seven continuous-span double-deck viaduct structures were built in the San Francisco Bay Area during the 1950's and 1960's. Six of these viaducts were damaged during the 1989 Loma Prieta earthquake (Housner, 1990). Damage ranged from cracking to collapse. The deficiencies in these viaducts included the lack of beam-column joint shear reinforcement, inadequate column shear and confinement reinforcement, insufficient cap beam positive moment capacity, and the absence of an effective longitudinal framing system. To simplify the analysis and design, the frames contained numerous pin connections (shear keys) at the ends of columns, rendering the structure statically determinate at many locations. This lack of redundancy is also a serious seismic deficiency. Comprehensive retrofit measures were required to prevent collapse of these viaducts during a major earthquake. However preventing collapse was not the only performance criteria imposed on the double deck viaducts. Due to their importance as lifeline structures, a serviceability requirement was imposed on these structures.

RETROFIT CONCEPT

The retrofit concept presented in this paper was the result of consultations between Caltrans engineers, outside design consultants, a peer review panel, and technical advisors from various universities. The initial retrofit concepts focused primarily on increasing the seismic resistance to transverse loading. However, studies of double-deck viaducts containing in-span expansion joints, and extending for long distances over varying soil conditions (Singh *et al.*, 1994) demonstrated that significant longitudinal seismic response is to be expected. Since the existing longitudinal load carrying system was inadequate, it was necessary that the retrofit concept included measures for the longitudinal as well as the transverse direction.

After considering various alternatives for the longitudinal direction, the edge girder alternative, which offered a conventional load carrying system, was chosen. The retrofit called for adding edge girders between the centers of columns at the level of the lower deck. The edge girder could be integrated with the superstructure to increase its effectiveness and minimize the problems in transferring the inertial loads. Adding the edge girders required the removal of the concrete in the beam-column joints. Since removing the joint concrete isolates the existing columns from the superstructure and requires complete shoring of the superstructure, it became possible to replace the existing columns with spiral reinforced columns consistent with modern design. The cap beams could be strengthened with the addition of bolsters. The bolsters may be reinforced and partially prestressed to carry all the required transverse moments.

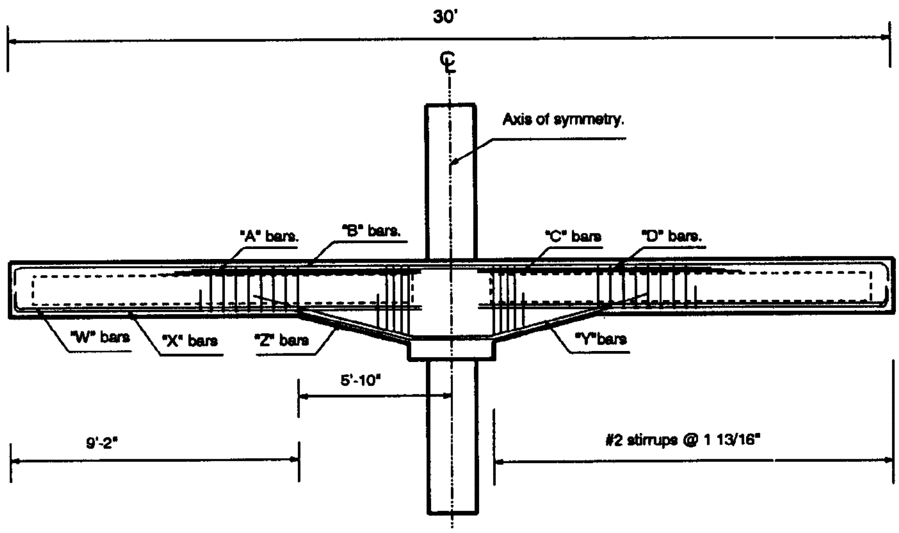

Fig. 1. Test Structure, longitudinal elevation.

PROOF TEST PROGRAM

New and potentially controversial design and detailing approaches were required in the proposed viaduct retrofits (Zayati, 1995). Therefore the actual performance of the retrofitted structures in the design events was therefore uncertain. Furthermore, in the aftermath of a damaging earthquake, the repairability of the viaduct structure was not known. Given these uncertainties and the stringent performance criteria, the peer review panel recommended that laboratory proof tests be undertaken. The proof tests had the following goals: (1) To characterize response of the retrofitted structure under the action of bi-directional cyclic loading, (2) to identify areas where design improvements could be made, and (3) to assess the repairability of the system.

To help achieve these objectives, a one-third-scale model (Fig. 1) of a half bent (bent B-8) from the Alemany freeway was designed and built at the Structures Laboratory at the University of California at Berkeley. The test structure was equipped with hydraulic actuators, such that bi-directional loads could be applied. The lower level was under displacement control, while a control algorithm applied forces at the upper level that were proportional to those of the lower level. The test specimen was instrumented to measure global forces and displacements, reinforcement strains, and local deformations.

Analytical investigations were also undertaken with the goal of studying certain response aspects that affected structural demands. Among the aspects investigated were the effects of soft story response and bi-directional seismic loading on structural and local displacement demands.

RESPONSE OF THE RETROFIT MODEL

The retrofit specimen performed well up to structural displacement ductility of 4.3. Damage was concentrated in the lower column and bottom face of the beam-column joint. At the conclusion of the original tests, concrete cover spalling spread over a significant portion of the column, the concrete core in the column plastic hinge zone was completely pulverized, and buckling of the column's longitudinal reinforcement was noticed. Concrete cones at the bottom face of the joint around the column had pulled out. This type of joint cover spalling is usually associated with bar pullout (Soleimani, 1979).

The lateral displacement-base shear relationships developed in the lower level during transverse and longitudinal displacement cycles are shown in Fig. 2 and 3. The hysteretic loops are generally full, exhibiting the stiffness deterioration characteristic of reinforced concrete, but with limited pinching or degradation of strength. A positive post-yield stiffness is observed. This is an important factor in lessening the P-delta effects on the performance of the structure.

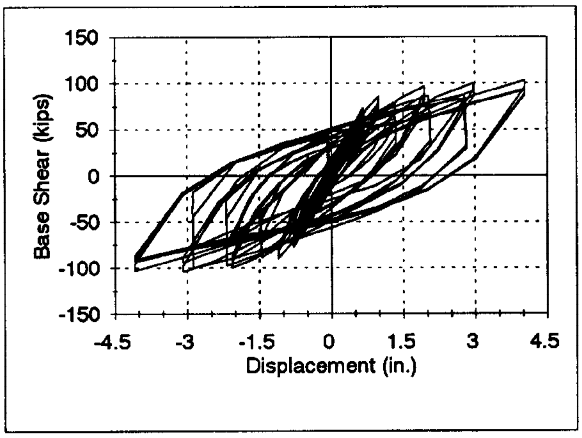

Fig. 2. Lower-level force-displacement history (longitudinal).

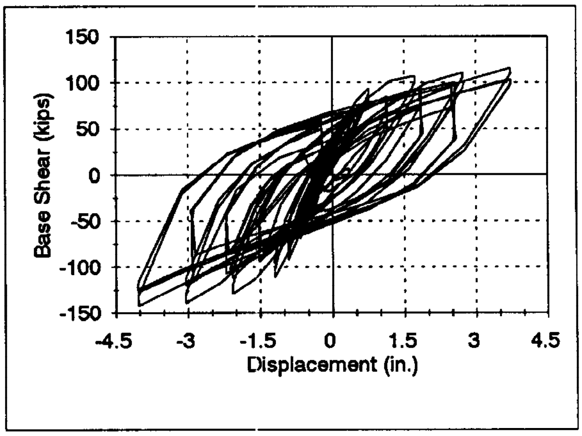

Fig. 3. Lower-level force-displacement history (transverse).

RECOMMENDED DESIGN IMPROVEMENTS

The nominal capacities for the various component of the structure are compared to the applied moments in Table 1. With the exception of the lower column, the applied moment was significantly lower than the nominal capacity for the other elements. Note that the cap beam and edge girder were significantly overdesigned.

When longitudinal force moments are applied to the columns, flexure in the edge girder and box girder and torsion in the cap beam, all participate in resisting these column moments. A similar mechanism occurs under transverse force moments. However when the edge girder/cap beam was designed, it was assumed to be the only member resisting the column moments. The design moments were based on plastic hinging in the upper and lower columns and a plastic to nominal moment capacity ratio of 1.5. The experimental data showed the box girder to carry as much as 27% of the total applied moments (a higher contribution can be achieved by reducing the stiffness of the edge girder relative to that of the box girder. To achieve the 1.5 factor of safety against box girder yielding it is sufficient to design the edge girder for 1.1 (1.5 *(1-0.27)) times column nominal moments. Since the column's plastic moment is likely to be about 1.3 times its nominal moment capacity, the edge girder would be at about 86% of its nominal moment capacity, when both the upper and lower columns reach their plastic moments. Torsion in the edge girder and flexure in the box girder should have similar effects in the transverse direction.

Table 1. Comparison of Applied Moments to Member Capacities.

	Positive (Kip.in)			Negative (Kip.in)		
Member	Applied Moment	Nominal capacity	Ratio app./cap.	Applied Moment	Nominal capacity	Ratio app./cap.
Upper Column	3706	5725	0.65	4653	5725	0.81
Lower Column	7488	5865	1.28	9212	6472	1.42
Cap Beam	14692	30147	0.49	18203	34661	0.53
Edge Girder	4448	10253	0.43	5374	18116	0.30
Box Girder	2388	5124	0.47	1294	7290	0.18

It was earlier noted that the structural displacement ductility capacity was equal to 4.3. The corresponding lower-column ductility capacity was equal to six. The lower structural ductility capacity was due to the fact that the structural plastic displacement were mainly contributed to the lower column while the structural yield displacement was contributed by all of the structural components. Soft story response resulted in limited structural reserve displacement ductility. The ductility capacity exceeded the required ductility by 8% ((4.3-4)/4). This implies that the structure would have to rely on factors of safety associated with the design ductility to help prevent serious damage following a design level earthquake. To increase the factor of safety associated with the displacement ductility, the design displacement ductility could simply be reduced. This implies that the bent would be designed for higher base shears, which would lead to a higher retrofit cost.

Alternatively, the contribution of the upper column to plastic displacements can be increased. Other elements, such as the cap beam edge girder and beam column joint could also contribute to the plastic deformations. However this could not be achieved without compromising the serviceability criteria. The upper-level columns are not likely to yield under current design. To achieve any level of plastic deformations in the upper level, upper-level column strength should be between 50 and 75% of the lower-level column strength. A reduction to less than 50% is likely to produce a soft story in the upper level which could lead to adverse dynamic effects. Increased damage to the structure (increased repair cost) would result under this approach as a result of upper column hinging. It should be noted that this approach would lead to a reduction in joint design forces.

RESULTS OF TESTS ON THE REPAIRED STRUCTURE

<u>Repair of the Damaged Structure.</u> A major advantage to concentrating structural damage into limited areas is the relative ease of repair following a major earthquake. Repair of the test structure involved the injection of epoxy into lower-column and beam-column joint cracks and recasting the cover concrete in the lower-column's plastic hinge zone and at the bottom face of the joint. A partial height steel jacket covering the upper half of the column was also added. A one inch gap was introduced in the jacket at six inches below the joint in an attempt to spread the plasticity over a longer portion of the column.

<u>Behavior of the Repaired Structure.</u> The ability of the structure to withstand additional earthquake effects after repair was assessed. The load cycles imposed on the repaired specimen were similar to those of the original tests. Displacement amplitudes were increased beyond the design level in later cycles to nominal displacement ductilities

of 5 and 5.5. Behavior of the specimen was similar to that observed for the initial retrofit structure, with damage concentrating in the lower column. The jackets were observed to slip relative to the column concrete, and large horizontal cracks were observed at the top of the column and at the gap between the jackets. During the last cycle the reinforcement began to fracture. Concrete in the upper two inches of the lower column was pulverized, allowing the longitudinal reinforcement bars to buckle and subsequently fracture under the repeated cyclic loading imposed by the bi-directional loading. At the conclusion of the test, more than half of the reinforcement had fractured.

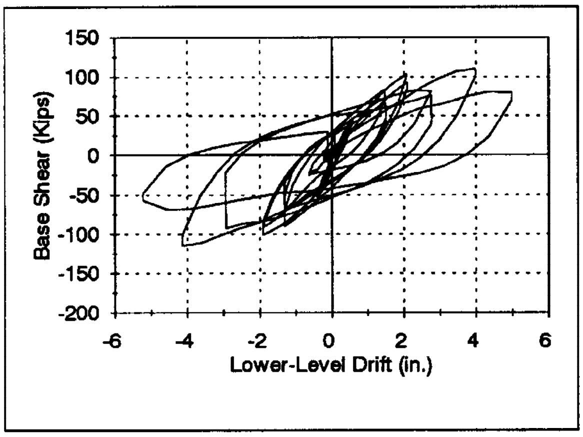

Fig. 4. Lower-level force-displacement history (longitudinal).

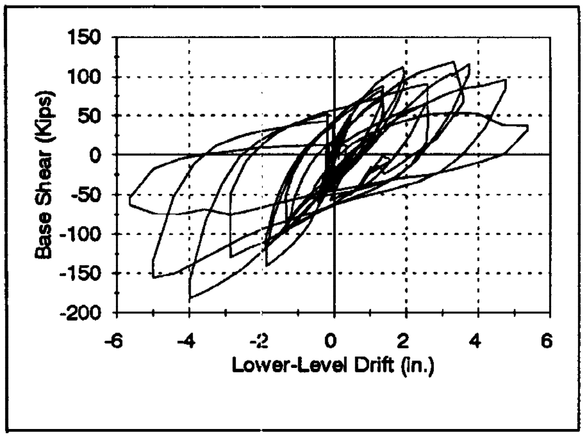

Fig. 5. Lower-level force-displacement history (transverse).

The hysteretic loops (Fig. 4 and 5) for the repaired specimen were not as full as those of the initial retrofit structure. Some pinching of the loops was detected, and resistance degraded more, compared to the original test) between the first and subsequent cycles to the same displacement levels. However, the strength of the structure was maintained to displacements in excess of the design level. The initial stiffness of the repaired structure was slightly smaller than that of the original specimen. Many cracks throughout the structure were not (or could not) be repaired. It is believed that attempts to repair the bond deterioration along column bars extending into the joint were largely ineffective. The strength of the structure eventually exceeded that observed for the original specimen.

Table 2. Elastic vs. Inelastic Upper-Level Displacement Demands

Load Case #	1	2	3	5	7	9	11	13
UL Peak Inelastic Displacement (in.)	5.4	12.0	20.6	7.6	9.3	9.5	10.8	10.8
UL Peak Elastic Displacement (in.)	6.1	12.2	18.3	8.2	12.1	10.0	7.6	8.7
Ratio of Inelastic to Elastic Displacement	0.9	1	1.1	0.9	0.8	1	1.4	1.2
Force Reduction Factor	1.8	3.5	5.3	3.6	3.8	3.7	4.5	2.7

TIME HISTORY ANALYSES RESULTS

The effects of bi-directional input and soft-story response on structural and local displacement ductility demands were evaluated using non-linear time history analyses using DRAIN-3DX (Prakash *et al.*,1992). The analysis model was based on a single bent from a full scale retrofit double-deck viaduct structure. Rock and soft soil site records from the 1992 Northridge earthquake (Silmar and Santa Monica Records) were adopted for the analyses. The Rock site records were filtered through the SHAKE program (Schnabel *et al.*, 1972) using a soft soil column from a typical San Francisco site resulting in a total of four records for the study. Thirteen load cases were formulated using one and two components of each of the records and varying the peak ground acceleration in part of the load cases (see Zayati, 1995 for details).

The peak elastic displacement demand in the upper-level (Table 2) was on average 7% lower than the inelastic demand, which suggested that elastic analyses could be used to estimate upper-level displacement demands. The lower-level displacement , on the other hand was significantly different from the elastic displacement demand (Table 3). Note in Table 3 that while the elastic displacement ratio is equal to 0.55 on average, the inelastic displacement ratio varied with the force reduction factor and was significantly higher than 0.55. Given the force reduction factor (Z) upper-level peak displacement (δ_{ul}), which could be obtained from an elastic analysis, the lower level peak displacement (δ_{ll}) may be computed as,

$$\delta_{ll} = \delta_{ul} * (Z + 15) / 24 \qquad (1)$$

It should be noted here that the Z value should not be the design value, but rather the actual value based on the actual bent capacity divided by the elastic force demand.

Table 3. Elastic and inelastic ratio of Upper to Lower-level displacement

Load Case #	1	2	3	5	7	9	11	13
Force Reduction Factor	1.76	3.52	5.28	3.65	3.76	3.72	4.55	2.72
Elastic Displacement Ratio	0.52	0.52	0.52	0.58	0.58	0.57	0.57	0.55
Inelastic Displacement Ratio	0.73	0.79	0.86	0.8	0.76	0.7	0.87	0.71
Inelastic Ratio (Based on Eq. 1)	0.73	0.8	0.87	0.81	0.81	0.81	0.84	0.77

The effect of using one component of the earthquake or both components was examined (Table 3). The uni-directional peak displacement was on average 12% higher in the bi-directional analysis, while the peak of the vector sum of transverse and longitudinal displacements was 19% higher on average. It is therefore recommended that the peak structural displacement from an elastic analysis be increased to account for non-linear bi-directional effects.

Table 4. Effect of Bi-Directional Input Motion on Structural Demands

Load Case #	2/4	5/6	7/8	9/10	11/12	13/14
Upper-Level Displacement Ratio	1.04	1.18	1.05	1.01	1.07	1.37
Upper-Level diagonal Ratio	1.10.	1.31	1.17	1.07	1.08	1.38
Force Reduction Factor	3.52	3.65	3.76	3.72	4.55	2.72

CONCLUSIONS

A three-dimensional, one-third-scale model of a retrofit bent from the Alemany freeway was constructed and tested. Nearly all the damage was concentrated in the lower column, which placed a higher ductility demand on the column (compared to the structure). Nonetheless, the structure was able to develop its full design ductility, even though it was subjected to a large number of cycles and bi-directional loading.

Adding the edge girder to the structural system provided a simple and effective alternative to modifying the box girder itself. It provided excellent protection for the box girder against any significant damage. Strain levels in the edge girder reinforcement were about one-third of yield, which suggests that the strength of the girder could be reduced. The cap beam strength should also be reduced.

It was observed that the test structure possessed limited reserve ductility. Conservative design assumptions and stringent design criteria lead to this result. Two alternative approaches to increasing the reserve displacement ductility capacity were presented. The first approach reduced the design ductility, while the second approach relied on increasing the displacement ductility capacity of the structure by reducing the strength of the upper column so that a plastic can form in the column. It is recommended that the strength of the box girder and

The damaged test structure was repaired and retested. The repair process showed that it was relatively easy to repair the structure. This was the direct result of the concentration of the damage in the lower-column and the lower part of the joint. Tests indicate that it is feasible to repair the structure so that it can withstand the effects of a second design-level event. The use of steel jackets was shown to concentrate damage in a narrow band near the top of the column.

While this study is focused on the response of retrofit double-deck structures, many of the findings in this study can be also be applied to the retrofit of single-deck multiple-column viaducts. Take the example of a single level bent with tall columns that may have problems satisfying drift limits. Transverse drifts of such structure can be reduced by about 400%, when a transverse beam is added at mid-height of the columns. This beam can be designed in an identical fashion as the retrofit cap beam of the structure in this study. The joint design approach(reference) can also be applied to the joint connecting the new transverse beam to the existing columns.

Results of analytical studies showed that the soft story response required lower level displacements that are significantly higher than would be predicted using elastic analyses which results in local displacement ductility demands that are significantly higher than the structural ductility demands. Using two-component input motion resulted in 12% higher displacement demands than using a single component input. It is recommended that lower level displacement demands be increased to account for both the soft story response and bi-directional input.

REFERENCES

Housner, G. W., Chairman (1990). Competing against time. The governor's board of inquiry on the 1989 Loma Prieta earthquake, State of California, Office of Planning and Research.

Prakash, V., G.H. Powell, and F.C. Filippou (1992). DRAIN-3DX: Base program and user guide. Report No. UCB/EERC-92/30, Earthquake Engineering Research Center, University of California, Berkeley.

Schnabel, P.B., J. Lysmer, and H.B. Seed (1972). SHAKE: A computer program for earthquake response analysis of layered sites. Report No. UCB/EERC-92/30, Earthquake Engineering Research Center, University of California, Berkeley.

Singh, S.P. (1994). Earthquake analysis and response of multi-frame bridges. Doctoral dissertation, Department of civil engineering, University of California, Berkeley.

Soleimani, D., E.P. Popov, and V.V. Bertero (1979). Hysteretic behavior of reinforced concrete subassemblages. *ACI journal*, **76**, 1179-1195.

Zayati, F. (1995). Experimental and analytical evaluation of a retrofit double-deck viaduct structure. Doctoral dissertation, Department of civil engineering, University of California, Berkeley.

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