SEISMIC PERFORMANCE AND RETROFIT OF BRIDGE FOOTINGS

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Abstract

This study investigated retrofitting measures for improving the seismic performance of the foundations of existing bridges. Tests were conducted on 1/3-scale footing and column assemblages which incorporated details selected to represent deficiencies common in older bridges. Tests on as-built specimens resulted in a brittle failure due to insufficient joint shear strength in the column/footing connection. An added reinforced concrete overlay provided an effective retrofit for the as-built footings. Increased overturning resistance was achieved by enlarging the footing plan size, by providing additional piles, or by providing tie-downs through the footing. For multi-column bents, the addition of a stiff link beam just above the footings was effective in preventing damage in the footings during testing.

Introduction

Many older bridges were designed for primarily gravity loads with little or no lateral forces from earthquake loading being considered. As a result, the foundations in many older bridges are undersized, making them overturning critical. Further, the foundations typically contain no top reinforcement and may be susceptible to brittle flexural failures in an earthquake. Older foundations may also be susceptible to shear failures, both within the footings and in the column/footing joints. When piles are present, there is often no structural connection between the piles and the pile cap. All of these foundation problems may be exacerbated by retrofit measures applied to other sections of the bridge.

Major efforts have been undertaken to develop and implement strengthening or retrofit strategies to improve the seismic performance of older bridges (FHWA, 1995). Much of the initial focus of the retrofit programs was to improve the performance of the superstructures in earthquakes. Subsequently, substructure retrofit efforts were undertaken, mainly associated with column retrofitting. Only recently have strengthening efforts been directed at improving the performance of existing footings, and limited testing has been performed to verify footing retrofit methods. The results of this research provide a basis for designing retrofit measures to improve the seismic performance of the foundations of existing bridges.

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Test Specimens

For this study, a section of a typical bridge substructure, consisting of one or more columns and supporting footings, was used as the basis for evaluating as-built and retrofitted substructure performance. The prototype columns and foundations were formulated by compiling design plans from the 1950's and 1960's for bridges in Washington State. The prototype parameters were selected to be representative of the reviewed designs, without necessarily representing any specific bridge, and to reveal potential undesirable failure modes within the substructures.

Experimental tests were conducted on 1/3-scale specimens that modeled the prototype dimensions, reinforcing ratios and arrangement, deficient detailing, and material properties. Test parameters included evaluating the performance of as-built specimens, methods for improving the footing shear strength, and methods for increasing footing overturning resistance. The specimen columns incorporated reinforcement lap splices at their base and thus required retrofitting. The columns of the specimens were retrofitted using circular steel jacketing in order to focus any distress into the foundations.

The test specimens were supported either directly on a sandy soil, for the spread footing specimens, or on short wood piles in the sandy soil, for the pile-supported specimens. The overall test setup for a single column specimen is shown in Figure 1. The specimens were subjected to reversed cyclic lateral loading under a constant axial load. A ram mounted on a low-friction trolley was used to apply the axial load. Lateral loads were applied using a horizontal actuator.

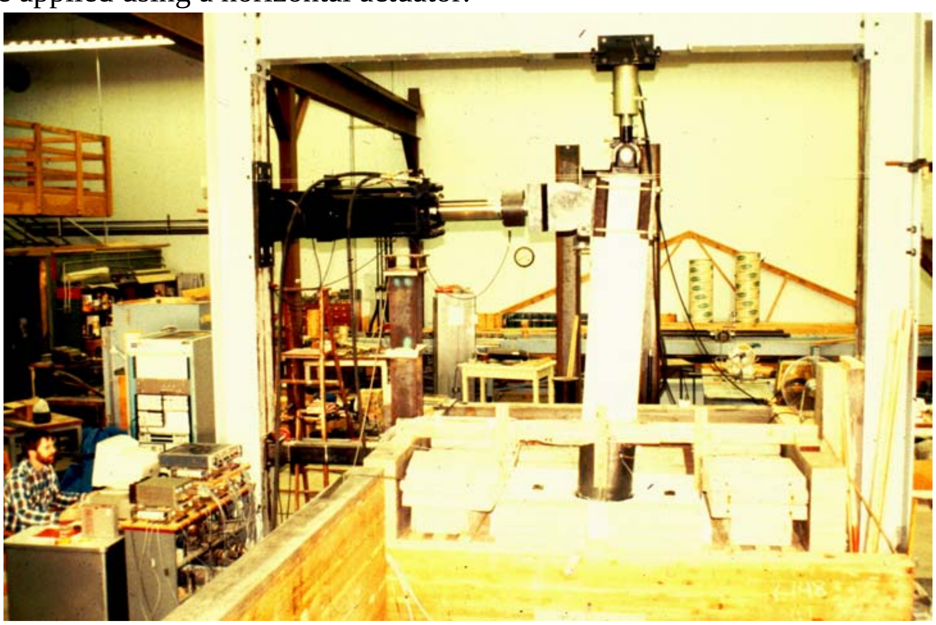

Figure 1 Test Setup

Tests on As-built Specimens

As-built specimens were designed to be representative of existing conditions in which the footing is shear critical. The performance of these specimens was intended as the basis for designing and evaluating retrofit methods for the subsequent specimens. Tests were performed on as-built specimens with both spread and pile-supported footings. The columns of the as-built specimens contained a lap splice at their base with a length of 20 bar diameters (20 d_b) and were retrofitted with a steel jacket.

Failure in the as-built specimens occurred after cycling at low displacement levels. The resulting hysteresis curves for an as-built specimen with a pile-supported footing are shown in Figure 2 and indicate little energy dissipation. The column reached only 65% of its moment capacity before the specimen failed. The column showed only minimal signs of cracking. During testing, the top of the pile cap developed cracking radiating outward from the column. After removing the specimen from the testing setup, cracks were also observed on all four sides of the footing. Only minor cracking was observed on the bottom of the footing. The major cracks in the footing are shown in Figure 3. Testing of an as-built specimen with a spread footing produced similar results.

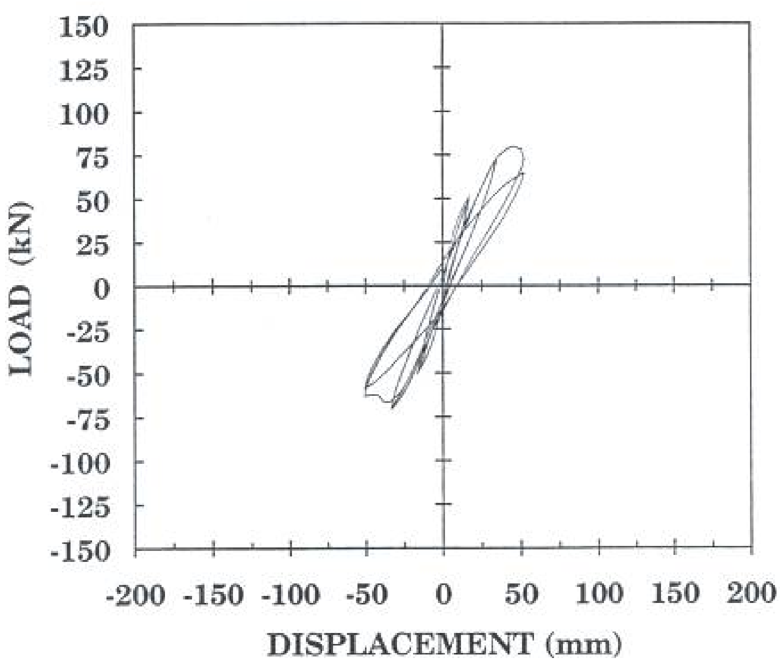

Figure 2 Hytereses Curves For As-Built Specimen

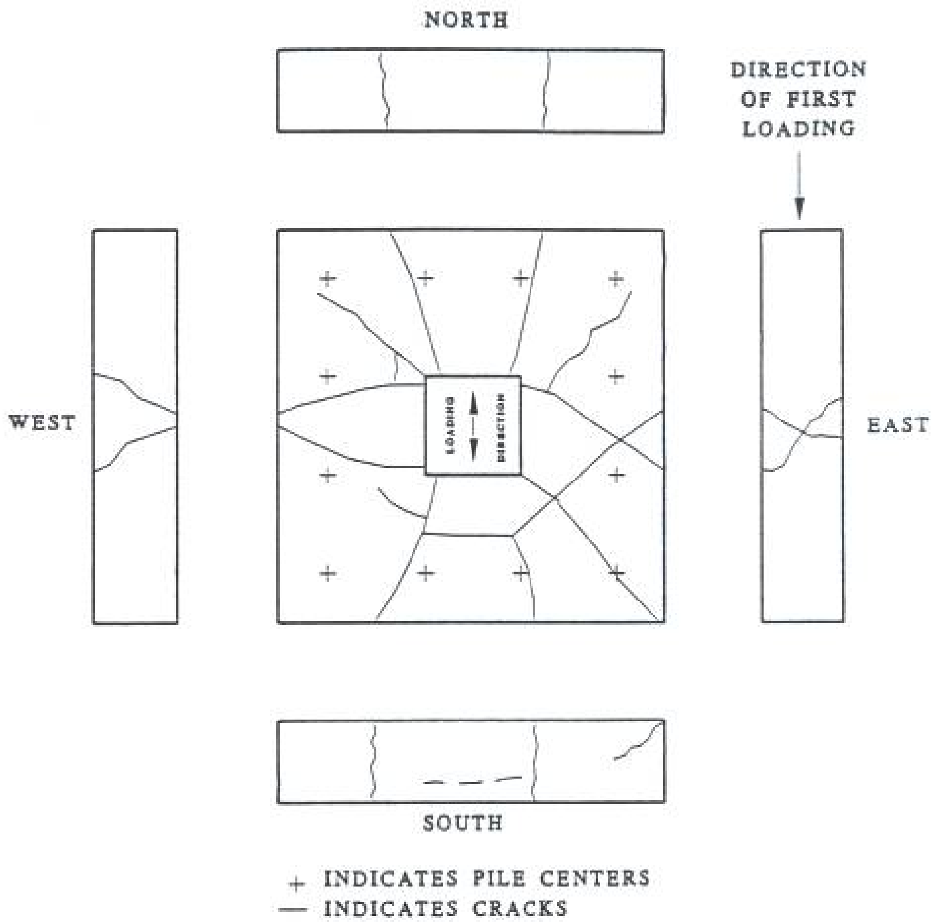

Figure 3 Cracks In As-Built Footing

The cracks observed in the as-built footings are indicative of a shear failure. However, due to the cyclic loading, the exact sequence and the origin of the cracks were difficult to determine, resulting in some uncertainty as to the exact cause of the failure. It was postulated that failure in the footings was a result of one or more of the following failure modes: one-way beam shear, concrete failure associated with pullout of the dowel hooks comprising the column splice, and/or a joint shear failure at the column/footing connection similar to that reported by Xiao, et al (1994, 1996). To gain an understanding of the cause of the failure, a qualitative study was conducted using small-scale specimens (approximately 1/18-scale) which replicated the details of the as-built specimens. The small-scale specimens allowed for cross sectioning of the specimens after testing.

Tests on the small-specimens resulted in the same apparent failure mode observed in the tests on the larger-scale as-built specimens. A cross section, showing the internal cracking patterns within the column/footing joint region, is shown in Figure 4. A major diagonal crack developed within the column/footing connection. In the cross section, loading was applied to the column from right to left. Thus, the inclination of the crack precludes a beam shear failure. Instead, the observed cracking is typical of that associated with a joint shear failure in a beam/column connection. Using the approach suggested by Priestley (1991) for assessing joint shear, maximum principal tensile stress values of approximately $0.46\sqrt{f'_c}$ MPa ($5.5\sqrt{f'_c}$ psi) and $0.43\sqrt{f'_c}$ MPa ($5.2\sqrt{f'_c}$ psi) were calculated for as-built specimens with the pile-supported and spread footings, respectively.

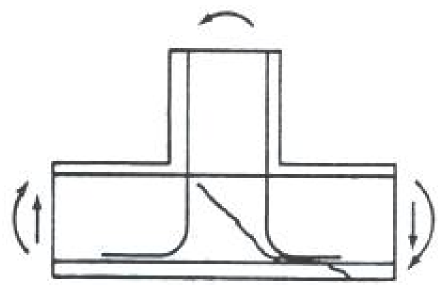

Figure 4 Crack Patterns In Column-Footing Joint Region

Retrofitting for Joint Shear

The as-built specimens were retrofitted to increase the thickness of the pile cap by adding a concrete overlay. The overall thickness of the pile cap was increased by adding a reinforced concrete overlay on top of the existing pile cap. The overlay was designed to act compositely with the existing pile cap by providing dowels. The overlay also allowed for the addition of a mat of horizontal reinforcement, thus providing negative moment strength to the footing. The amount of top reinforcement added was equivalent to that present in the bottom of the existing footing, and a check was made to ensure that this would be sufficient to develop the column flexural strength without yielding of the reinforcement in the footing. The thickness of the overlay was selected to produce joint shear stresses below the tensile stress limit proposed by Priestley (1991) and to allow for development of the shear friction dowels.

The presence of a 20 d_b lap splice required special detailing since the overlay intersected the splice. Since the working interface for the column hinging is at the top of the overlay, the embedment of the splice would no longer be 20 d_b. As a consequence, the column reinforcement may not fully develop and the splice may degrade, no matter the amount of confinement provided. Thus, a pedestal extending to the top of the splice was incorporated into the retrofit scheme to maintain the integrity of the splice. The column retrofit jacket was still required to provide confinement in the new plastic hinge region, now located at the top of the pedestal, due to the inadequate transverse reinforcement present in the as-built column. Figure 5 illustrates the retrofit details.

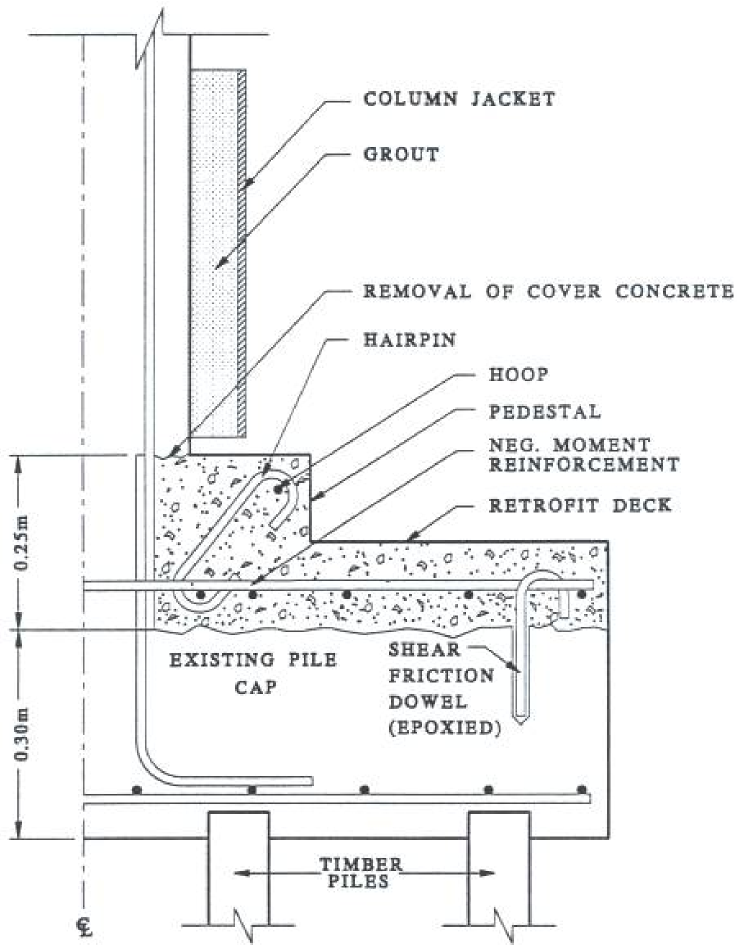

Figure 5 Retrofit Details With 20 d_b Lap Splice

For a lap splice length of 35 d_b, the use of a pedestal to fully contain the splice would result in an unreasonably large pedestal. The overlay thickness was chosen based on joint shear considerations and allowed for a 25 d_b lap splice above the overlay. Previous research (Chai, et al, 1991) has shown that a lap splice length of 20 d_b can fully develop the reinforcement if proper confinement is present. Therefore, no pedestal was used in the retrofit. In order to maintain the original column strength and stiffness, the column longitudinal bars were cut at the top of the overlay prior to pouring the retrofit. Figure 6 illustrates the retrofit details.

Failure in the retrofitted specimens occurred after cycling to large displacement levels. The resulting hysteresis curves for a retrofitted specimen are shown in Figure 7. A plastic hinge developed at the base of the column resulting in a very ductile response. The hysteresis curves are large, have little pinching and exhibit good energy dissipation. Cracking in the pile cap and added overlay was minimal. Footing movements and rotations were very small.

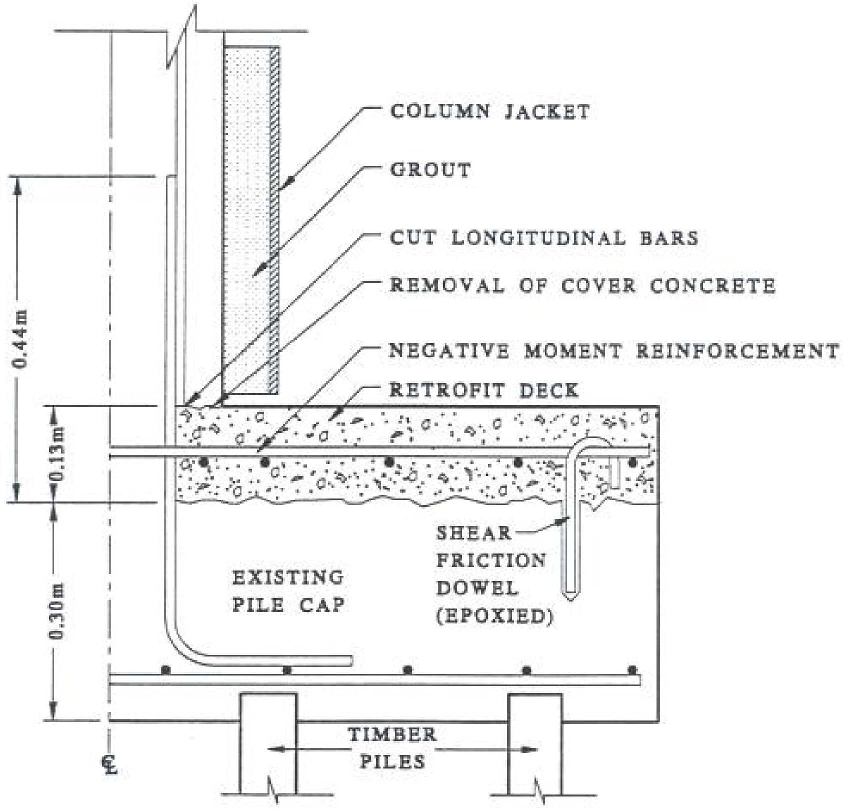

Figure 6 Retrofit Details With 35 d_b Lap Splice

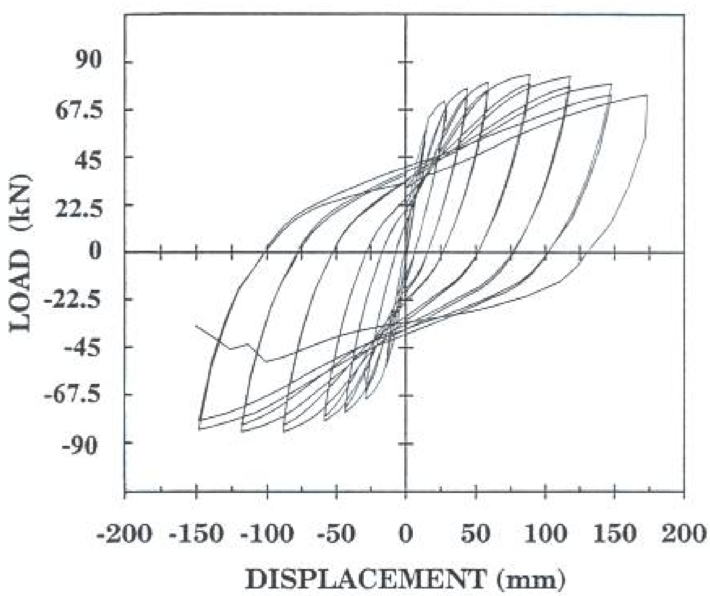

Figure 7 Hytereses Curves For A Retrofitted Specimen

Retrofitting to Increase Overturning Resistance

Several different retrofit methods were evaluated for effectiveness in increasing the overturning resistance of the footings. These methods included enlarging the footing, adding additional piles, and incorporating anchors through the footing to increase the overturning resistance.

Composite action between the existing and the enlarged sections of the footings was achieved by chipping out the concrete around the bottom mat of reinforcement in the existing footing and welding the existing and new positive reinforcement together. The top mat of reinforcement provided in the overlay also enhanced composite action between the sections. Shear reinforcement was provided in the enlarged portion of the footing.

Hysteresis curves for a retrofitted specimen with an enlarged footing are shown in Figure 8. The hysteresis curves are large and exhibit good energy dissipation. In the specimens with additional piles or soil anchors, uplift of the specimen was negligible. In the specimen with the enlarged footing size only, some uplift did occur. However, with all three retrofit methods, specimen response was ductile, with failure resulting from eventual low-cycle fatigue fracture of the longitudinal bars during cycling to large displacement levels.

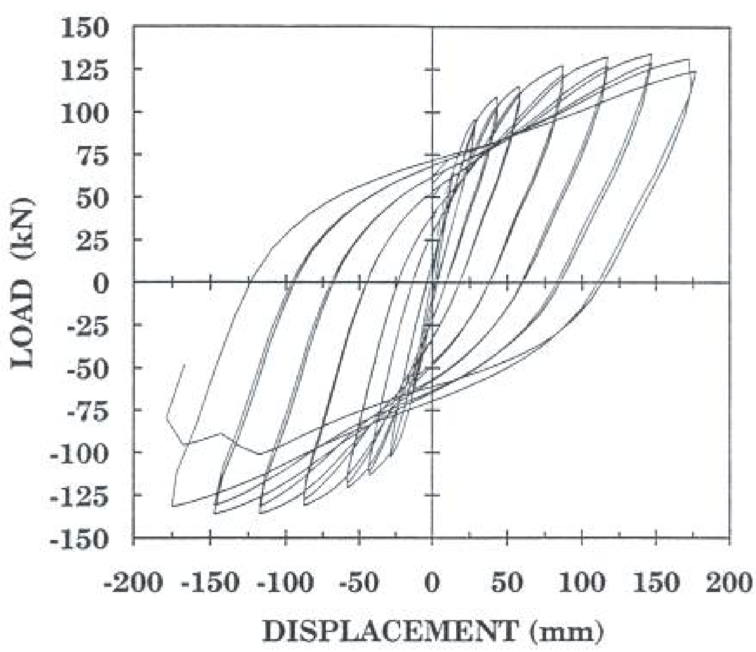

Figure 8 Hytereses Curves For Specimen Retrofitted To Increase Overturning Resistance

Link Beam Retrofit

A three-column bent specimen was constructed and detailed similarly to the single-column as-built specimens. Based on Priestley, et al (1996), a link beam was added to the bent just above the footings as an alternative to retrofitting the footings. The cross-section and amount of reinforcement in the beam were chosen such that the link beam would be stiff compared to the columns. Additional longitudinal reinforcement was used to resist any direct tension in the beam. Closely-space shear reinforcement was provided in the beam, and special joint reinforcement was placed in the joint region of the beam. The cover concrete of the columns was removed to provide a positive connection between the columns and the link beam. Also, a gap was provided between the top of the footings and the bottom of the beam in order to reduce the moment transferred into the footings. No jacketing was applied to the columns.

A picture of link-beam retrofitted specimen at the end of testing is shown in Figure 9. The system load-displacement hysteresis curves are shown in Figure 10. The addition of the link beam was successful in preventing damage and rotation in the footings. Plastic hinging occurred at both the top and bottom of each column. Bar buckling and spalling in the plastic hinge regions began after cycling to moderate displacement levels, with rapid degradation soon after. Some minor cracking occurred in the link beam around the column/beam joint.

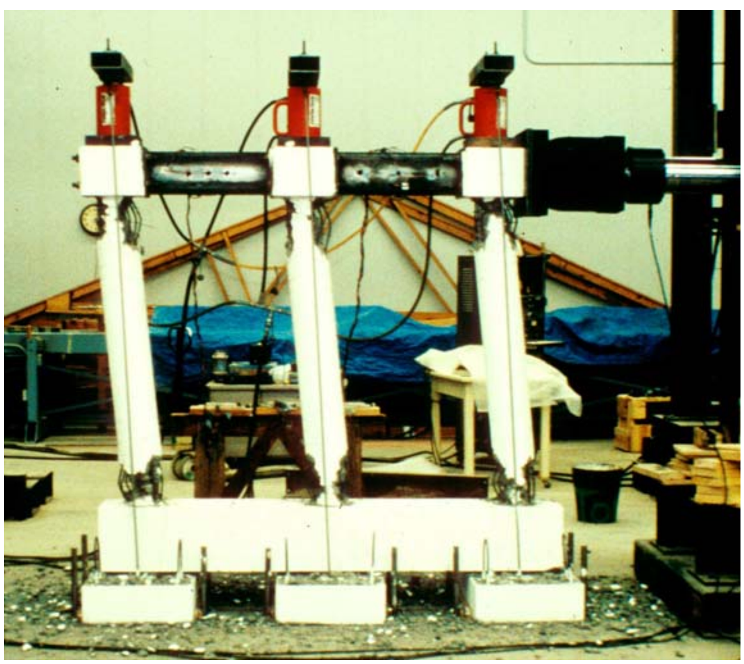

Figure 9 Link-Beam Retrofit Specimen Following Testing

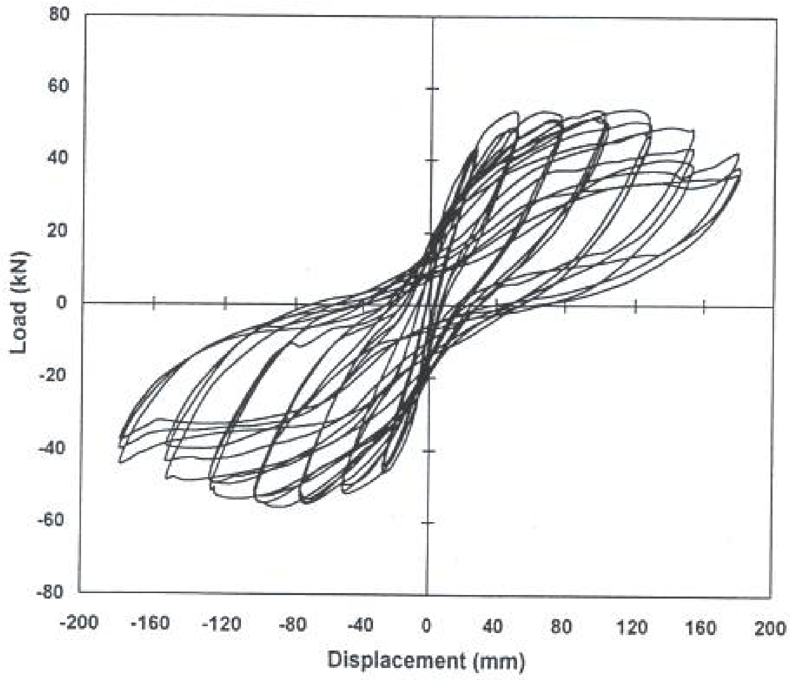

Figure 10 Hytereses Curves For Link-Beam Retrofit Specimen

Conclusions

The experimental test results of this study indicate that bridge foundations, whether spread footing or pile cap, that are not designed to transfer the full column hinging forces, may perform poorly under seismic loading. The as-built specimens of this study exhibited significant cracking in the footings and failed as a result of inadequate joint shear strength in the column/footing connection. The failure was relatively brittle and with little energy dissipation.

It was found that an added reinforced concrete overlay provided an effective retrofit for the as-built footings. The overlay resulted in increased shear resistance, allowed for the addition of a top mat of reinforcement to provide negative moment strength, and increased the positive moment capacity by increasing the effective depth of the footings. All retrofitted specimens developed plastic hinging in the columns with a resulting ductile response under the simulated seismic loading.

Special detailing was required in the column lap splice regions in order to maintain the integrity of the splices. With a 20 d_b splice, a pedestal enclosing the full height of the splice was incorporated into the retrofit. With a 35 d_b splice, no pedestal was used; however, the column bars were cut at the top of the overlay and a remaining confined splice length of at least 20 d_b was maintained.

In the specimens that were overturning critical, increased overturning resistance was provided by enlarging the footing plan size, providing additional piles, and/or providing footing tie-downs.

The addition of a stiff link beam just above the footings was found to be effective in preventing damage in the footings during testing, and a reasonably ductile bent response was achieved. Because the link beam retrofit may not require retrofitting of the footings, this strategy may be the most cost-effective and therefore optimal approach for retrofitting multi-column bents.

Acknowledgments

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