Seismic Retrofitting of Bridge Substructures

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Retrofitting measures for improving the seismic performance of the substructures of existing bridges were investigated. Experimental tests were conducted on 1/3-scale specimens consisting of a square column supported on a pile footing. Details of the column and footing were selected to represent deficiencies present in older bridges. Retrofit measures were applied to both the columns and footings. The specimens were subjected to increasing levels of cycled inelastic lateral displacements under constant axial load. Specimen performance was evaluated on the basis of load capacity, displacement ductility, strength degradation, and hysteretic behavior. Tests on the as-built specimen resulted in a brittle failure due to insufficient joint shear strength in the column and footing connection. 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 pile cap. 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. In a specimen that was overturning critical, successful retrofitting was achieved by enlarging the footing plan size and providing additional piles.

Bridge structures have historically been vulnerable to seismic loading, with numerous examples of damage occurring to both superstructure and substructure elements and, in some cases, complete and catastrophic collapse. The watershed event in changing seismic design philosophies was the 1971 San Fernando earthquake. Bridges built under design criteria developed after 1971 have generally performed well in recent earthquakes. However, the vulnerability of older, pre-1971 bridges was clearly evident in the 1987 Whittier Narrows and 1989 Loma Prieta earthquakes. In the Loma Prieta earthquake alone, damage to bridges resulted in more than 40 deaths, $1.8 billion in damage to transportation structures, and severe economic disruptions due to the loss of major transportation routes (*1*).

As a result of the damage that occurred to older bridges, a major research effort was directed at developing strengthening or retrofit strategies to upgrade the performance of older bridges. Significant retrofit efforts began in California in the 1970s, with the initial focus of the retrofit schemes being to improve the performance of the superstructures in earthquakes. Following the 1987 Whittier Narrows earthquake in which extensive damage occurred to many columns, it became apparent that retrofit efforts must address the entire bridge structure. Column retrofit strategies were subsequently developed. Only recently have strengthening methods been developed for improving the performance of existing footings, and very limited testing has been performed to verify the methods.

The objective of this study was to experimentally evaluate retrofit methods for improving the seismic performance of existing footings. The focus was on pile-supported substructures. A detailed account of the research program can be found elsewhere (*2*). This paper presents an overview of the study and discussion of the test results and conclusions.

BRIDGE SUBSTRUCTURE RETROFITTING

Column Retrofitting

A common detail found in older bridge columns is an insufficient amount of transverse reinforcement. Typically, No. 3 or No. 4 hoops at 0.3 m (12 in.) on center were used in columns regardless of the column cross-sectional dimensions, and the hoops had short extensions and anchorage only by lapping the ends in the cover concrete. Further, intermediate ties were rarely used. This detail results in the susceptibility of many older columns to shear failures, and it provides little confinement for developing the full flexural capacity or preventing buckling of the longitudinal reinforcement.

Another detail commonly used in older bridges was splicing of the longitudinal bars at the bottom of the columns. Typically, starter bars were extended only 20 longitudinal bar diameters (d_b) from the foundations, which does not provide sufficient length to develop the yield strength of the reinforcement. Bond failure is also likely once the cover concrete spalls. These deficiencies result in a high potential for flexural strength degradation in the event of an earthquake.

Previous research (*3*) has shown that the most effective column retrofit method for both circular and rectangular columns is steel jacketing. The steel jacket is made slightly larger than the columns, and the space between the jacket and column is filled with grout. Research has shown that in order to achieve the needed lateral confinement with the retrofit, circular or elliptical jacketing is necessary. Test results showed that jacketing of the columns can improve the splice region performance (partial-height jacketing) and column shear performance (full-height jacketing).

On the basis of recent research studies (*4,3*), the California Department of Transportation (Caltrans) (*5*) implemented standardized column retrofit procedures: the Class P retrofit and the Class F retrofit. Steel jackets with a minimum thickness of 10 mm (3/8 in.) are used. Circular or elliptical jackets are used depending on whether the column is circular or rectangular. The Class P retrofit provides partial confinement in the plastic hinging region, with the intent of providing a pseudo pin at the bottom of the column. The Class F retrofit results in the preservation of the full flexural capacity of the column and typically requires retrofitting of the footing in order to carry the forces transferred from the column.

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Footing Retrofitting

Footings in older bridges were designed primarily for gravity loads. As a result, the footings often contain little or no top reinforcement and may be susceptible to brittle flexural failures in an earthquake. Older footings may also be susceptible to shear failures, both through the footings and at the column and footing joints. Many existing footings are vulnerable to overturning, pile failures, or both. All of these problems may be exacerbated by retrofit measures applied to other sections of a bridge, such as column jacketing.

Caltrans (*5*) developed procedures for designing footing retrofits. Based on the plastic moment capacity of the columns, the footing is checked for flexural and shear strengths and overturning. To increase overturning resistance, the footing may be enlarged, additional piles provided, or soil anchors added. To provide negative moment strength and to increase shear strength, a concrete overlay is added to the top of the existing footing. Horizontal reinforcement is incorporated into the overlay, and reinforcing dowels connect the overlay to the existing footing.

Xiao et al. (*6*) tested specimens with as-built and retrofitted footings. Tests on the as-built specimen resulted in a column and footing joint shear failure. Retrofitted specimens incorporating an overlay designed using current Caltrans standards performed better, but the researchers concluded that the standards do not adequately address the joint shear problem. An improved retrofit design using longer dowels to develop more effective joint shear resistance mechanisms was proposed and verified.

EXPERIMENTAL TESTING PROGRAM

Test Specimens and Parameters

For this study a section of a typical bridge substructure consisting of a single column and supporting pile footing was used as the basis for evaluating as-built and retrofitted substructure performance. The prototype column and pile footing were formulated by compiling design plans from the 1950s and 1960s for bridges in Washington State. Emphasis was placed on single-column bent bridges because these bridges are likely to be more critical than multicolumn bent bridges and thus would be the first type of bridges targeted for retrofit. The prototype substructure section chosen for study consisted of a 3.7- by 3.7-m (12- by 12-ft) square pile cap with a thickness of 0.9 m (3 ft) and a 0.9-m (3-ft) square column. The reinforcing ratios selected for the pile cap were 0.42 and 0.28 percent for the longitudinal and transverse steel, respectively, and the column reinforcing ratio selected was 2.5 percent. Details included column lap splice lengths of $20d_b$ and $35d_b$. Timber piles were selected for study in this investigation because they are common in many older foundations in Washington State. On the basis of the reviewed plans, the timber piles were typically spaced at 0.9-m (3-ft) intervals and were approximately 0.3 m (12 in.) in diameter.

The 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 the performance of as-built specimens and methods for improving the pile-cap shear strength and for increasing footing overturning resistance. The specimen columns incorporated both $20d_b$ and $35d_b$ splices. The columns of all specimens were retrofitted using circular steel jacketing in order to focus any distress into the footings. A summary of the test specimens is given in Table 1; five specimens were tested. Details of Specimen No. 1, representing as-built footing details, are shown in Figure 1. The various retrofit measures applied to the remaining specimens are discussed later along with the test results.

Test Setup and Procedures

The test specimens were supported on short wood piles in a sandy soil contained within a stiff box constructed of large glue-laminated wood beams. Soil was compacted between the piles before con-

TABLE 1 Summary of Test Specimens

Specimen No.	Pile-Cap Deficiency	Pile-Cap Retrofit Applied	Column Type	Column Deficiency	Axial Load $\frac{P}{f'_c A_g}$
1	Shear	None	Square	20 d_b hinge splice	0.104
2	Shear	Top Deck and Pedestal	Square	20 d_b hinge splice	0.104
3	Shear	Top Deck	Square	35 d_b hinge splice	0.104
4	Shear and Overturning	Top Deck and Low Tension Capacity Piles	Square	35 d_b hinge splice	0.069
5	Shear and Overturning	Top Deck, Low Tension Capacity Piles, and Added Piles	Square	35 d_b hinge splice	0.069

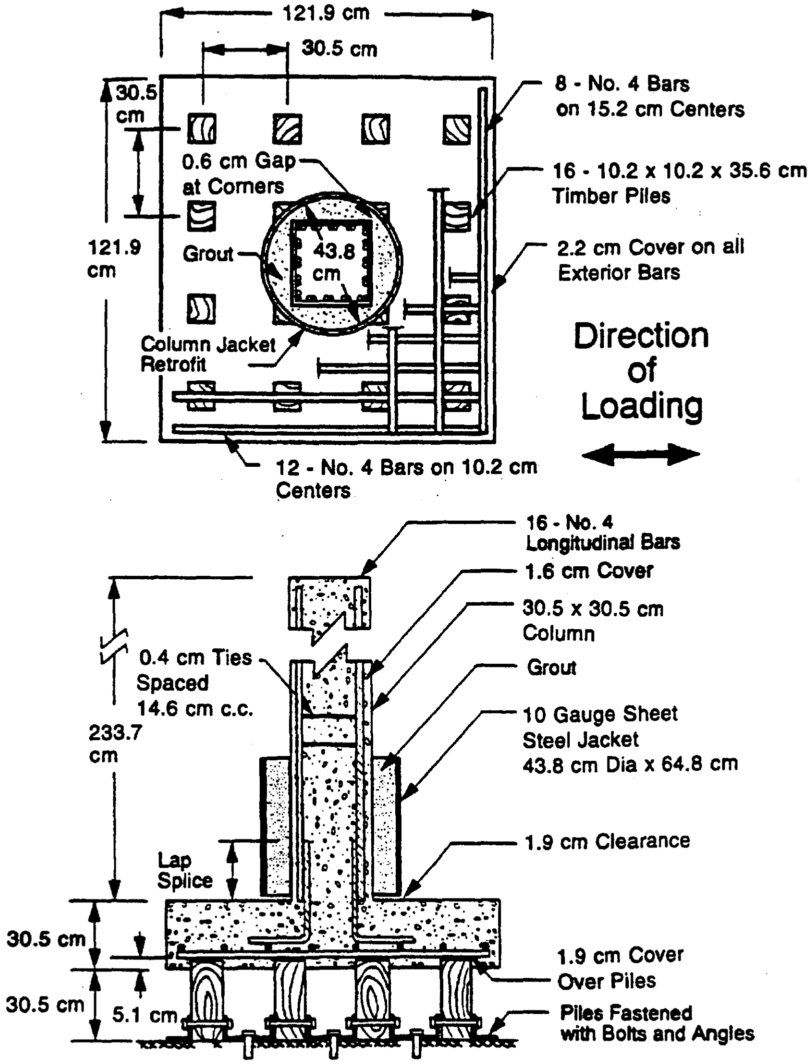


FIGURE 1 Details of specimen representing as-built conditions, Specimen No. 1 (1 in. = 2.5 cm).

struction of the pile cap and then around the pile cap after it was poured, as shown in Figure 2. The objectives of this test setup were to allow the pile cap to rotate and to approximately simulate the actual footing support conditions. This setup overconfines the soil when compared with field conditions, and there is significant labor involved in setting up and removing a specimen. However, the setup is more realistic than the support conditions often used in laboratory tests in which the footing is bolted to a strong floor, thus not allowing any footing rotations.

The overall test setup is shown in Figure 3. The specimens were subjected to reversed cyclic lateral loading under a constant axial load. Axial loads of 270 kN (60 kips) and 180 kN (40 kips) were used to facilitate the study of various failure mechanisms. A ram mounted on a low-friction trolley was used to apply the axial load. Lateral loads were applied using a horizontal actuator.

The determination of the column tip horizontal displacement at first yield (Δ_y) and the loading sequence were similar to the procedures used by Priestley and Park (*7*). The specimens were subjected to a simulated seismic loading pattern consisting of increasing multiples of Δ_y in order to demonstrate the ductility and hysteretic behavior of the test specimens. The loading pattern for the specimens consisted of two cycles at displacement levels of ±1, ±2, ±3, ±4, ±6, ±8, ±10, and ±12 times Δ_y unless failure occurred first.

Strain gauges were used to monitor the strains in the flexural and transverse reinforcement. Linear variable displacement transformers (LVDTs) and load cells measured column displacements and applied loads. LVDTs were also placed on the top of the pile cap to determine footing displacements and rotations. Several of the wood piles were instrumented with strain gauges and were calibrated under compressive loading in an attempt to monitor loads in the piles. All data were recorded intermittently during testing.

TEST RESULTS AND DISCUSSION

In this section results of the experimental tests are summarized. Results from Specimen No. 1 are presented first. These results were used to formulate the retrofits for the four subsequent specimens. A

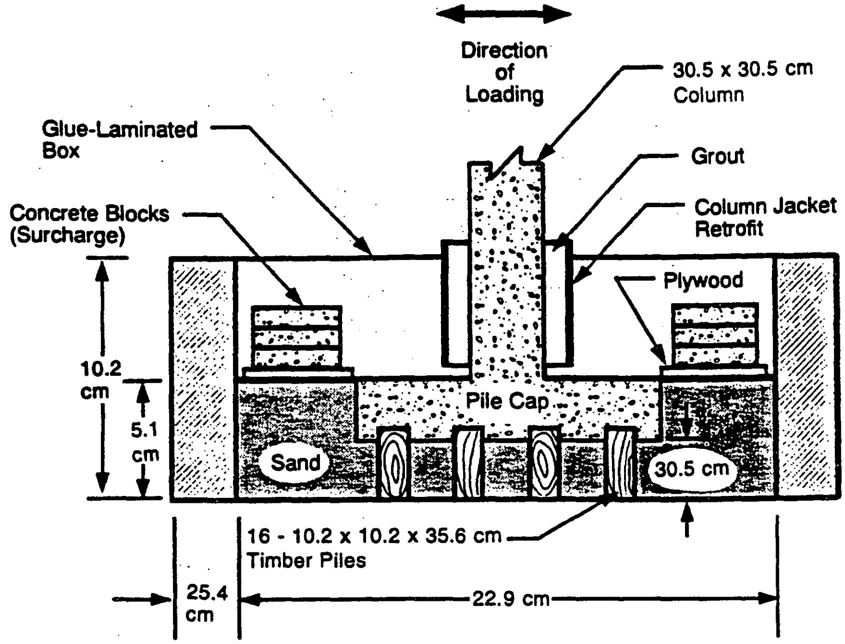


FIGURE 2 Testing support conditions (1 in. = 2.5 cm).

description of the retrofit methods applied and specimen performance is then presented for Specimen Nos. 2 through 5. Specimen performance was evaluated on the basis of moment capacity, displacement ductility, strength degradation, and hysteretic behavior.

Specimen No. 1

Specimen No. 1 was designed to be representative of as-built conditions in which the pile cap is shear critical. The performance of this specimen was intended as the basis for designing and evaluating retrofit methods for the subsequent specimens. The column of Specimen No. 1 contained a $20d_b$ lap splice and was retrofitted at the base with a steel jacket.

General Behavior

Failure in Specimen No. 1 occurred during loading to a displacement level of $2\Delta_y$. The resulting hysteresis curves for Specimen No. 1 are shown in Figure 4 and indicate little energy dissipation. The peak applied lateral load was 49.8 kN (11.2 kips) and occurred at a column tip displacement of 36.6 mm (1.44 in.). The column reached 65 percent of its moment capacity before the specimen failed and showed only minimal signs of cracking.

During testing, the top of the pile cap developed cracking radiating outward from the column. After the specimen was removed from the testing setup, cracks were also observed on all four sides of the pile cap. Only minor cracking was observed on the bottom of the pile cap. The major cracks occurring in Specimen No. 1 are shown in Figure 5.

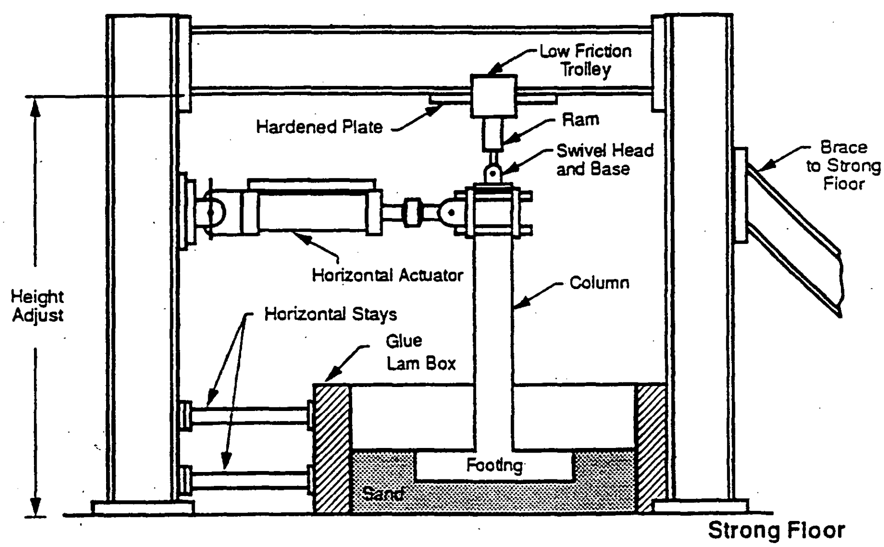


FIGURE 3 Testing setup (1 kip = 4.448 kN; 1 in. = 25.4 mm; 1 ft = 0.304 m).

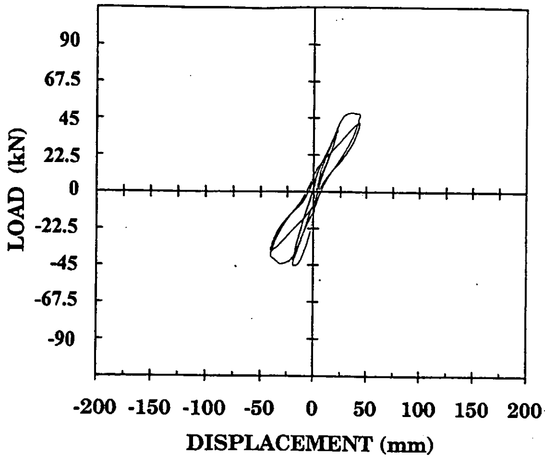


FIGURE 4 **Load-deflection curves for Specimen No. 1 (1 kN = 0.2248 kip; 1 mm = 0.039 in.).**

Failure Mechanism

The cracks observed in the pile cap of Specimen No. 1 are indicative of a shear failure. However, because of 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 pile cap 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 forming the column splice, or a joint shear failure at the column and footing connection similar to that reported by Xiao et al. (*6*). To gain an understanding of the cause of the failure, a qualitative study (*8*) was conducted using small-scale specimens that replicated the details of Specimen No. 1. The small-scale specimens (approximately 1/18 scale) allowed for cross sectioning of the specimens after testing.

Tests on the small specimens resulted in the same apparent failure mode observed in the test on the larger-scale Specimen No. 1. A cross section showing the internal cracking patterns within the column and footing joint region is shown in Figure 6(*a*). A major diagonal crack developed within the column and footing connection. In Figure 6, 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 or column connection [see Figure 6(*b*)].

Priestley (*9*) has suggested a simple method of checking principal tensile stress in the column and footing joint region to assess joint shear failure. The principal tensile stress in the joint region is calculated using Mohr's circle for stress and accounting for the axial and shear stresses within the joint. Details of the procedure are given by Xiao et al.(*6*). A tensile stress value of 0.42 $\sqrt{f_c'}$ MPa

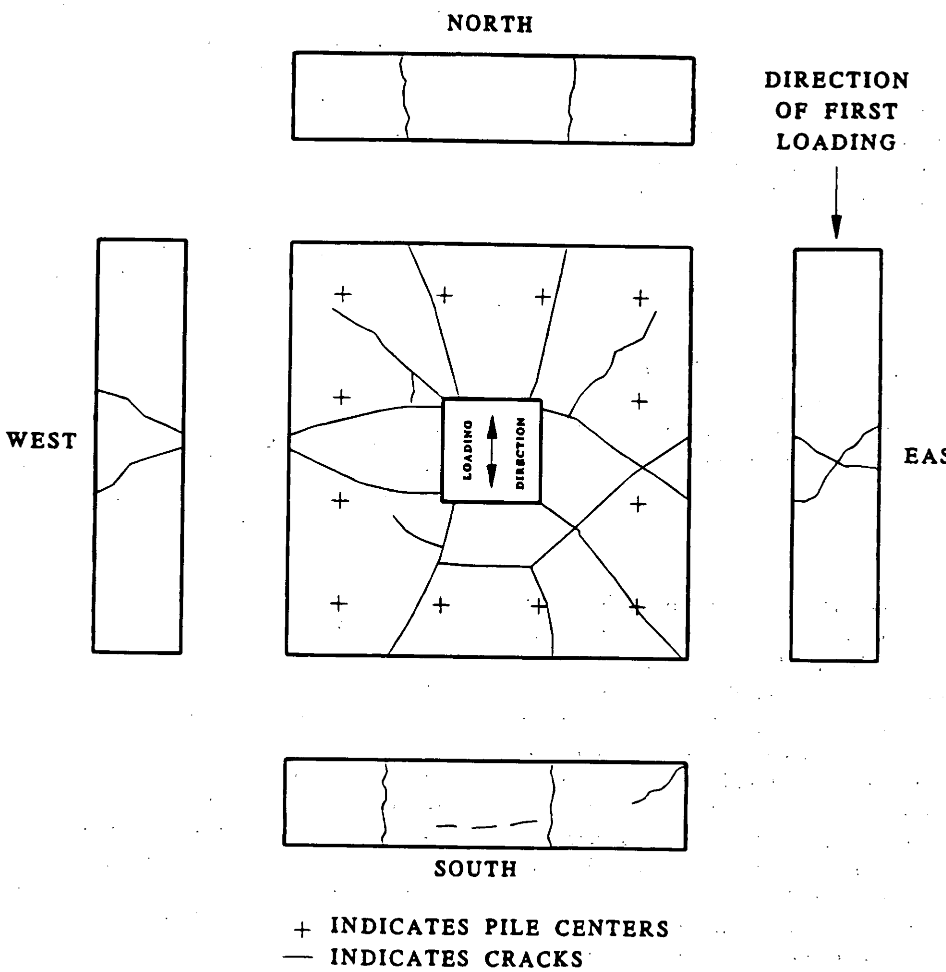
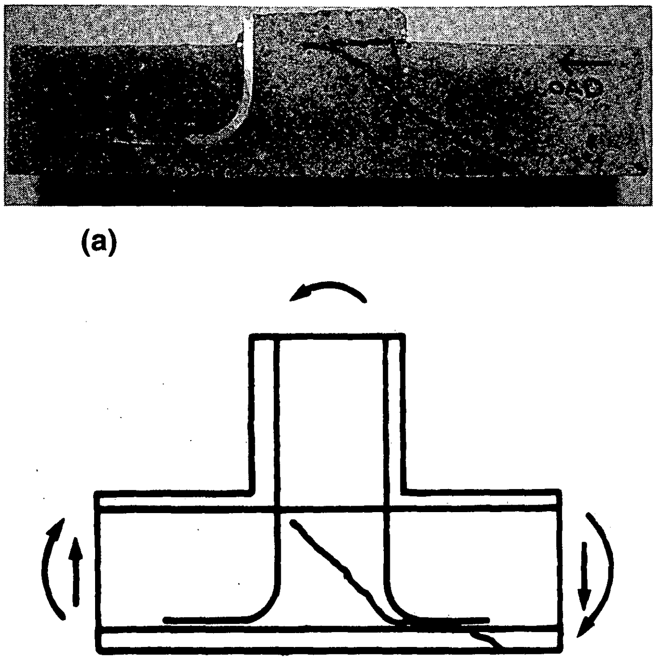


FIGURE 5 **Cracking patterns in pile cap of Specimen No. 1.**



(a)

(b)

FIGURE 6 Cracking patterns in (*a*) small-scale column and footing joint and (*b*) column and beam joint.

(5.0 $\sqrt{f'_c}$ psi) was suggested as the limit at which joint failure will occur. Using this approach, a maximum tensile stress value of approximately 0.46$\sqrt{f'_c}$ MPa (5.5 $\sqrt{f'_c}$ psi) was calculated for Specimen No. 1, reinforcing the conclusion that a joint shear failure in the column and footing connection was the failure mechanism.

Specimen No. 2

Specimen No. 2 was constructed and detailed as in Specimen No. 1 except that the pile cap was retrofitted to increase its thickness by adding a concrete overlay. This overlay intersected the splice region of the column and thus required special detailing.

Retrofit Description

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 dowels were designed using shear friction theory and drilled and epoxied into the top of the existing pile cap. The ends of the dowel were anchored into the retrofit overlay with 180-degree hooks. The overlay also allowed for the addition of a mat of horizontal reinforcement, thus providing negative moment strength to the footing. The thickness of the overlay was selected to produce joint shear stresses below the limit proposed by Priestley (*9*) and to allow for development of the shear friction dowels. An overlay thickness of 13 cm (5 in.) was used in the specimen.

The $20d_b$ splice present in the column of Specimen No. 2 required special detailing since the overlay intersected the splice (in this case, at midheight of the splice). If the splice was intersected by the overlay, the working interface for the column hinging would be at the top of the overlay, and the embedment of the splice would no longer be $20d_b$. As a consequence, the column reinforcement may not fully develop and the splice may degrade, no matter what 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. Crack control steel consisting of a hoop and hairpins was provided in the pedestal. Figure 7 illustrates the details of the retrofit used for Specimen No. 2.

The column cover over the full height of the splice was removed before constructing the retrofit overlay to enable composite action and load transfer between the column and the added overlay. The column retrofit jacket was still required to provide confinement in the new plastic hinge region, now located at the top of the pedestal, because of the inadequate transverse reinforcement present in the as-built column.

Test Results

The specimen performed very well, with failure occurring at a displacement level of $10\Delta_y$, as illustrated by the hysteresis curves shown in Figure 8. The peak applied lateral load was 87.2 kN (19.6 kips) and occurred at a displacement of 118 mm (4.65 in.). During the second cycle of loading to a displacement level of $10\Delta_y$, a column longitudinal bar fractured. Before this low-cycle fatigue fracture of the reinforcement, the development of a plastic hinge at the base of the column resulted in a very ductile response. The hysteresis curves are large, show little pinching, and exhibit good energy dissipation.

Cracking in the pile cap, added overlay, and pedestal was minimal. Some cracking did occur in the pedestal around the column as a result of plastic hinge penetration. Pile cap movements and rotations were very small. On the basis of the instrumented piles, significant pile tension forces were observed despite the lack of any structural connection between the top of the wood piles and the pile cap.

Specimen No. 3

The as-built portion of Specimen No. 3 was detailed and constructed as in Specimen No. 1. However, the column of Specimen No. 3 incorporated a $35d_b$ lap splice rather than the $20d_b$ splice used in Specimen Nos. 1 and 2.

Retrofit Description

A concrete overlay retrofit was again used to improve the performance of the footing. With a lap splice length of $35d_b$, the use of a pedestal to fully contain the splice would result in an unreasonably large pedestal. As in Specimen No. 2, an overlay thickness of 13 cm (5 in.) was chosen on the basis of joint shear considerations. Thus, the overlay would intersect the splice at 13 cm (5 in.) or $10d_b$ from the bottom of the splice, leaving a $25d_b$ lap splice above the overlay. Previous research (*4*) has shown that a lap splice length of $20d_b$ can fully develop the reinforcement if proper confinement is present. Therefore, no pedestal was used in the retrofit. However, in order to maintain the original column strength and stiffness, the column longitudinal bars

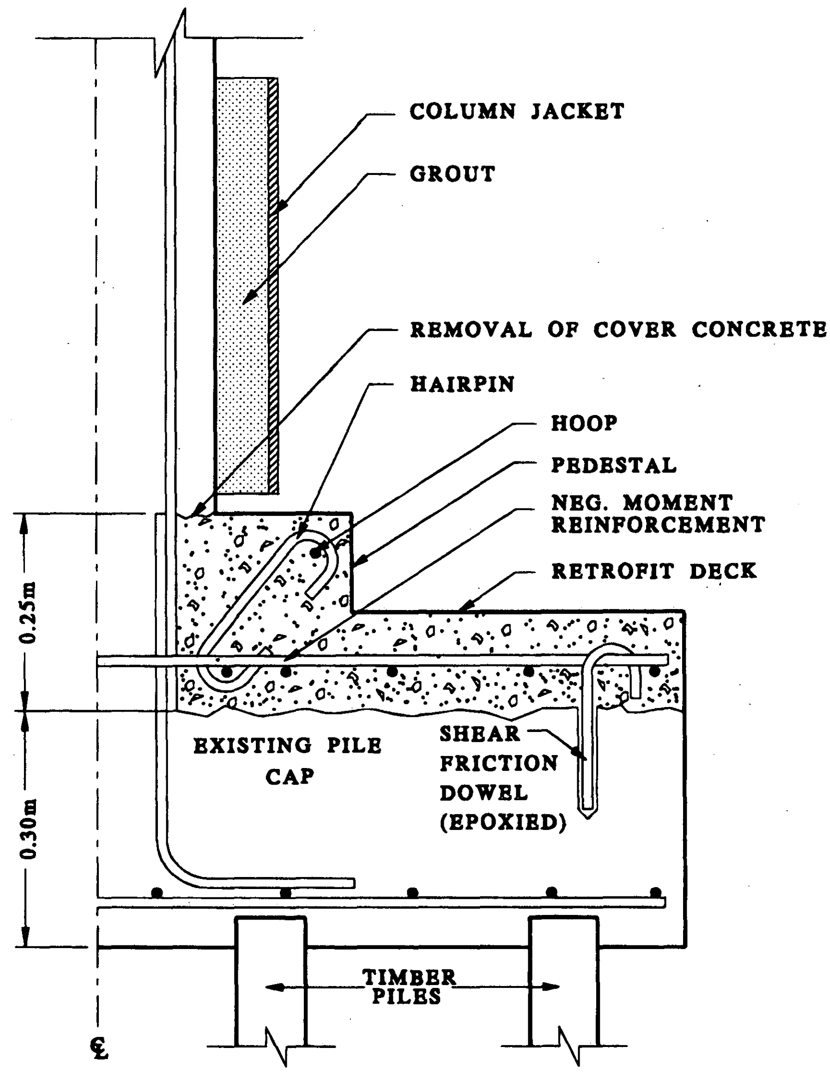


FIGURE 7 Retrofit scheme for Specimen No. 2 (1 m = 3.3 ft).

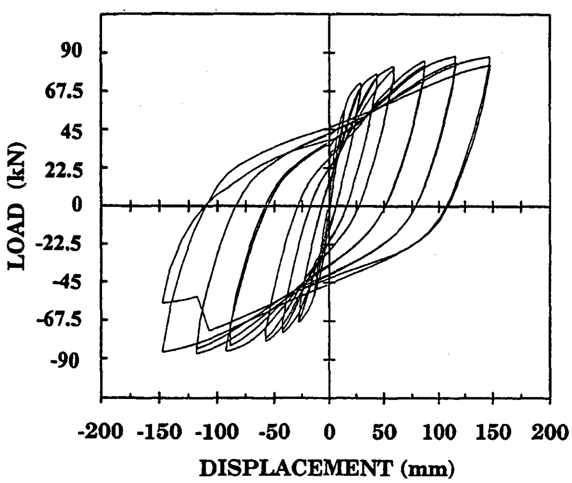


FIGURE 8 Load-deflection curves for Specimen No. 2 (1 kN = 0.2248 kip; 1 mm = 0.039 in.).

were cut at the top of the overlay before the retrofit was poured. All other details of the retrofit were the same as in Specimen No. 2. Figure 9 shows the retrofit measures applied to Specimen No. 3.

Test Results

The hysteresis curves for Specimen No. 3 are shown in Figure 10 and indicate good energy dissipation. The peak applied lateral load was 83.6 kN (18.8 kips) and occurred at a displacement of 90.1 mm (3.55 in.). During the first cycle to a displacement level of $12\Delta_y$, several dowel bars fractured and the test was stopped. Cracking resulting from plastic hinge penetration occurred in the top of the pile cap and was more extensive than the cracking observed in Specimen No. 2. After the specimen was removed from the test setup, some diagonal cracking was also evident in the as-built portion of the pile cap. However, the pile cap maintained its integrity and the overall performance of the specimen was satisfactory.

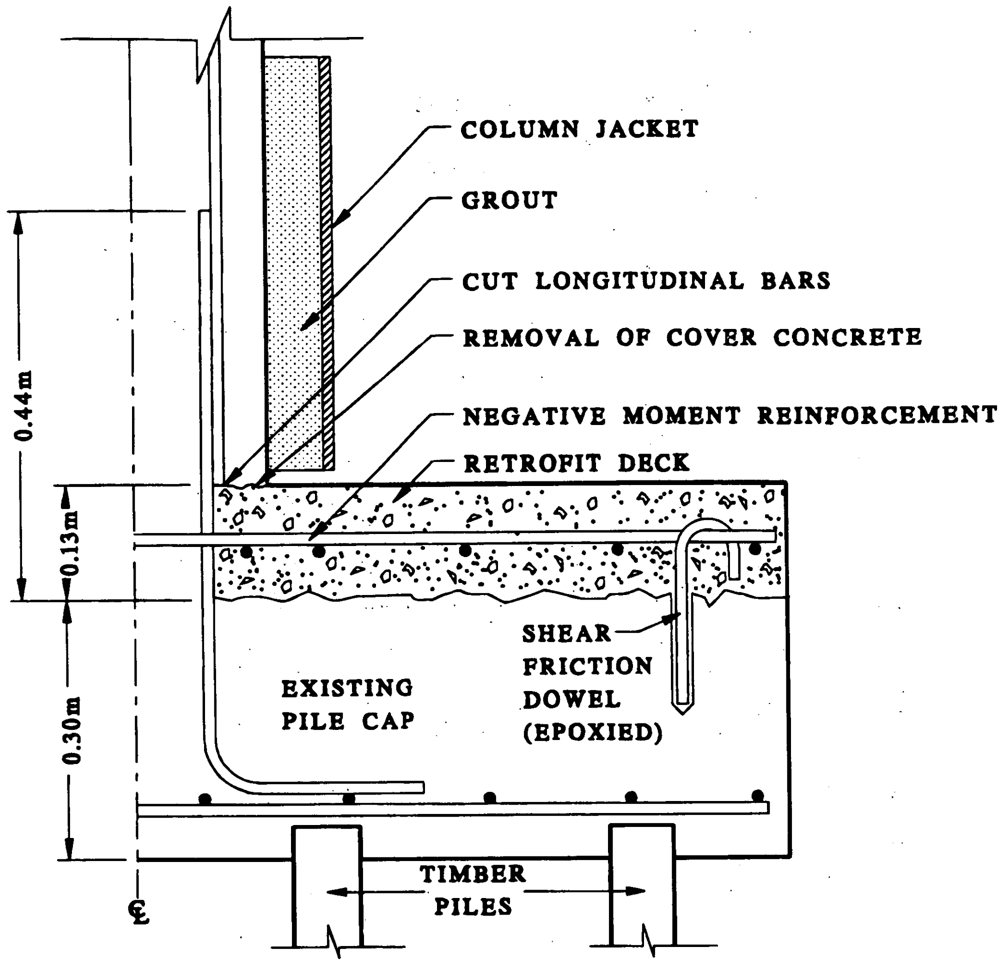


FIGURE 9 Retrofit scheme for Specimen No. 3 (1 m = 3.3 ft).

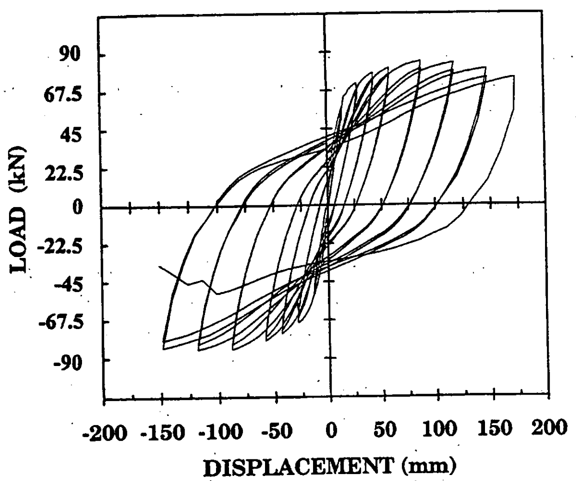


FIGURE 10 Load-deflection curves for Specimen No. 3 (1 kN = 0.2248 kip; 1 mm = 0.039 in.).

Specimen No. 4

Specimen No. 4 was designed to examine the "rocking" behavior of a footing system when the tension capacity of the piles is lost or nonexistent. In the previous tests, the timber piles were found to have a tensile capacity. If this tensile capacity was not present, the overturning resistance of the footing would be less than the column flexural capacity and overturning or rocking would occur. This rocking behavior would be relevant to foundations that, perhaps by choice, were not retrofitted.

Retrofit Description

Specimen No. 3 incorporated a $35d_b$ lap splice, and the details of the retrofit were identical to those used for Specimen No. 3. However, the tops of the piles were greased, and a layer of crushable foam was placed around the sides of the piles embedded in the pile cap. These measures effectively destroyed the tensile capacity of the piles while at the same time preserving the compressive capacity. A reduced axial load was used on Specimen No. 4 to ensure that the specimen would be overturning critical.

Test Results

Figure 11 shows the hysteresis curves for Specimen No. 4. The S shape of the curves is the result of the uplift and rotation of the pile cap. The peak applied lateral load is approximately 67 kN (15 kips) and is only 80 percent of the column capacity. The hysteresis curves enclose small areas, indicating low energy dissipation. However, the response was very stable, indicating the potential for beneficial load redistribution and cost savings if some footings were left unretrofitted and allowed to rock.

Specimen No. 5

The tensile capacity of the piles in Specimen No. 5 was suppressed as in Specimen No. 4. However, Specimen No. 5 was retrofitted by enlarging the footing and adding additional piles to increase the overturning resistance.

Retrofit Description

The footing size for Specimen No. 5 was enlarged by adding 0.3 m (12 in.) to each end in the direction of loading. Eight additional piles were added, four at each end, to increase the overturning resistance. A $35d_b$ lap splice was present in the column, and an overlay was added to increase the shear resistance of the footing. The overlay was detailed in a manner similar to that for Specimen Nos. 3 and 4.

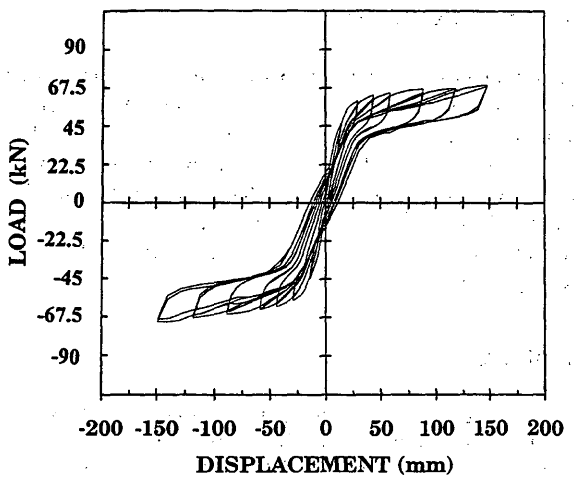


FIGURE 11 Load-deflection curves for Specimen No. 4 (1 kN = 0.2248 kip; 1 mm = 0.039 in.).

The additional piles were selected to represent steel-encased cast-in-place concrete piles. This type of pile was chosen for its tension capability, ease of construction, and the likelihood of its being used in actual retrofits. In the scaled specimen tests, the added piles consisted of concrete cast into steel tubing bolted to the floor. A reinforcing bar was cast into the center of the pile to provide tension capacity between the added piles and the cap.

Composite action between the existing and the enlarged sections of the pile cap 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 pile caps. The retrofit design for Specimen No. 5 is shown in Figure 12.

Test Results

The hysteresis curves for Specimen No. 5 are shown in Figure 13. The pile cap experienced essentially no uplift. The peak applied lateral load was 81.4 kN (18.3 kips) and occurred at a displacement of 90.1 mm (3.55 in.). During cycling to a displacement level of $12\Delta_y$, five of the outermost dowel bars fractured because of low-cycle fatigue and testing was stopped. The hysteresis curves are large and exhibit good energy dissipation.

Similar to the cracking observed in Specimen No. 3, cracks developed in the top of the pile cap and extended toward the sides. Cracks also developed in the top of the pile cap around the column because of plastic hinge penetration. However, the cracking was controlled by the top mat of reinforcement in the retrofit overlay and specimen performance was very satisfactory.

CONCLUSIONS

The experimental test results from this study indicate that existing bridge footings may perform poorly under seismic loading. The as-built specimen exhibited significant cracking in the pile cap and failed as a result of inadequate joint shear strength in the column and footing connection. The failure was relatively brittle and showed 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 pile cap. 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 $20d_b$ splice, a pedestal enclosing the full height of the splice was incorporated into the retrofit. With a $35d_b$ 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 $20d_b$ was maintained.

In a specimen that was overturning critical, successful retrofitting was achieved by enlarging the footing plan size and providing additional piles.

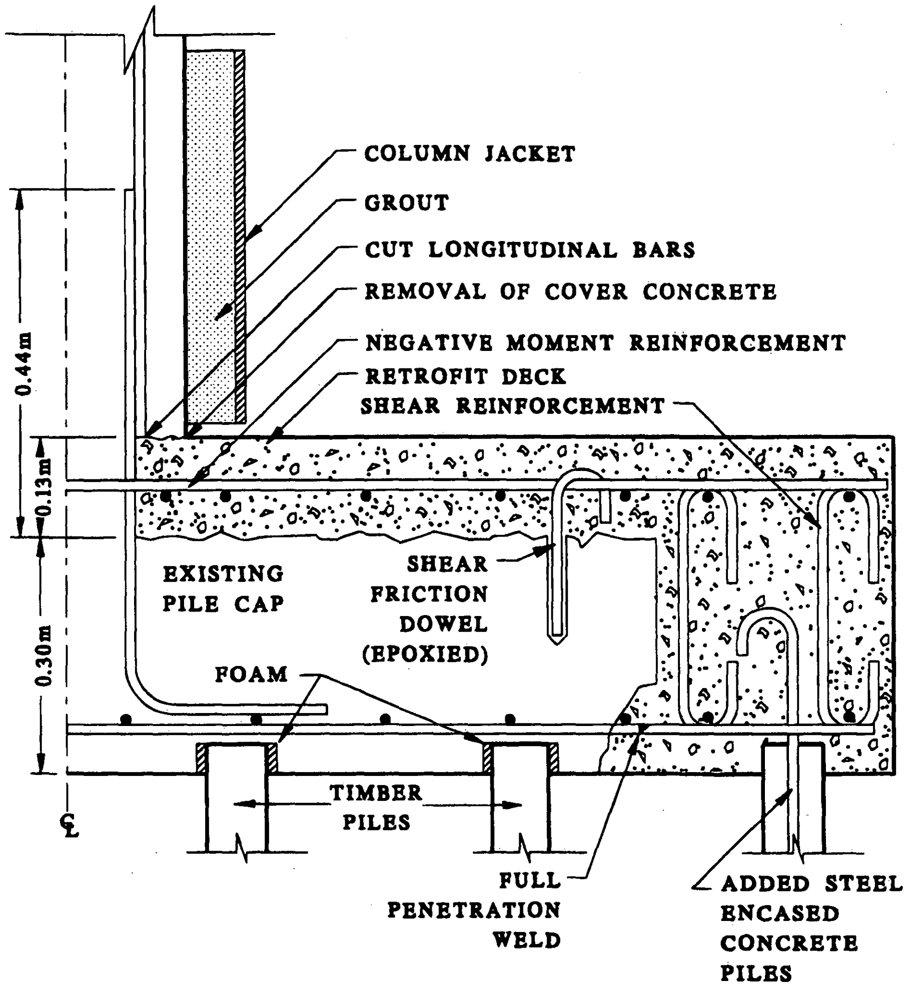


FIGURE 12 Retrofit scheme for Specimen No. 5 (1 m = 3.3 ft).

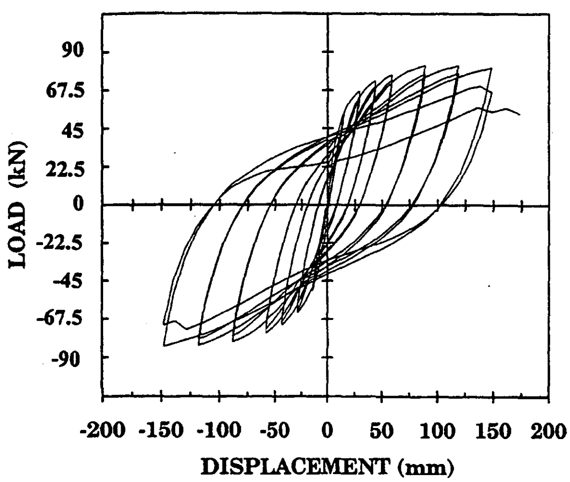


FIGURE 13 Load-deflection curves for Specimen No. 5 (1 kN = 0.2248 kip; 1 mm = 0.039 in.).

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