Research Report

Research Project T9234-10
Bridge Substructure Retrofit

SEISMIC EVALUATION AND RETROFIT OF BRIDGE SUBSTRUCTURES WITH SPREAD AND PILE-SUPPORTED FOUNDATIONS

by

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Prepared for

Washington State Transportation Commission
Department of Transportation
and in cooperation with
U.S. Department of Transportation
Federal Highway Administration

December 1995

TECHNICAL REPORT STANDARD TITLE PAGE

1. REPORT NO.	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO.
WA-RD 382.1		

4. TITLE AND SUBTITLE	5. REPORT DATE
Seismic Evaluation and Retrofit of Bridge Substructures with Spread and Pile-Supported Foundations	December 1995
	6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S)	8. PERFORMING ORGANIZATION REPORT NO.
David I. McLean, Thad D. Saunders and Harold H. Hahnenkratt	
9. PERFORMING ORGANIZATION NAME AND ADDRESS	10. WORK UNIT NO.
Washington State Transportation Center (TRAC) Civil and Environmental Engineering; Sloan Hall, Room 101 Washington State University Pullman, Washington 99164-2910	11. CONTRACT OR GRANT NO. T9234-10
12. SPONSORING AGENCY NAME AND ADDRESS	13. TYPE OF REPORT AND PERIOD COVERED
Washington State Department of Transportation Transportation Building, MS 7370 Olympia, Washington 98504-7370	Research Report
	14. SPONSORING AGENCY CODE

15. SUPPLEMENTARY NOTES

This study was conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

16. ABSTRACT

This study investigated retrofitting measures for improving the seismic performance of the substructures of existing bridges. Retrofit measures for both pile-supported and spread footings were investigated. Experimental tests were conducted on 1/3-scale footing and column assemblages which incorporated details that 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/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 specimens that were overturning critical, increased overturning resistance was provided by enlarging the footing plan size, by providing additional piles, or by providing tie-downs through the footing.

17. KEY WORDS	18. DISTRIBUTION STATEMENT
Key words: Seismic retrofitting, reinforced concrete, bridges, substructures.	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22616

19. SECURITY CLASSIF. (of this report)	20. SECURITY CLASSIF. (of this page)	21. NO. OF PAGES	22. PRICE
None	None	55	

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EXECUTIVE SUMMARY

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INTRODUCTION

INTRODUCTION AND BACKGROUND

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, the 1989 Loma Prieta, and the 1994 Northridge 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 (Cooper, et al, 1994).

As a result of the damage that occurred to older bridges, major research efforts were directed at developing strengthening or retrofit strategies to upgrade the performance of older bridges. Significant retrofit efforts began in California in the 1970's, 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.

This research report presents the main results and conclusions from a study investigating retrofit methods for improving the seismic performance of existing bridge substructures. Retrofit measures for both pile-supported and spread footings were investigated. Experimental tests were conducted on 1/3-scale footing and column assemblages which were subjected to reversed cyclic inelastic deformations representative of earthquake loadings. The test specimens incorporated both footing and column deficiencies, and the retrofit methods were evaluated in terms of benefits and feasibility.

RESEARCH OBJECTIVES

The objectives of this study were as follows:

1. to identify typical seismic deficiencies in bridge substructures of existing bridges in Washington State;
2. to identify and assess potential substructure retrofit methods resulting from recent and ongoing research efforts;
3. to experimentally evaluate the effectiveness of the identified retrofit methods for improving the flexural and shear strengths and overturning resistance of both pile-supported and spread foundations;
4. to evaluate possible adverse interaction effects resulting from retrofitting the various components of the substructure; and
5. to draw conclusions and make recommendations for practical methods of evaluating and improving the seismic performance of substructures in existing bridges.

PREVIOUS RESEARCH AND CURRENT PRACTICE

SUBSTRUCTURE DEFICIENCIES

An example of a bridge substructure from a pre-1971 design is shown in Figure 1. Based on current understanding of seismic performance, a number of deficiencies can be identified. 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 many older columns being susceptible 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.

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 through the

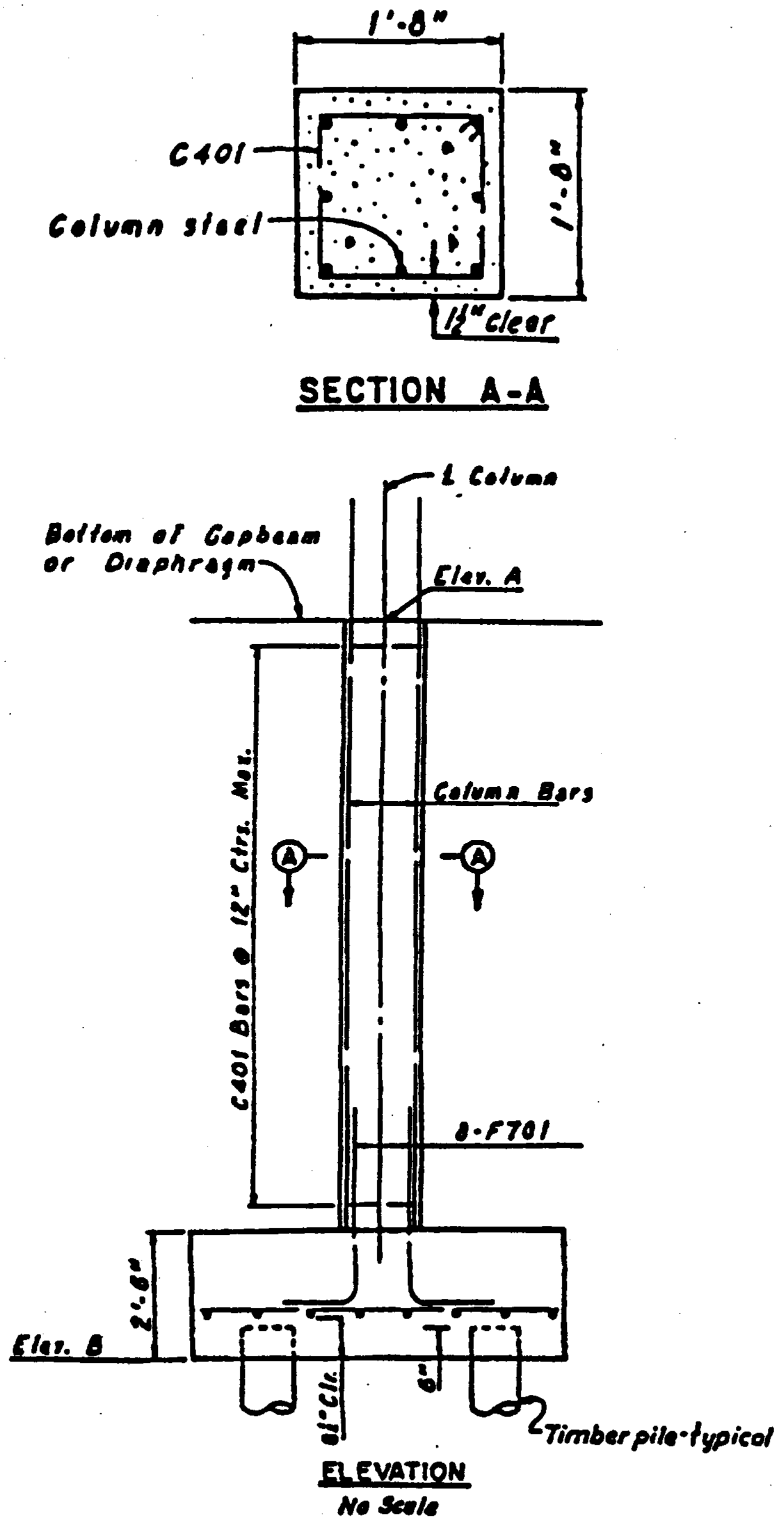

Figure 1 Example of Pre-1971 Bridge Substructure

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.

COLUMN RETROFITTING

Previous research (Priestley and Seible, 1991; Chai, Priestley and Seible, 1991) has shown that the most effective column retrofit method for both circular and rectangular columns is steel jacketing of the columns. The steel jacket is made slightly larger in size than the columns, and the space between the jacket and column is filled with grout. The 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 hinge and/or splice region performance (partial height jacketing) and column shear performance (full height jacketing).

Based on these research studies, CALTRANS (1992) has implemented standardized column retrofit procedures: the Class P retrofit and the Class F retrofit. Steel jackets with a minimum thickness of 1 cm (3/8 in.) are used, although details are also provided for a high strength fiber epoxy casing. 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 a 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. Details of the CALTRANS procedures for column retrofit design are provided in the "Memo to Designers, 20-4" (CALTRANS, 1992). Based largely on these procedures, the FHWA provides guidelines for retrofitting columns in its *Seismic Retrofitting Manual for Highway Bridges* (1995).

FOOTING RETROFITTING

CALTRANS (1992) has developed general 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.

Two areas of concern have been raised with the CALTRANS footing retrofit procedures. Priestley (1991) has noted the possible unconservativeness of using the full width of the footing in both shear and flexural calculations. He recommended an effective section width, and thus the participating reinforcement, be taken not larger than the column width plus twice the effective depth of the footing. Priestley also noted that the column/footing joint may be susceptible to shear failure. A test conducted at the University of California, San Diego of a typical 1960's design of a footing resulted in such a joint shear failure.

Priestley (1991) developed a simple method of checking the principal tension stress in the column/footing joint region for assessing joint shear failure. The principal tension stress in the joint region is calculated using Mohr's circle of stress based on the axial and shear stresses within the joint. The principal tension, f_t, is given by:

$$f_t = -f_a / 2 + \sqrt{(f_a / 2)^2 + v_{jv}^2} \qquad \text{(Equation 1)}$$

The effective axial stress, f_a, is given by

$$f_a = W_t / A_{eff} \qquad \text{(Equation 2)}$$

where the effective area, A_{eff}, over which the total axial load W_t at the column base is distributed, is taken as:

$$A_{eff} = (B_c + d_f)(D_c + d_f) \text{ for rectangular columns} \qquad \text{(Equation 3)}$$

or $$A_{eff} = \pi(D_c + d_f)^2 / 4 \text{ for circular columns} \qquad \text{(Equation 4)}$$

where D_c = overall section depth of rectangular column or the diameter of circular column; B_c = section width of rectangular column; and d_f = effective depth of footing. The vertical joint shear, V_{jv}, is assessed by subtracting the pile hold-down force, R_t, (if present) from the total tensile force in the critical section of the column, as shown in Figure 2:

$$V_{jv} = T_c - R_t \qquad \text{(Equation 5)}$$

The average joint shear stress, v_{jv}, is calculated as:

$$v_{jv} = V_{jv} / (b_{jeff}\, d_f) \qquad \text{(Equation 6)}$$

where $$b_{jeff} = B_c + D_c \text{ for rectangular columns} \qquad \text{(Equation 7)}$$

or $$b_{jeff} = \sqrt{2}\, D_c \text{ for circular columns} \qquad \text{(Equation 8)}$$

Based on this approach, Priestley has proposed joint shear distress will occur when the principal tension stress exceeds 0.29 $\sqrt{f'_c}$ MPa (3.5 $\sqrt{f'_c}$ psi), where f'_c is the concrete compressive strength. However, values up to 0.42 $\sqrt{f'_c}$ MPa (5.0 $\sqrt{f'_c}$ psi) can be sustained if the column and footing remain in the elastic range.

Xiao, Priestley, Seible and Hamada (1994) tested specimens with as-built and

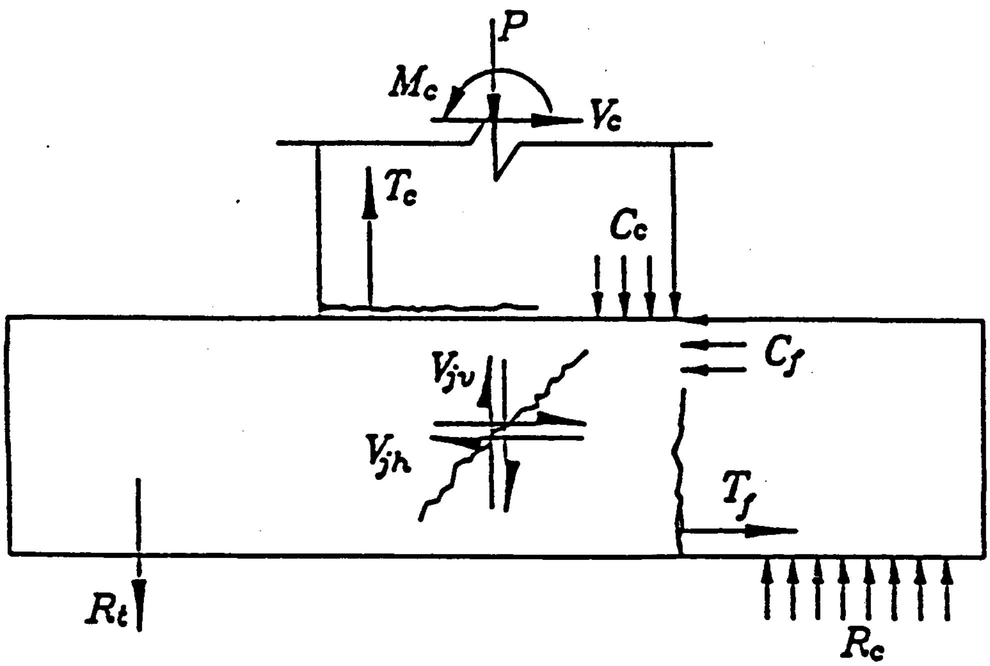

Figure 2 Forces in the Column/Footing Joint Region (from Xiao, et al, 1994)

retrofitted footings. Tests on the as-built specimen resulted in a column/footing joint shear failure. The calculated principal tension stress in the column/footing joint region was 0.44 $\sqrt{f'_c}$ MPa (5.22 $\sqrt{f'_c}$ psi), supporting the previously discussed stress limits. 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 resisting mechanisms was proposed and verified. A strut-and-tie model with a yield line mechanism was developed to analyze the resisting mechanisms in the retrofitted footings.

In the FHWA's *Seismic Retrofitting Manual for Highway Bridges* (1995), only general guidelines based on theoretical considerations are given. The retrofit manual notes the need for experimental testing of footing retrofit strategies as several recommendations are given as being tentative pending verification by tests.

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 or spread footing, was used as the basis for evaluating as-built and retrofitted substructure performance. The prototype column and footing were formulated by compiling design plans from the 1950's and 1960's for bridges in Washington State. Emphasis was placed on single-column bent bridges as these bridges are likely to be more critical than multi-column bent bridges, and thus they will be the first type of bridges targeted for retrofit. 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.

The prototype pile-supported substructure section chosen for study consisted of a 3.7 m by 3.7 m (12 ft 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 of the pile-cap were selected as 0.42% and 0.28% for the longitudinal and transverse steel, respectively, and the column reinforcing ratio was selected as 2.2%. Details included column lap splice lengths of 20 d_b and 35 d_b. Timber piles were selected for study in this investigation, as they are common in many older foundations in Washington State. Based on 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 prototype spread footing substructure section chosen for study consisted of a 4.6 m by 4.6 m (15 ft by 15 ft) square spread footing, with a thickness of 0.9 m (3 ft), and a 1.2 m (4 ft) circular column. The reinforcing ratios of the footing were selected as 0.55% and 0.31% for the longitudinal and transverse steel, respectively, and the column reinforcing ratio was selected as 2.5%. Details included column lap splice lengths of 20 d_b and 35 d_b.

The experimental tests were conducted on 1/3-scale specimens which 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 both 20 d_b and 35 d_b *splices*, and thus required retrofitting. 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. A total of ten specimens were tested: five pile-supported specimens and five spread footing specimens. Details of Specimen No.s P1 and S1, representing the as-built details for the pile-supported and spread footings, are shown in Figures 3 and 4, respectively. The various retrofit measures applied to the remaining specimens are discussed later along with the test results.

All specimens were constructed using concrete with a 28-day target compressive strengths of 21 MPa (3000 psi) for the footings, 28 MPa (4000 psi) for the columns, and 28 MPa (4000 psi) for the footing retrofits. Grout used for the retrofit jacket on the columns had a 28-day compressive strength of 48 MPa (7000 psi). Grade 40 reinforcement was used for the portions of the specimens representing the as-built substructures. Grade 60 reinforcement was used within the footing retrofit sections. The steel jacketing for the columns consisted of 10 gauge (3.4 mm (0.13 in.) thick) hot-rolled sheet metal with a yield strength of 255 MPa (37 ksi). A high modulus, low viscosity epoxy was used to anchor retrofit reinforcing dowels into existing concrete. Additional details of the testing program are given in Saunders (1993).

TEST SETUP AND PROCEDURES

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 soil was contained within a stiff box constructed of large glue-laminated wood beams. For the spread footings, soil was compacted within the testing area, the fully constructed specimen placed on the compacted soil, and then soil compacted around the footing. For the

Table 1 Summary of the Test Specimens

Specimen No.	Footing Type	Footing Deficiency	Footing Retrofit Applied	Column Type	Column Splice	Column Axial Load P/f'_cA_g
P1	pile	shear	none	square	20 d_b	0.104
P2	pile	shear	overlay and pedestal	square	20 d_b	0.104
P3	pile	shear	overlay	square	35 d_b	0.104
P4	pile	shear and overturning	overlay, low tension piles, rocking	square	35 d_b	0.069
P5	pile	shear and overturning	overlay, low tension piles, added piles	square	35 d_b	0.069
S1	spread	shear	none	circular	20 d_b	0.075
S2	spread	shear and overturning	overlay, pedestal, tie-downs	circular	20 d_b	0.075
S3	spread	shear and overturning	overlay and tie-downs	circular	35 d_b	0.075
S4	spread	shear and overturning	overlay, rocking	circular	35 d_b	0.075
S5	spread	shear and overturning	overlay, pedestal, enlarged footing	circular	20 d_b	0.075

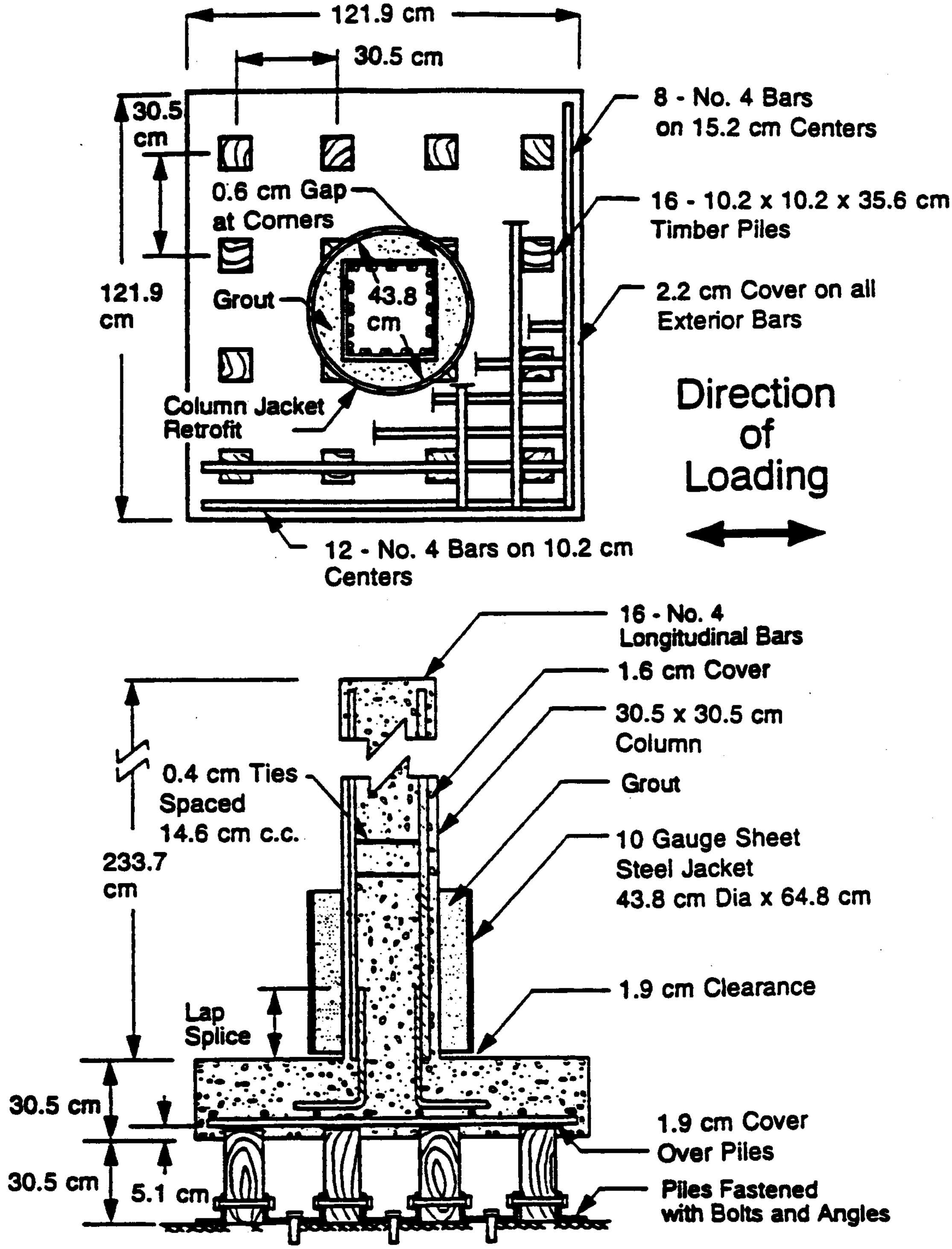

Figure 3 Details of Specimen No. P1 Representing the As-Built Pile-Supported Specimen

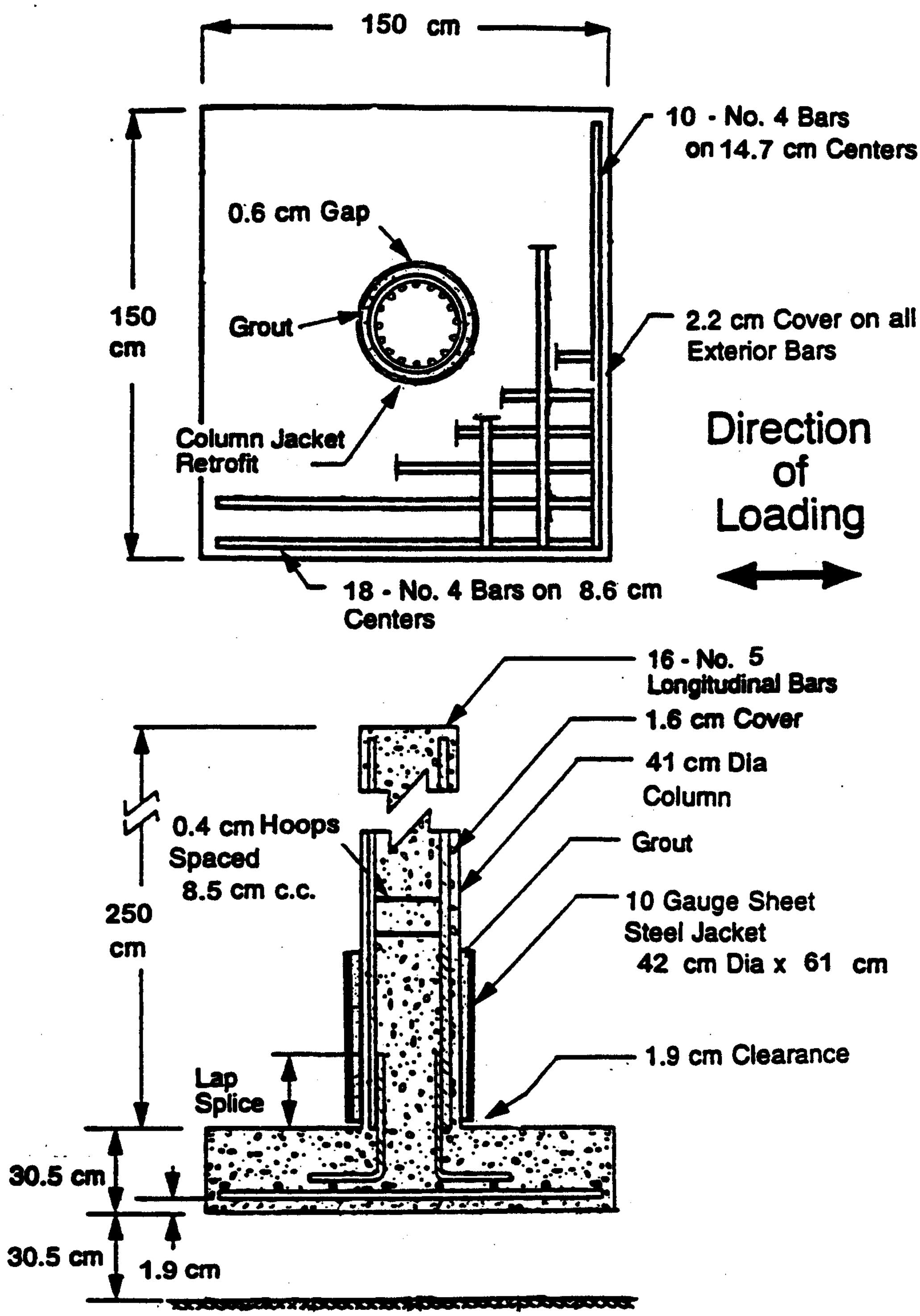

Figure 4 Details of Specimen No. S1 Representing the As-Built Spread Footing Specimen

pile-supported specimens, soil was compacted between the piles prior to construction of the pile cap, and then around the pile cap after it was poured in place, as shown in Figure 5.

The soil possessed a friction angle of 35° and was compacted to approximately 95% modified Proctor. The objectives of this test setup were to allow the footings to rotate and to approximately simulate the actual footing support conditions. This setup over-confines the soil, when compared to field conditions, and there is significant labor involved in setting up and removing a specimen. However, this setup is more realistic than the support conditions often used in laboratory tests whereby the footing is bolted to a strong floor, thus not allowing any footing rotations.

The overall test setup is shown in the drawing of Figure 6 and the photograph of Figure 7. 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 (1987). 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 gages were used to monitor the strains in the flexural and transverse reinforcement. Linear variable displacement transformers (LVDT's) and load cells measured

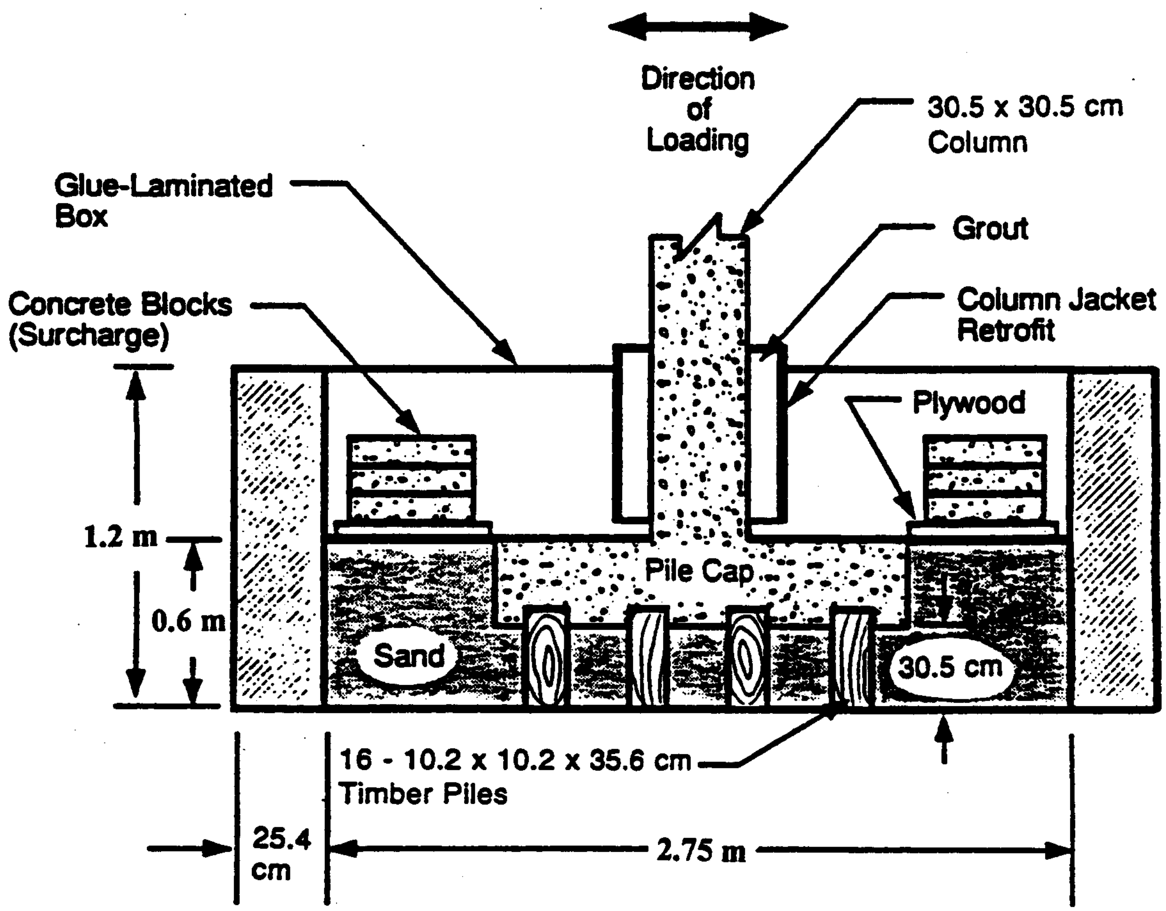

Figure 5 Testing Support Conditions for the Pile-Supported Specimens

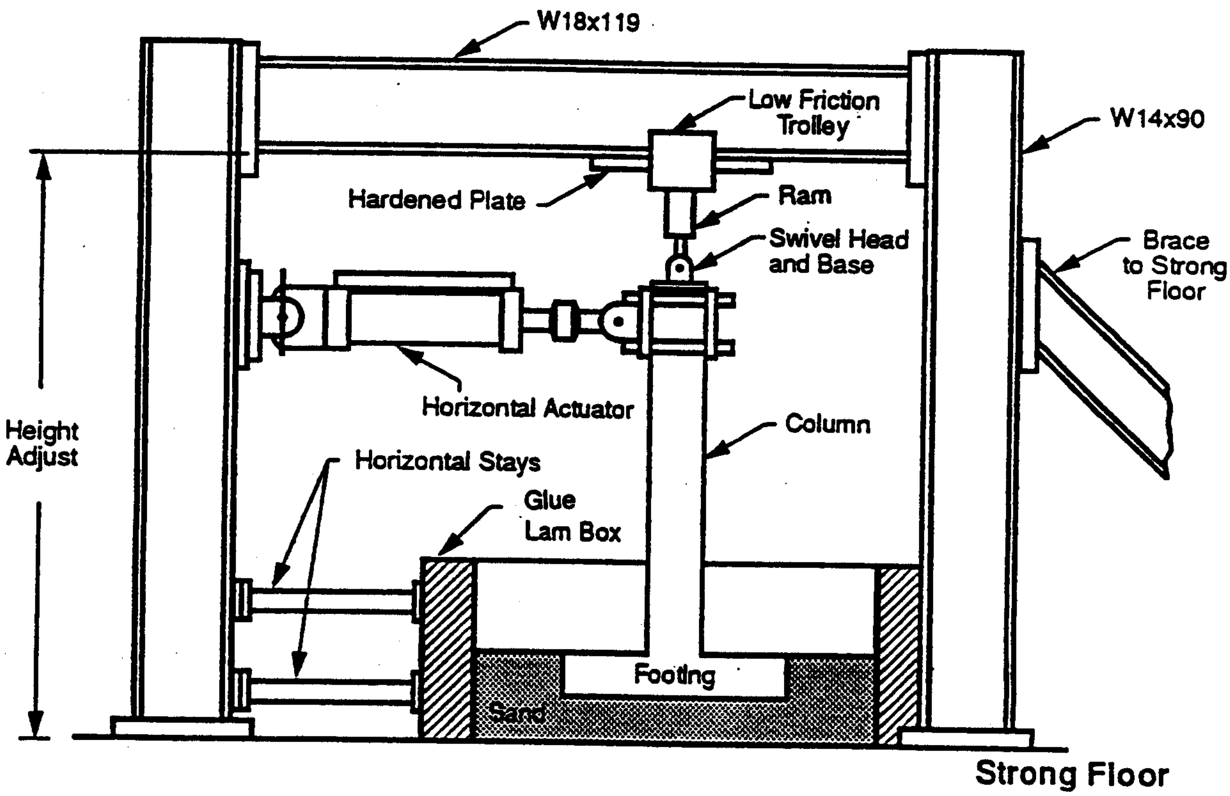

Figure 6 Testing Setup

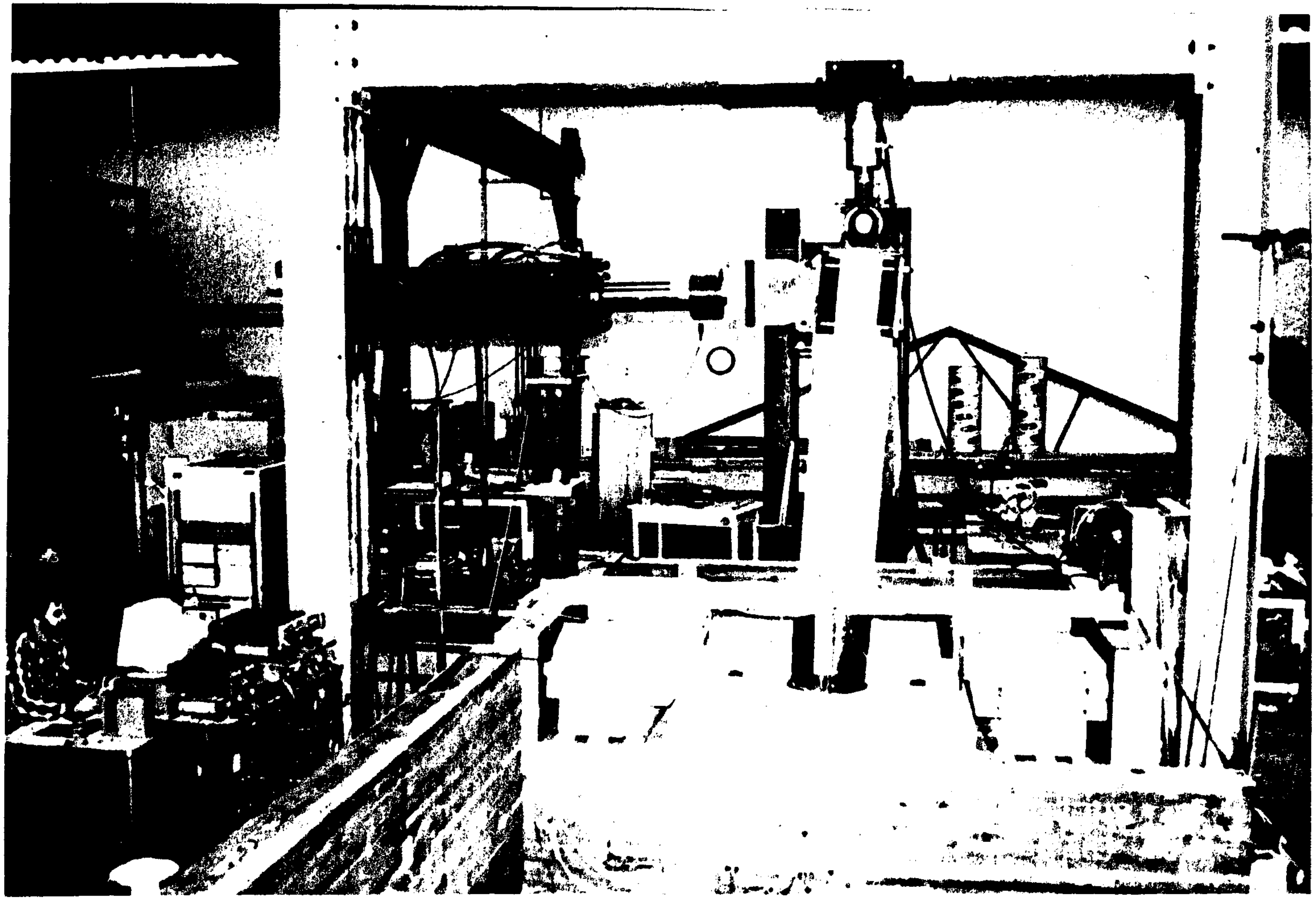

Figure 7 Photograph Showing the Test Setup

column displacements and applied loads. LVDT's were also placed on the top of the footings to determine footing displacements and rotations. Several of the wood piles were instrumented with strain gages and were calibrated under compressive loading in an attempt to monitor loads in the piles. All data was recorded intermittently during testing.

RESEARCH FINDINGS AND DISCUSSION

In this section, results of the experimental tests are summarized. Results from the specimens representing the as-built conditions, Specimen No.s P1 and S1, are presented first. These results were used to formulate the retrofits for the subsequent specimens. Specimen performance was evaluated on the basis of load capacity, displacement ductility, strength degradation and hysteretic behavior.

TESTS ON THE AS-BUILT SPECIMENS

Specimen No.s P1 and S1 were designed to be representative of as-built 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. The columns of both specimens contained a 20 d_b lap splice and were retrofitted at the base with a steel jacket.

General Behavior

Failure in Specimen No. P1 occurred during loading to a displacement level of 2 Δ_y. The resulting hysteresis curves for Specimen No. P1 are shown in Figure 8 and indicate little

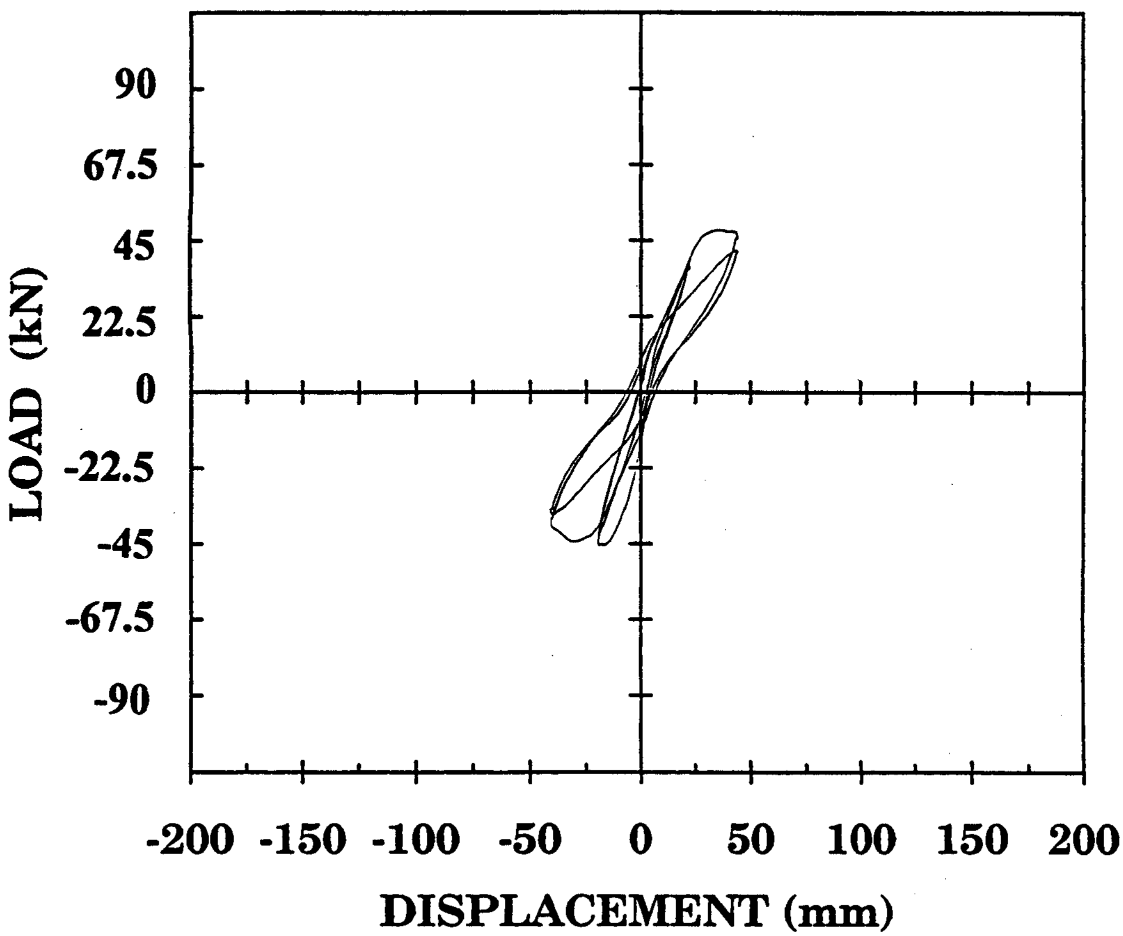

Figure 8 Load-deflection Curves for Specimen No. P1

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% 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 pile cap. Only minor cracking was observed on the bottom of the pile cap. The major cracks occurring in Specimen No. P1 are shown in Figure 9.

Failure in Specimen No. S1 occurred during loading to a displacement level of 3 Δ_y. The resulting hysteresis curves for Specimen No. S1 are shown in Figure 10 and indicate little energy dissipation. The peak applied lateral load was 80.0 kN (18.0 kips) and occurred at a column tip displacement of 47.3 mm (1.86 in.). The column reached approximately 68% of its moment capacity before the specimen failed. The column showed only minimal signs of cracking. During testing, the top of the footing developed cracking radiating outward from the column similar to that observed in the pile-cap specimen. After removing the specimen from the testing setup, cracks were also observed on the sides of the footing parallel to the direction of loading. All indications were that the spread footing specimen failed in a similar manner to that with the pile-supported specimen.

Failure Mechanism

The cracks observed in the both the pile cap of Specimen No. P1 and the footing of Specimen No. S1 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

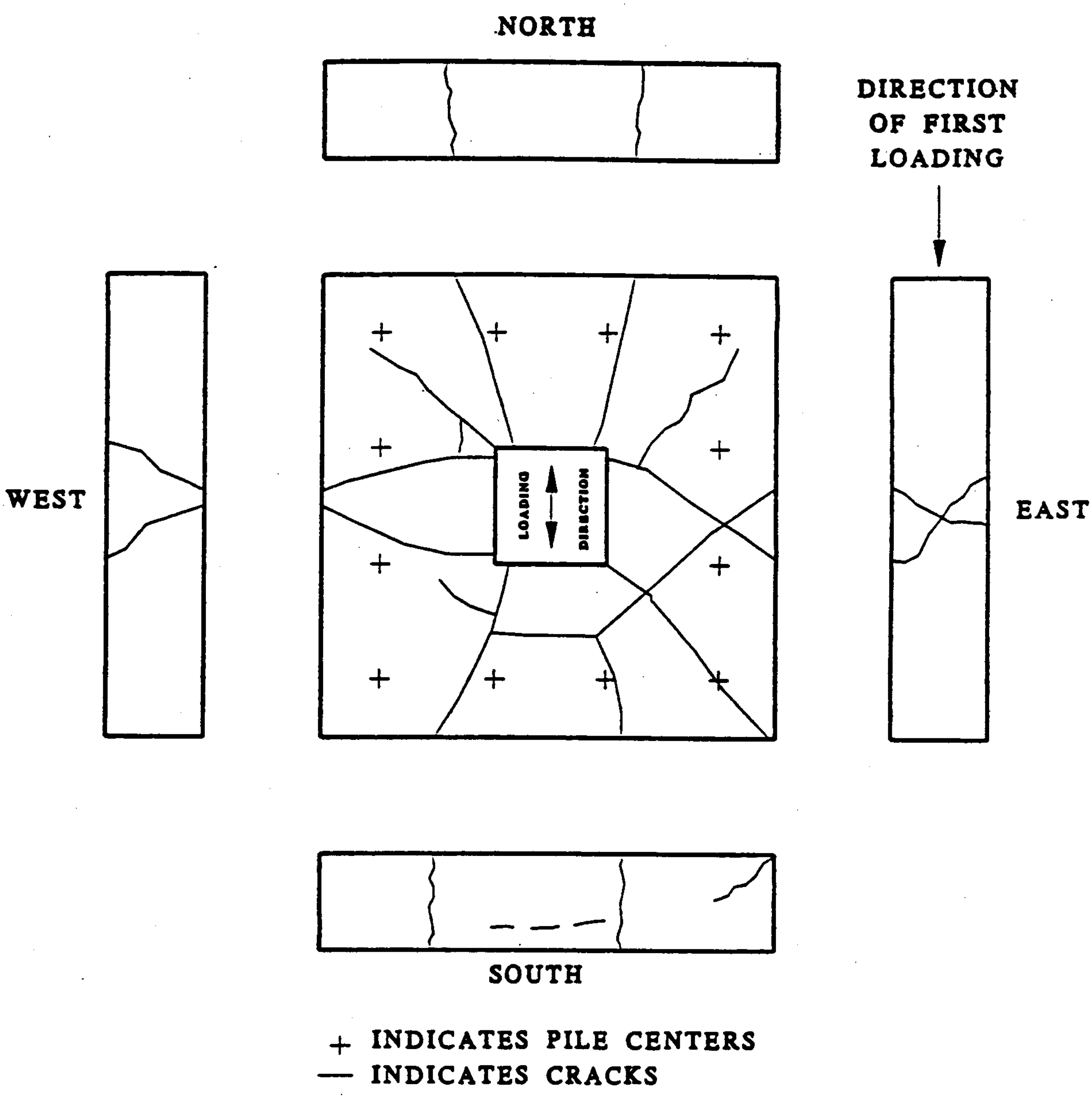

Figure 9 Major Cracks in the Top of Specimen No. P1

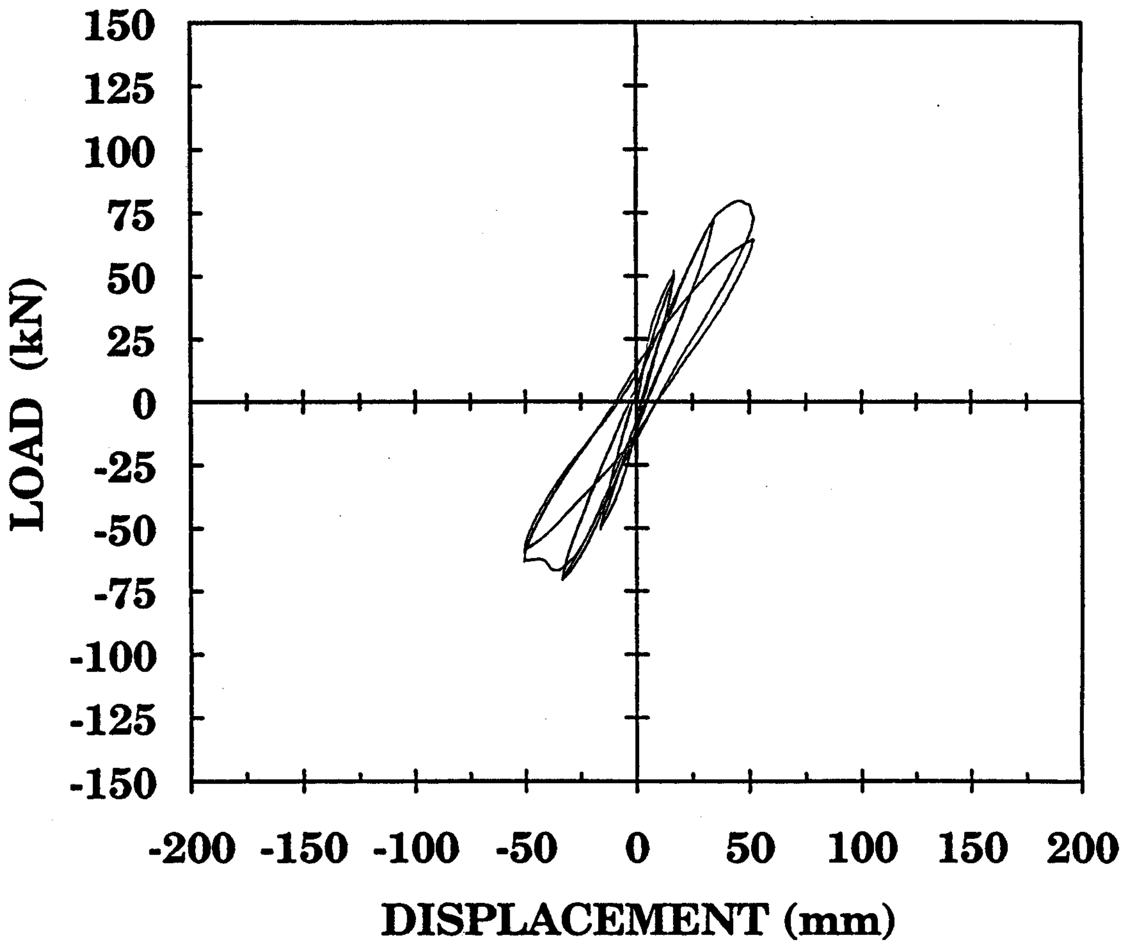

Figure 10 Load-deflection Curves for Specimen No. S1

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). To gain an understanding of the cause of the failure, a qualitative study (Cahill, 1993) was conducted using small-scale specimens which replicated the details of Specimen No. S1. 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 tests on the larger-scale Specimen No.s P1 and S1. A cross-section, showing the internal cracking patterns within the column/footing joint region, is shown in Figure 11a. A major diagonal crack developed within the column/footing connection. In the figure, 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 (see Figure 11b). Using the approach suggested by Priestley (1991) for assessing joint shear, maximum 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 Specimen No.s P1 and S1, respectively, supporting the conclusion that a joint shear failure in the column/footing connection was the failure mechanism.

(a)

(b)

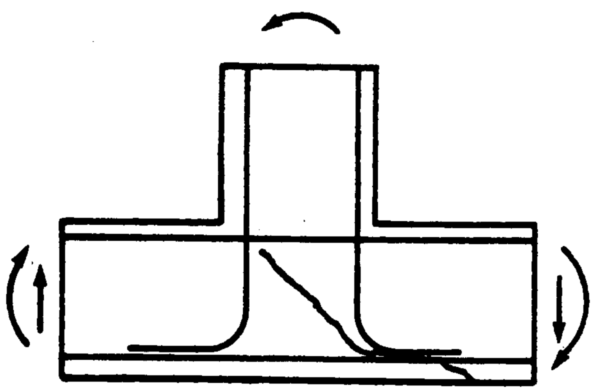

Figure 11 Cracking Patterns in (a) Small-Scale Column/Footing Joint and

(b) Column/Beam Joint

RETROFITTING FOR JOINT SHEAR

Specimen No. P2 was constructed and detailed similarly to Specimen No. P1, except that the pile cap was retrofitted to increase the thickness of the pile cap by adding a concrete overlay. This overlay intersected the splice region of the column and thus required special detailing. Specimen No. P2 incorporated a 20 d_b lap splice. In order to examine the effects of different lap splice lengths on the retrofit scheme, Specimen No. P3 was constructed and detailed similarly to Specimen No. P1 but with a 35 d_b lap splice. As the spread footing specimens are over-turning critical, in addition to being joint shear deficient, details of the retrofit measures for the spread footings are discussed in a later section.

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 they were drilled and epoxied into the top of the existing pile cap. The ends of the dowel were anchored into the retrofit overlay with 180° hooks. 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 limit proposed by Priestley (1991) and to allow for development of the shear friction dowels. An overlay thickness of 13 cm (5 in.) was used in the specimen.

The 20 d_b splice present in the column of Specimen No. P2 required special detailing since the overlay intersected the splice (in this case, at the 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 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. Crack control steel, consisting of a hoop and hairpins, was provided in the pedestal. Figure 12 illustrates the details of the retrofit used for Specimen No. P2. The column cover over the full height of the splice was removed prior to constructing the retrofit overlay. This was done 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, due to the inadequate transverse reinforcement present in the as-built column.

With the lap splice length of 35 d_b present in Specimen No. P3, the use of a pedestal to fully contain the splice would result in an unreasonably large pedestal. As in Specimen No. P2, an overlay thickness of 13 cm (5 in.) was chosen based on joint shear considerations. Thus, the overlay would intersect the splice at 13 cm (5 in.) or 10 d_b from the bottom of the splice, leaving 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. However, in order

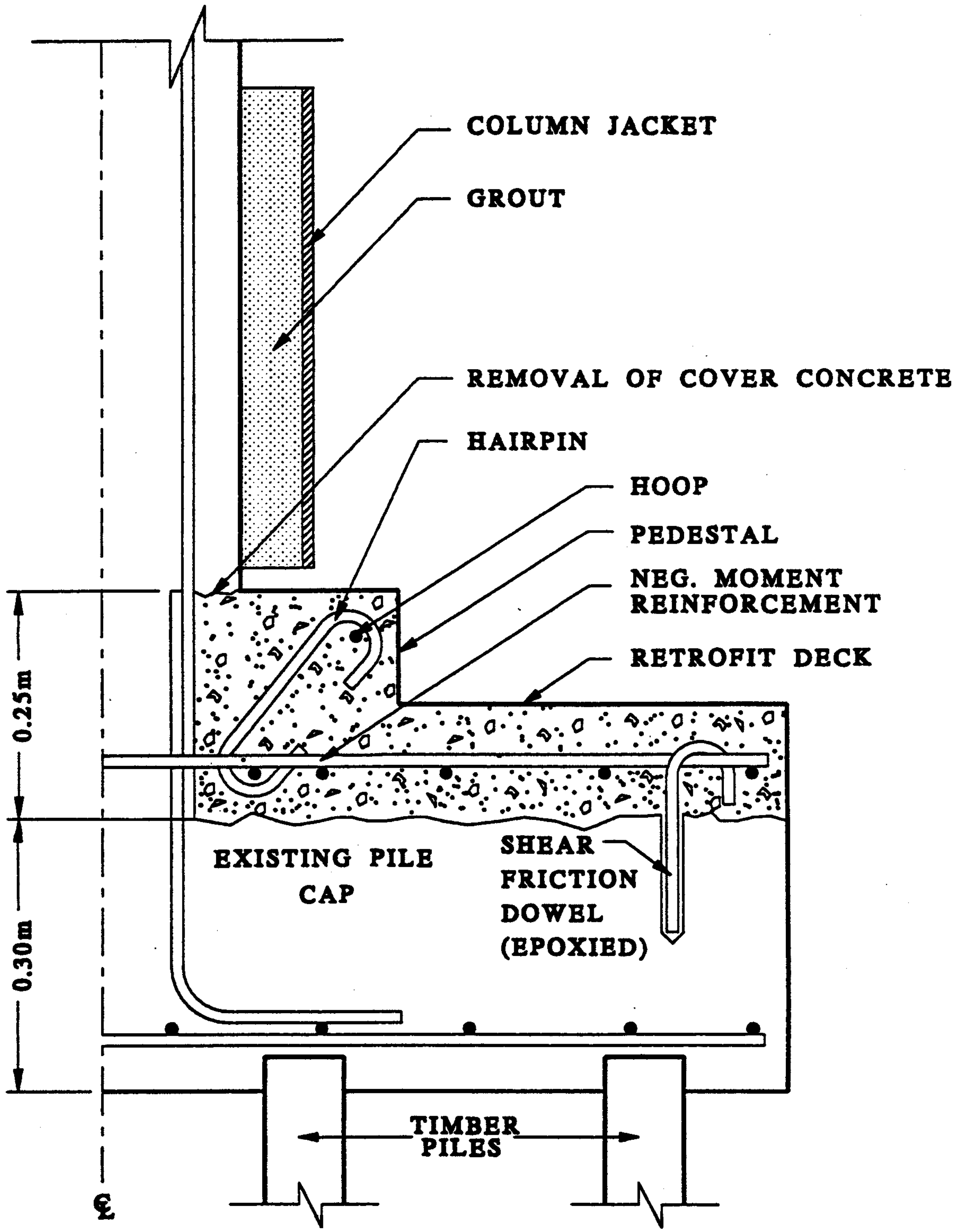

Figure 12 Retrofit Scheme Applied to Specimen No. P2

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. All other details of the retrofit were the same as for Specimen No. P2. Figure 13 shows the retrofit measures applied to Specimen No. P3.

Test Results

Specimen No. P2 performed very well with failure occurring at a displacement level of 10 Δ_y, as illustrated by the hysteresis curves shown in Figure 14. 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 Δ_y, a column longitudinal bar fractured. Prior to 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, have 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. Based on 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.

The hysteresis curves for Specimen No. P3 are shown in Figure 15 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 Δ_y, several dowel bars fractured and the test was stopped. Cracking resulting from plastic hinge

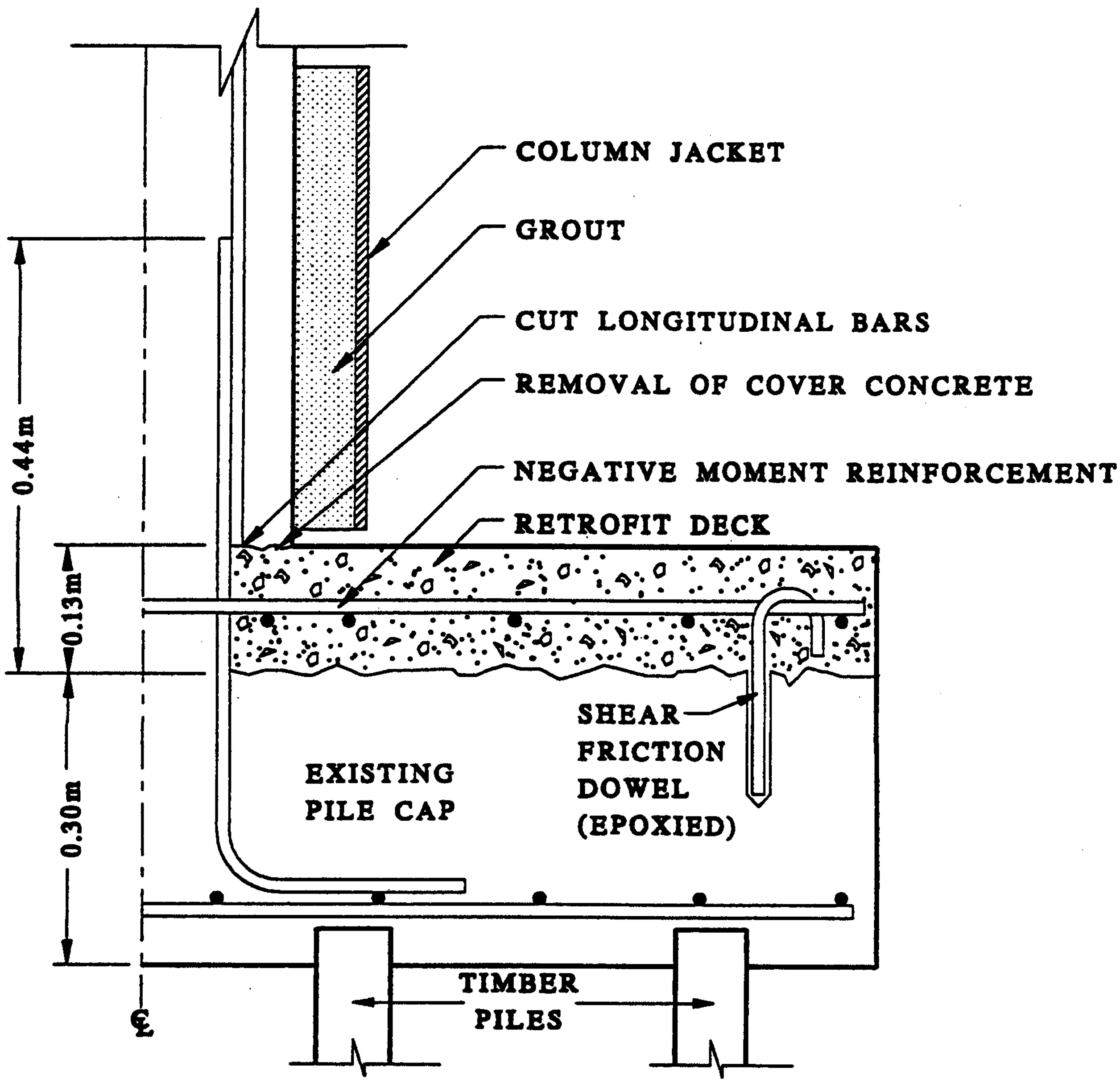

Figure 13 Retrofit Scheme Applied to Specimen P3

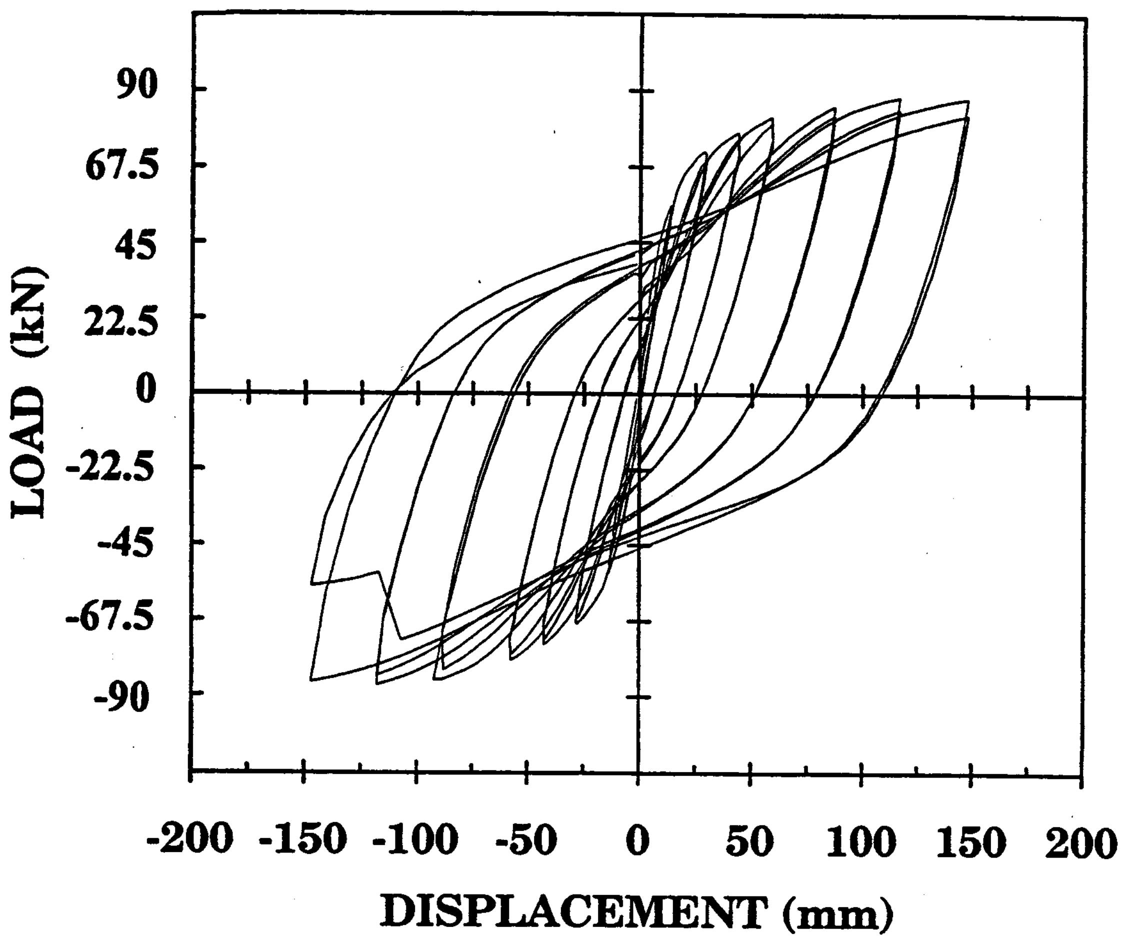

Figure 14 Load-deflection Curves for Specimen No. P2

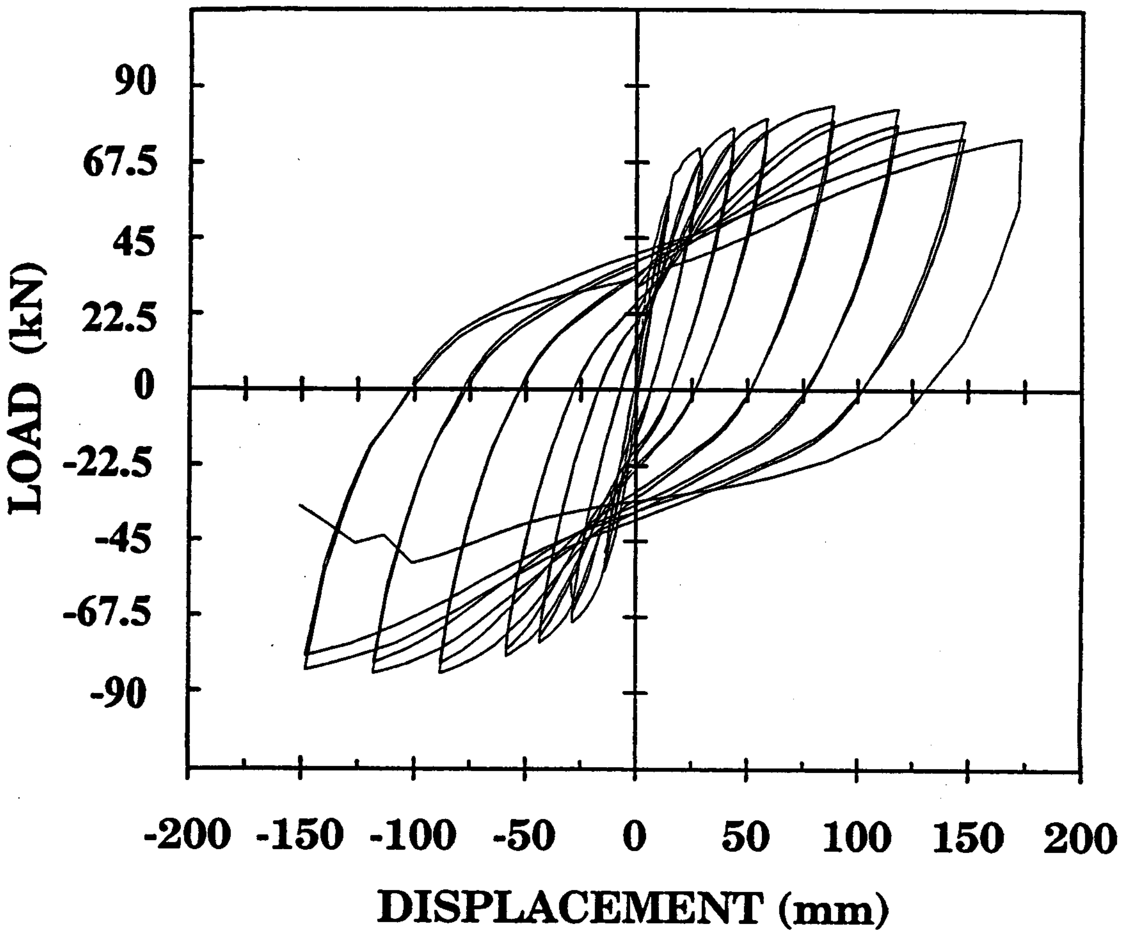

Figure 15 Load-deflection Curves for Specimen No. P3

penetration occurred in the top of the pile cap and was more extensive than the cracking observed in Specimen No. P2. After removing the specimen 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.

SPECIMENS WITH ROCKING

Specimen No.s P4 and S4 were designed to examine the "rocking" behavior of a footing system when uplift occurs. Rocking may result when the tension capacity of the piles is lost or nonexistent or when the footing is undersized with respect to developing the capacity of the columns. This rocking behavior would be relevant to foundations that, perhaps by choice, were not retrofitted.

Retrofit Description

Specimen No. P4 incorporated a 35 d_b lap splice, and the details of the retrofit were similar to those used for Specimen No. P3. However, the tops of the piles were greased and a layer of crushable foam 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. P4 to ensure that the specimen would be overturning critical.

Specimen No. S4 also incorporated a 35 d_b lap splice, and the details of the retrofit were similar to those used for Specimen No. P3, except that the specimen incorporated a spread footing. The size of the footing was such that it was unable to develop the capacity of the column without overturning.

Test Results

Figures 16 and 17 show the hysteresis curves for Specimen No.s P4 and S4, respectively. The "S" shape of the hysteresis curves is the result of the uplift and rotation of the footings. The peak applied lateral loads are approximately 67 kN (15 kips) (80% of the column capacity) and 102 kN (23 kips) (81% of the column capacity) for Specimen No.s P4 and S4, respectively. 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 rocked.

RETROFITTING TO INCREASE OVERTURNING RESISTANCE

Several different retrofit methods were evaluated for effectiveness in increasing the overturning resistance of the footings. Specimen No. P5 was retrofitted by enlarging the footing and adding additional piles to increase the overturning resistance. Specimen No.s S2 and S3 incorporated anchors through the footing to increase the overturning resistance, and Specimen No. S5 incorporated an enlarged footing.

Retrofit Description

The footing size for Specimen No. P5 was enlarged by adding 0.3 m (12 in.) to each end in the direction of loading. The tensile capacity of the wood piles in Specimen No. P5 was suppressed as in Specimen No. P4. Eight additional piles were added, four at each end,

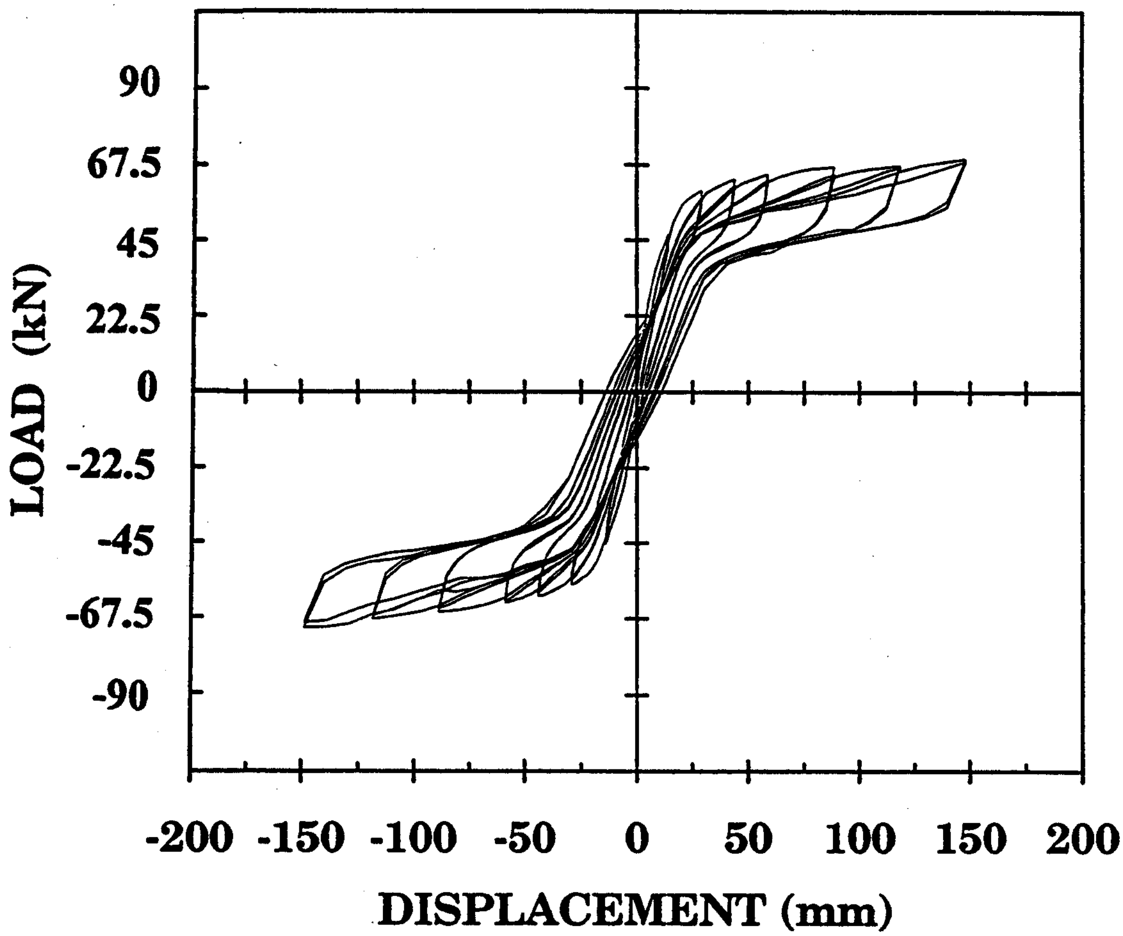

Figure 16 Load-deflection Curves for Specimen No. P4

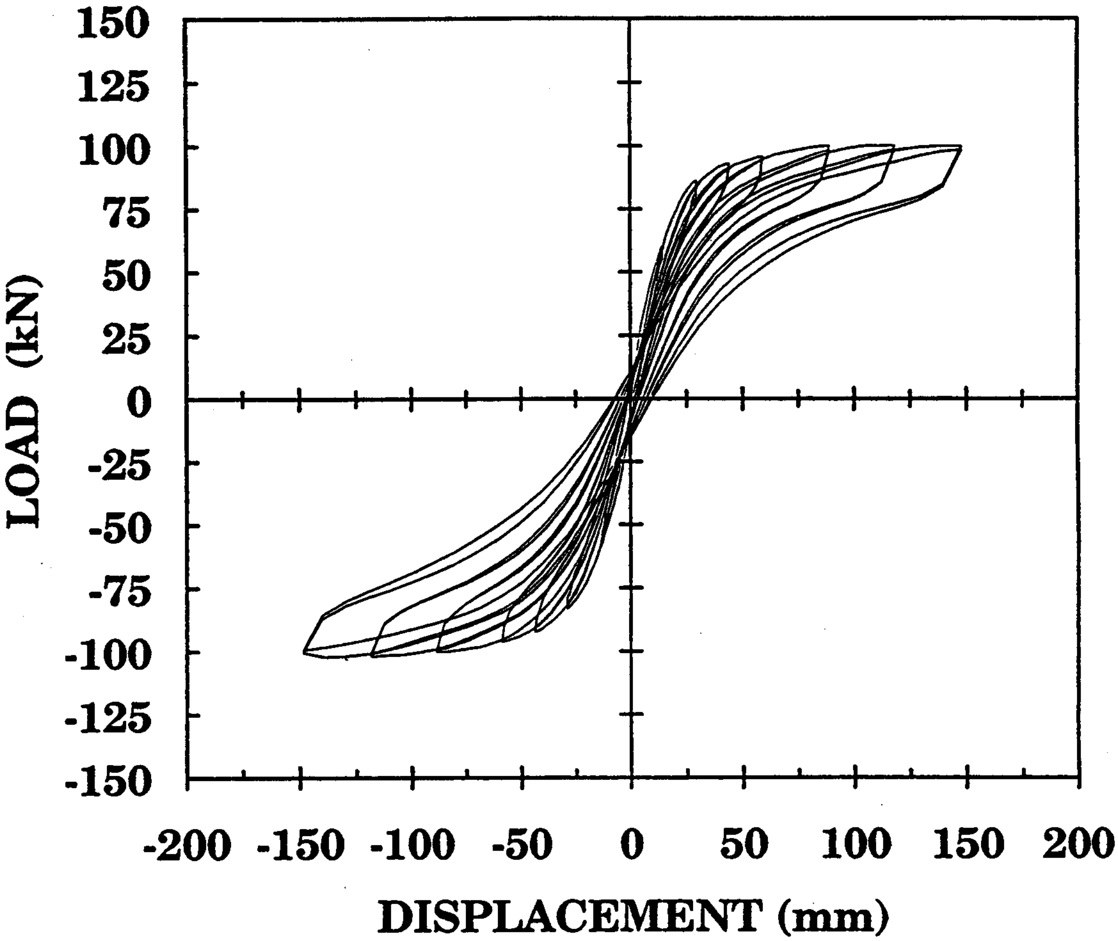

Figure 17 Load-deflection Curves for Specimen No. S4

to increase the overturning resistance. A 35 d_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 No.s P3 and P4. 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 this type of pile 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.

The footing size of Specimen No. S5 was also enlarged by adding 0.3 m 9 (12 in.) to each end in the direction of loading. A 20 d_b lap splice was present, and a footing overlay retrofit similar to that used for Specimen No. P2 was applied.

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. The retrofit design for Specimen No. P5 is shown in Figure 18.

Specimen No.s S2 and S3 incorporated footing tie-downs in order to increase the overturning resistance. In the specimens, the tie-downs consisted of steel rods which passed through holes in the footing and were anchored into the testing floor below the specimens. Specimen No. S2 had a 20 d_b lap splice and incorporated a retrofit overlay detailed similarly to that used for Specimen No. P2. Specimen No. S3 had a 35 d_b lap splice and incorporated

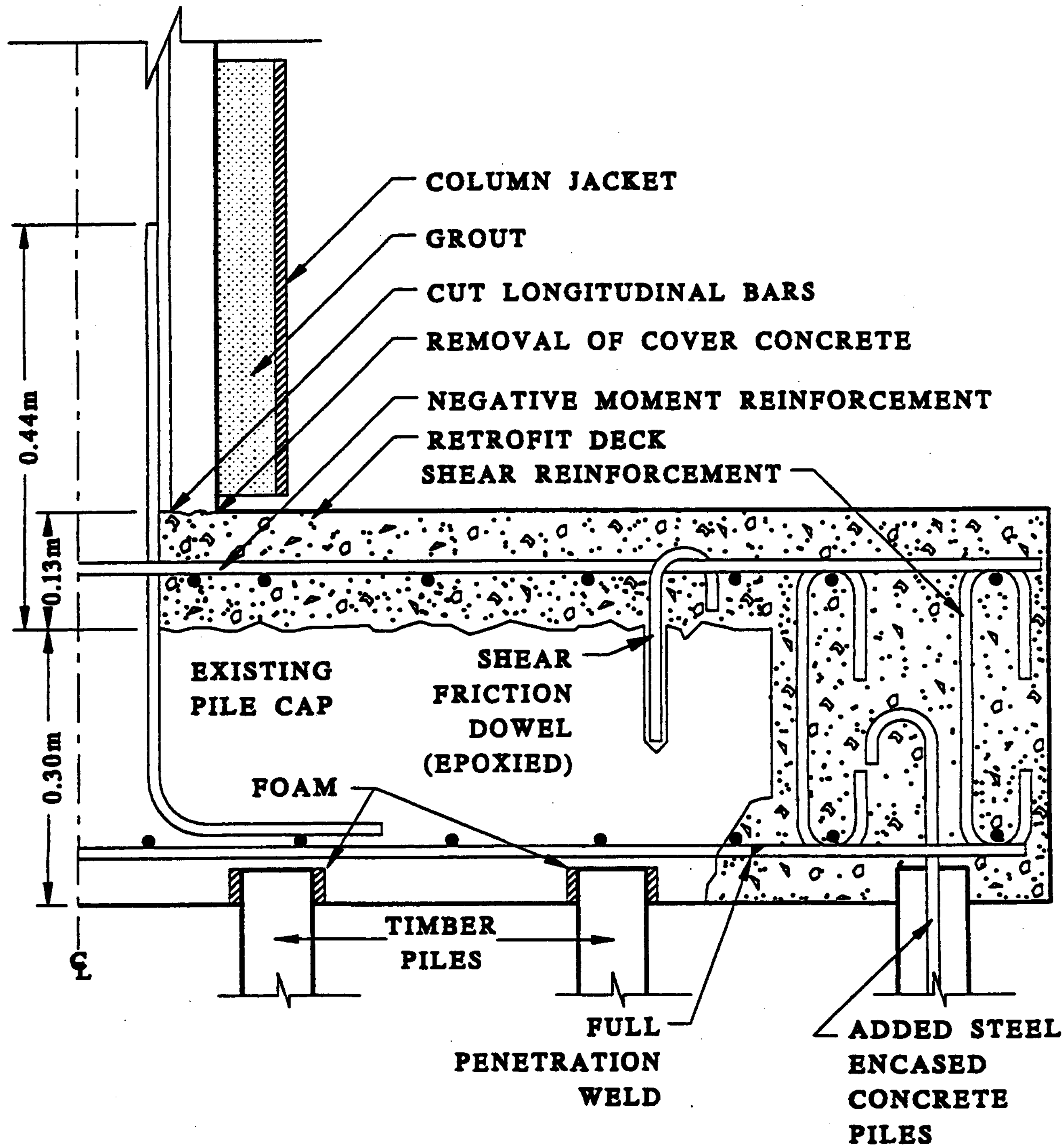

Figure 18 Retrofit Scheme Applied for Specimen P5

a retrofit overlay detailed similarly to that used for Specimen No. P3.

Test Results

The hysteresis curves for Specimen No. P5 are shown in Figure 19. 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 Δ_y, five of the outermost dowel bars fractured due to 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. P3, cracks developed in the top of the pile cap and extended towards the sides. Cracks also developed in the top of the pile cap around the column due to plastic hinge penetration. However, the cracking was controlled by the top mat of reinforcement in the retrofit overlay and specimen performance was very satisfactory.

The hysteresis curves for Specimen No.s S5, S2 and S3 are shown in Figures 20, 21 and 22, respectively. In the specimens with the soil anchors, uplift of the specimen was negligible. In the specimen with the enlarged footing size, some uplift did occur. However, in all three specimens, the specimen response was ductile, with failure resulting from eventual low-cycle fatigue fracture of the longitudinal bars during cycling to a displacement level of 12 Δ_y. The peak applied lateral loads were 135 kN (30.4 kips), 138 kN (31 kips) and 119 kN (26.8 kips) for Specimen No.s S5, S2 and S3, respectively.

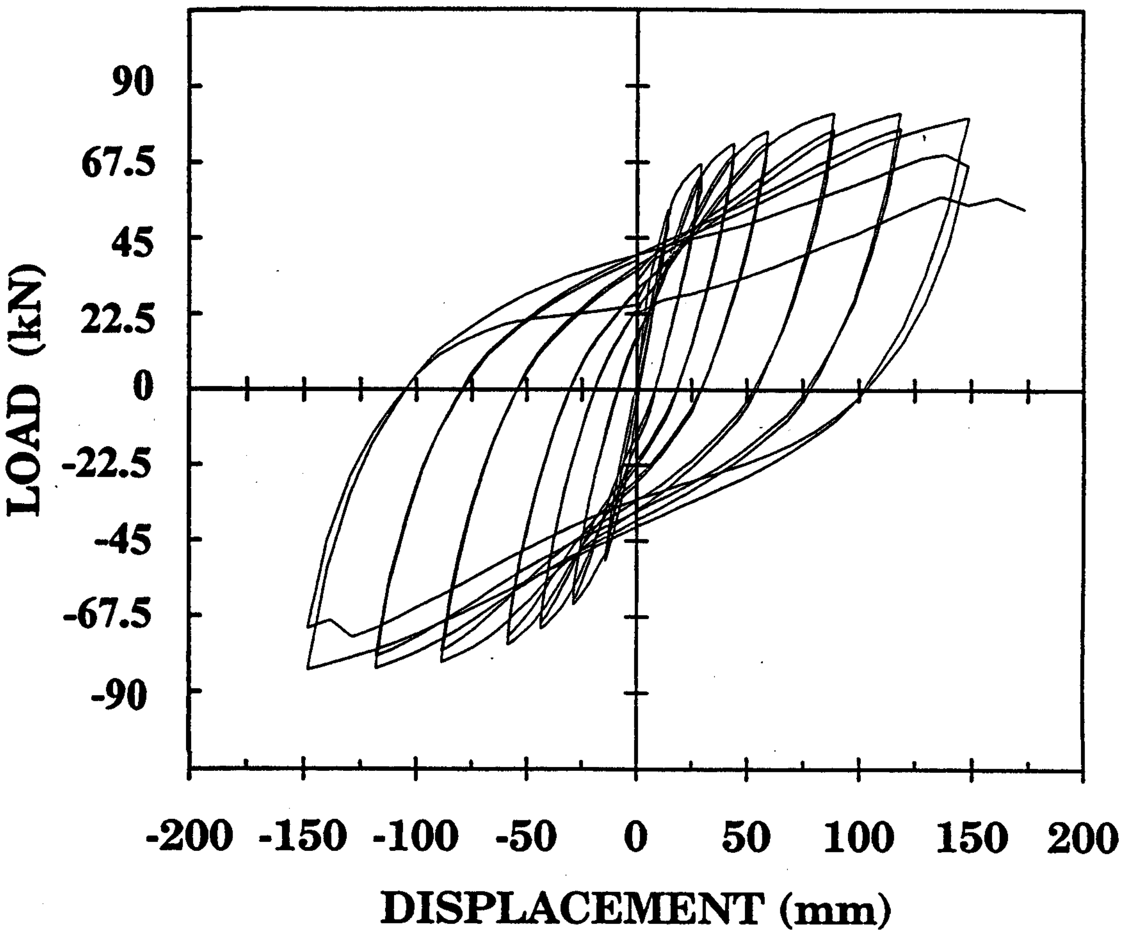

Figure 19 Load-deflection curves for Specimen No. P5

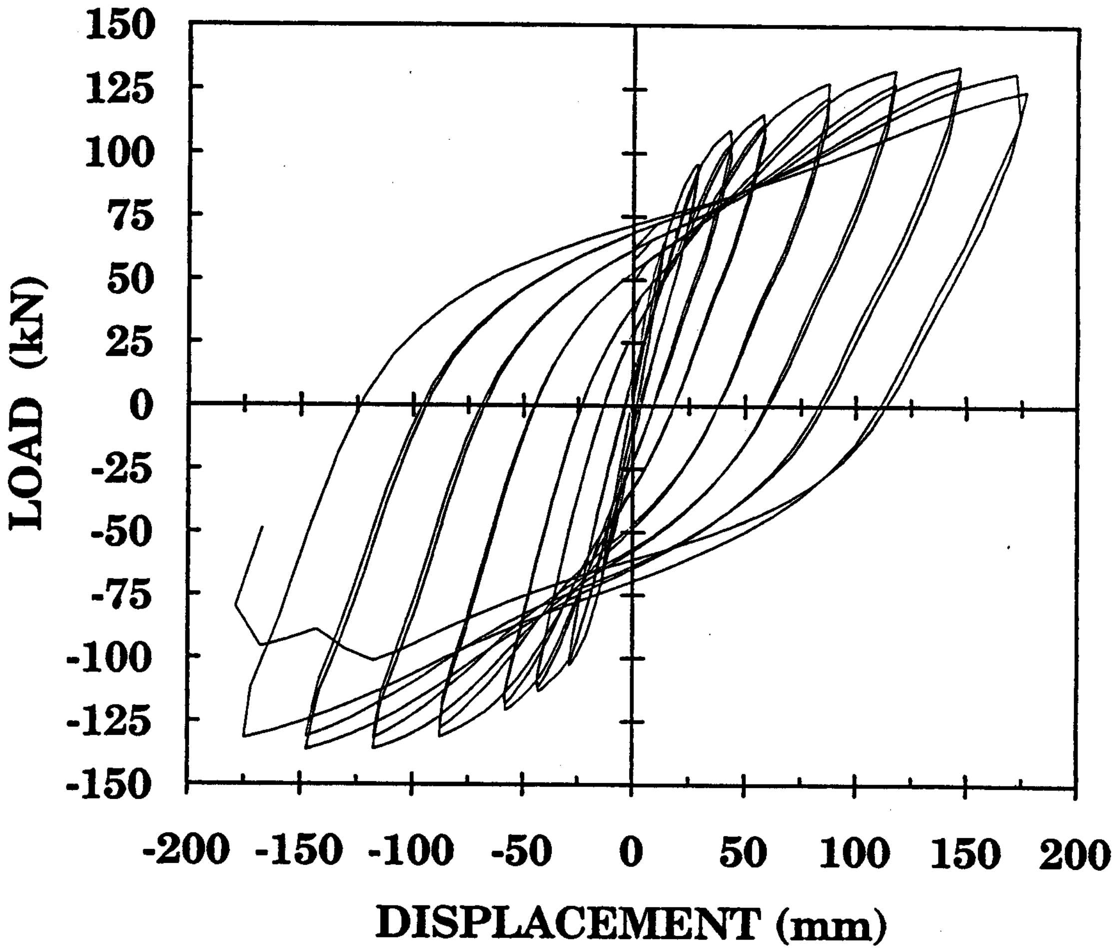

Figure 20 Load-deflection curves for Specimen No. S5

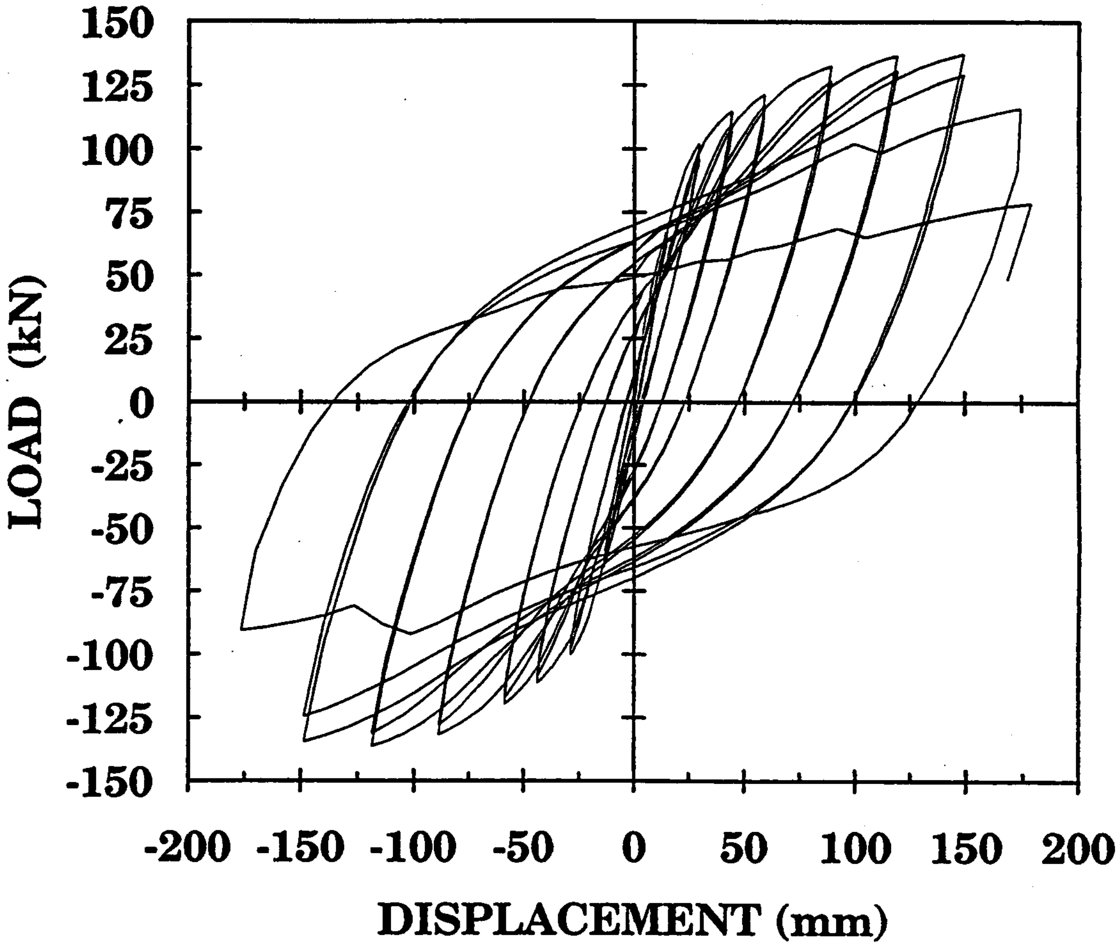

Figure 21 Load-deflection curves for Specimen No. S2

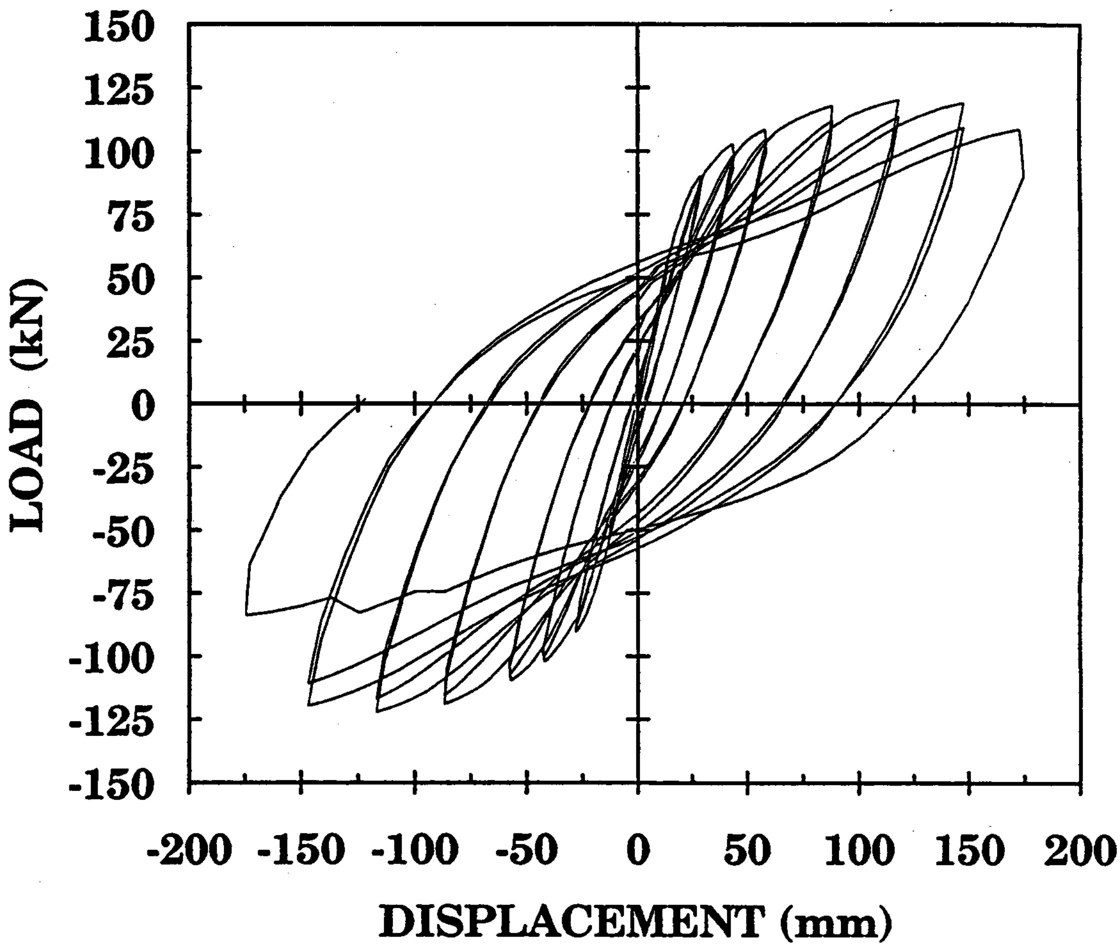

Figure 22 Load-deflection curves for specimen No. S3

CONCLUSIONS

The experimental test results of this study indicate that existing bridge footings may perform poorly under seismic loading. The as-built specimen 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 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 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.

RECOMMENDATIONS/APPLICATIONS/IMPLEMENTATION

The results of this research provide a basis for designing retrofit measures to improve the seismic performance of the substructures of existing bridges. An analysis of the existing bridge must first be performed and the seismic deficiencies identified. In those bridges in which the analyses indicate that the substructures are vulnerable, retrofit measures must be applied so as to produce a ductile response the in the overall system. The effects of the retrofitting on transferring forces to other components of the bridge must be considered in selecting the appropriate measures.

In single column bent bridges which have substructure deficiencies, the retrofit strategy will typically consist of retrofitting the bridge to produce ductile plastic hinging in the columns. Columns that are deficient in flexural ductility capacity or incorporate inadequate lap splices may be retrofitted using steel jacketing, pretensioned hoops, or fiberglass/epoxy jacketing. Circular jacketing or hoops are used for circular columns, and oval jacketing is used for rectangular columns. Columns deficient in shear strength can be retrofitted using similar techniques applied over the full height of the column. While laboratory tests have shown that rectangular jacketing can be effective in increasing column shear strength, additional measures are required in the hinging regions in order to obtain adequate flexural ductilities. Details and procedures for the design of column retrofit measures are provided in Priestley and Seible (1991), Caltrans Memo 20-4 (1992) and the FHWA *Seismic Retrofitting Manual for Highway Bridges* (1995).

The footings of existing bridges must be evaluated for their ability to carry the input column loads, including recognition of the maximum possible forces resulting from retrofitting measures applied elsewhere in the bridge. Footings need to be evaluated for flexural and shear strength, joint shear strength and footing uplift. Results from this research project and from other research indicate that existing footings may be particularly vulnerable to brittle joint shear failure. Procedures for assessing joint shear capacity have been proposed by Priestley (1991), and the applicability of these procedures was supported by the results of this study.

The results of this study show that an effective way to retrofit to increase joint shear capacity consists of providing an added reinforced concrete overlay to the existing footings. The thickness of the overlay is selected to produce joint shear stresses below the limit proposed by Priestley (1991). Dowel reinforcement, designed using shear friction theory, is included in the retrofit in order to provide composite action between the overlay and the existing footing. A top mat of reinforcement is included in the overlay so as to provide the required flexural strength in the footing. If lap splices are present in the overlay region, special detailing is required in order to maintain the integrity of the splices. If a 20 d_b lap splice is present, the overlay should incorporate a pedestal enclosing the full height of the splice. For lap splices of greater length, a similar pedestal may be used. Alternatively, the overlay may intersect the splice region, provided a confined splice length of at least 20 d_b is maintained above the overlay and the column bars are cut at the top of the overlay.

In footings that are susceptible to overturning, resistance may be improved by increasing the footing plan size, providing additional piles detailed to provide a tension connection between the piles and the footing, and by providing soil or rock anchors through the footing. Additionally, rocking of the footings may be an acceptable alternative if it can be shown that displacements associated with the rocking are tolerable by the bridge system.

Retrofit measures must also address the cap beams and joints and the possibility of reinforcement pullout or bond distress. These areas were not addressed in this research. Guidelines for assessing and designing retrofit measures for areas are provided in Priestley (1991), Priestley and Seible (1991) and the FHWA *Seismic Retrofitting Manual for Highway Bridges* (1995).

ACKNOWLEDGMENTS

This research project was funded by the Washington State Department of Transportation. The investigators gratefully acknowledge the contributions of the engineers in the WSDOT Bridge Office, particularly those of Edward Henley and Harvey Coffman. The investigators also acknowledge the valuable contributions, insight and discussions provided by Lee Marsh. The grout and epoxy materials used in this study were donated by the Sika Corporation through the efforts of Thad Brown.

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