Seismic Retrofit of Bridge Columns by Steel Jacketing

Y. H. Chai, M. J. Nigel Priestley, and Frieder Seible

Inadequate flexural strength and ductility of shear strength of concrete bridge columns has resulted in collapse or severe damage of a number of California bridges in recent moderate earthquakes. In general these bridges were designed prior to the new seismic design methods which were implemented in the mid-seventies. Bridges constructed in accordance with the new design methods have performed well in recent earthquakes. However, the large number of older bridges that are in service, particularly freeway overpasses designed and constructed in the 1950's to 1970's, are now recognized to have substandard design details and is presenting a cause for major concern. This paper reports the results of a theoretical and experimental program investigating retrofit techniques for circular columns by encasing the critical regions with a steel jacket. The jacket is bonded to the column using grout. Results from six large-scale column tests show that the casing acts efficiently as confinement reinforcement enabling displacement ductility factor of greater than 6 to be achieved. The casing also inhibits bond failures at the laps of longitudinal reinforcement in the critical regions of the column by restraining the dilation and spalling of the cover concrete which degenerates into bond failure. Comparisons of 'as-built' and retrofitted columns are presented, and experimental strengths and ductilities are compared with analytical predictions.

INTRODUCTION

The 1971 San Fernando Earthquake caused substantial damage to a number of recently completed bridge structures and forced a reassessment of the design philosophy for bridges. Research was undertaken both in the U.S. and overseas to improve on the analytical techniques and to provide basic data on both the strength and deformation characteristics of lateral load resisting mechanism in bridges. In the U.S., research emphasis was primarily directed towards development of sophisticated time-history analysis techniques for bridges. Experimental research was mainly pursued as a means of verifying the analytical techniques.

Parallel to the analytical development in the U.S., a comprehensive research program pertaining to the strength and ductility of bridge columns was carried out at the University of Canterbury, New Zealand, under the sponsorship of the New Zealand National Roads Board. The research program produced detailed information on the flexural strength and ductility, and on the shear strength, of both reinforced concrete columns and steel-encased concrete piles. Particular emphasis was placed on quantifying the influence and effectiveness of lateral confining steel in the plastic hinge region of the column to increase ductility.

While basic research was being carried out, the California Department of Transportation (CalTrans) was making an initial impact on the difficult problem of improving the safety of older bridges. Although column failure was recognized as a major problem, the greatest risk was assessed to be due to inadequate connection between adjacent spans of the superstructure across movement joints. Consequently, a major retrofit program was undertaken by CalTrans to install restrainers across movement joints to reduce the risk of span collapse when excessive relative movement occurs. This retrofit program has recently been completed.

The recent shear failure in the columns of the I-5/I-605 Separator (a major freeway overpass) during the Whittier Earthquake of October 1, 1987 [1] and the tragic collapse of the Cypress Viaduct, and other bridge failures, during the Loma Prieta Earthquake of October 17, 1989 re-emphasized the inadequacies of the pre-1971 design and the urgent need in upgrading the seismic resistance of older bridge substructure.

The structural inadequacies inherent in many of the older bridge columns can be categorized as follows:

- **Inadequate Flexural Strength**
  Lateral force coefficients for seismic design were typically less than 10% in pre-1971 designs and are comparatively low by the current standard. Although the use of elastic design generally resulted in the actual flexural strength being significantly higher than that required by the assumed lateral load, low lateral flexural design strength results in high potential ductility demand in many cases.

- **Inadequate Flexural Ductility**
  Bridge columns designed before the 1971 San Fernando Earthquake typically contain insufficient transverse reinforcement. A common provision for both circular and rectangular columns involved the use of #4 (12.7 mm diameter) transverse peripheral hoops placed at 12 inches (305 mm) centers regardless of the column section dimensions. These hoops were often closed by lap splices in the cover concrete, instead of being lap welded or anchored by bending back into the core concrete. As a result, the ultimate curvature developed within the potential plastic hinge region is limited by the strain at which the cover concrete begins to spall which is typically in the range of 0.005 strain. At higher longitudinal strains the hoop steel unravels and the meager amount of confinement provided by the hoops becomes ineffective.

- **Undependable Flexural Capacity**
  In many of the tall bridges designed using the pre-1971 guideline, the column longitudinal reinforcement was spliced with starter bars extending from the footing with a lap length of 20 times the bar diameter. This lap length is insufficient for developing the yield strength of the longitudinal bars especially when large diameter bars are involved. As a consequence flexural strength degrades rapidly under cyclic loading. Occasionally the column longitudinal reinforcement was extended straight into the footing or pile cap without 90 degree hooks. Such details

Department of Applied Mechanics and Engineering Sciences
University of California-San Diego, La Jolla, CA 92093-0411.

allow pulling out of column reinforcement when subjected to large intensity seismic load reversals [2].

- **Inadequate Shear Strength**
  Conservative flexural design, using elastic methods coupled with less conservative shear strength provisions of the 1950's and 1960's, typically result in actual flexural strength of short columns exceeding their actual shear strength. Inadequate anchorage of the transverse reinforcement in the cover concrete compounded the problem. As a consequence, the probable failure mode for shorter columns involves brittle shear failure with low ductility and energy absorption characteristics.

- **Footing Failures**
  Pile caps and footing in older bridges are often provided with only a horizontal layer of reinforcement in the bottom region of the member. Top steel and shear reinforcement were considered unnecessary and routinely omitted. Such practice may be attributed to the use of elastic design which assumes full gravity load acting during the seismic event while concurrently prescribing unrealistically low values of lateral seismic forces.

- **Joint Failures**
  Joint regions either between column and footing or between column and bent-cap beams are subjected to very high shear stresses during a severe seismic attack. These regions traditionally have not been designed to resist this high level of seismic shear stresses.

Although the above design deficiencies have been rectified in current seismic codes and should no longer affect new bridge design, the condition of many old bridge columns built before the 1970's is a cause for major concern. This paper describes the initial phase of a research program funded by CalTrans, and the Federal Highway Administration on flexural strength and ductility of bridge columns and on developing retrofit techniques for upgrading the seismic performance of existing bridges. Experimental testing is being carried out at the Large-Scale Structural Testing Facility at the University of California, San Diego.

CONFINEMENT BY STEEL JACKETING

Recent research [3] has established that closely spaced lateral confinement reinforcement in the potential plastic hinge regions increases both the compressive strength and the effective ultimate compressive strain in the core concrete. The ultimate compressive strain increases from a value of about 0.005 in unconfined concrete to a value of 0.03 or higher in confined concrete. The increase in ultimate compressive strain significantly enhances the ductility capacity of the concrete section. Provided that the transverse reinforcement is spaced no wider than 6 times the longitudinal bar diameter and is properly anchored by either lap welding or bending back into the core, the ultimate compressive strain in the concrete corresponds to longitudinal strain at fracture of the transverse reinforcement. A method for estimating the ultimate compressive strain of confined concrete by equating the strain energy capacity at fracture of the transverse reinforcement to the additional energy stored by the confined concrete above its unconfined state has been proposed [4]. The enhancement in compressive strength and strain due to confinement is illustrated in Fig.1.

Research results [5] have shown that columns designed with reasonable volumetric ratios of confinement reinforcement ($0.005 \le \rho_s \le 0.03$) can develop stable hysteresis loops during inelastic cycling to displacement ductilities exceeding $\mu = 6$. In columns where axial loads are high, significant enhancement in flexural strength can occur. Although it is technically feasible to place external hoops on existing circular columns which would later be lap welded and sprayed with gunite to ensure rigid connection with the existing concrete, the method would be costly and may be aesthetically unacceptable. Confinement by enclosing the potential plastic regions of circular columns with a site-welded cylindrical steel sleeve or jacket would be notably less expensive and contributes minimal visual impact. The jacket is introduced slightly oversize for ease of construction and the gap between the column and jacket is filled with a cement-based grout. The jacket is terminated about 2 inches (50 mm) above the critical section at the column base to avoid additional strength enhancement resulting from end bearing of the sleeve on the footing when in compression. Significant flexural strength enhancement can however be expected since an increase in concrete compressive strength will result from the confining action of the steel jacket. The behavior of jacketed columns is expected to be similar to that of the steel-encased concrete piles which have shown experimentally to possess remarkable ductilities in severe cyclic load tests [6].

The confining action of the steel jacket is illustrated in Fig.2. Under the combined effect of axial compression and column flexure, the compression zone attempts to dilate as the flexural strength of the member is approached. The dilation is restrained by the radial stiffness of the jacket; placing the jacket in circumferential tension and the concrete in radial compression. Ignoring any contribution from existing hoops to the confinement of concrete core, the radial confining stress at yield of the steel casing is given by:

$$f_l' = \frac{2f_{yj}t_j}{D_j - 2t_j} \tag{1}$$

$$= \frac{1}{2} f_{yj}\,\rho_{sj} \tag{2}$$

where

$$\rho_{sj} \equiv \frac{4t_j}{(D_j - 2t_j)} \tag{3}$$

represents the volumetric confinement ratio of steel jacket. The variables f_{yj}, t_j and D_j are the yield strength, thickness and outside diameter of the steel jacket respectively. Hence for a column 60 inches (1524 mm) in diameter and retrofitted with a 0.5 inch (12.7 mm) thick A36 steel jacket (f_{yj} = 36 ksi or 248 MPa), the equivalent volumetric confinement ratio ρ_{sj} would be 0.0328 when 0.5 inch gap is used for grout. Thus the maximum confining stress f_L' due to steel casing would be 590 psi (4.07 MPa). The level of confinement provided by the steel jacket would correspond to the upper

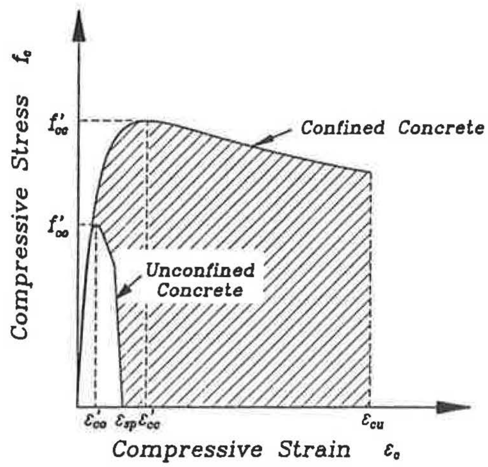


Fig. 1 Stress-Strain Model for Confined Concrete

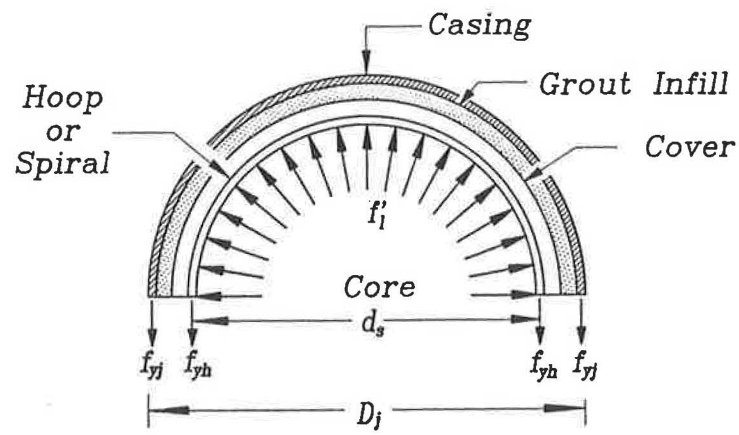


Fig. 2 Confining Action of Steel Casing

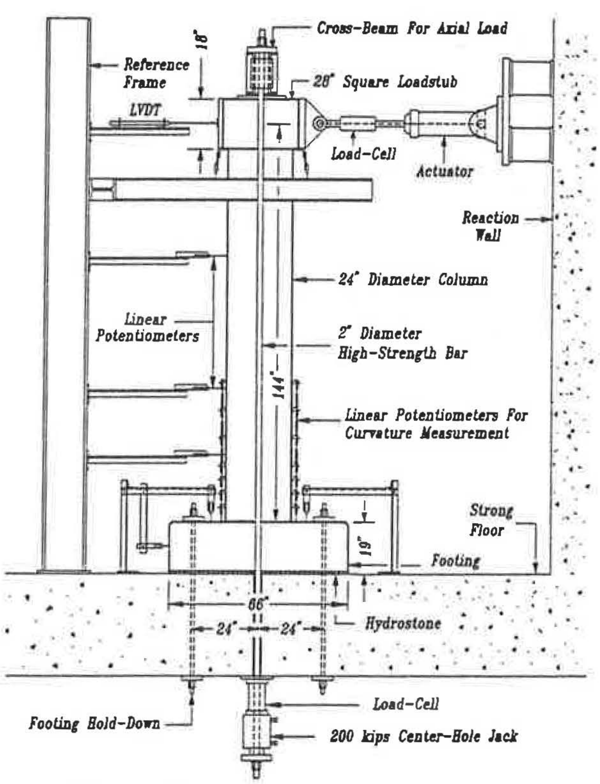


Fig. 3 Experimental Test Setup

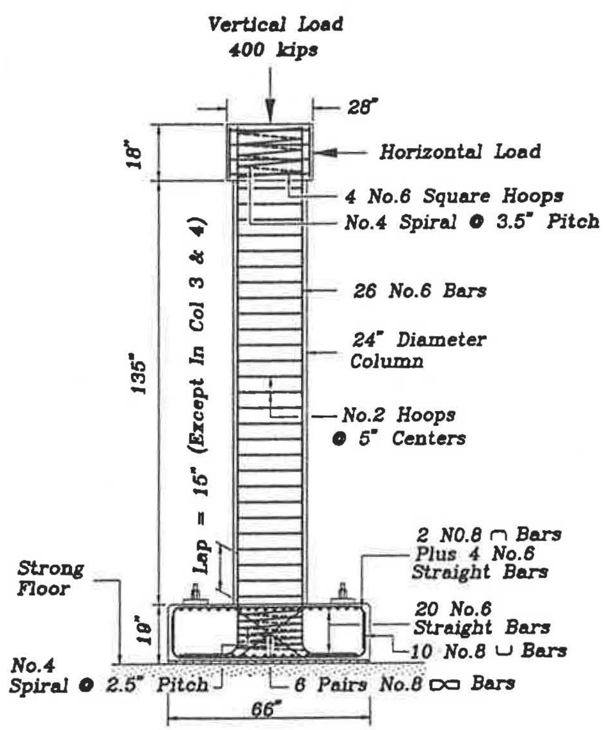


Fig. 4 Typical Reinforcement Details

limit of confinement provided by hoop reinforcement in current column design.

It should be clear that the steel jacket would also be effective in resisting a portion of the total column shear force. Analogous to a circular column having transverse reinforcement of either hoop or spiral, the contribution to shear strength from the steel casing, assuming an equivalent 45 degree truss mechanism, can be shown to be:

$$V_{sj} = \frac{\pi}{2} f_{yj} t_j (D_j - t_j) \tag{4}$$

It may also be expected that the lateral confining pressure from the casing would improve the bond transfer at the lap splices of column longitudinal reinforcement, possibly inhibiting bond failure in the potential plastic hinge region.

CIRCULAR COLUMN TESTS

In order to investigate the expected improved performance of the columns from steel jacketing, six large scale column models of 24 inches (610 mm) diameter and 12 ft. (3.657 m) height were recently tested using the test configuration shown in Fig.3. The columns were considered to be 0.4 scale models of a prototype 60 inches (1524 mm) diameter bridge column. The test columns were constructed with a footing to allow foundation influence or interaction to be monitored. A target concrete compressive strength of $f_c' = 5000$ psi (34.5 MPa) at 28 days was used to represent a 67% overstrength when compared to the typical 1960's design strength of 3000 psi (20.7 MPa). The overstrength is to reflect both the conservative concrete mix design and batching practices of the 1960's and the strength gain that has occurred in more than twenty years of natural aging. A vertical load of 400 kips was applied to the test column which corresponds to an axial load ratio of $0.18 f_c' A_g$ where Ag denotes the gross sectional area. Even though the axial load could not be kept constant during lateral displacement of the column, the variation of axial load is within ± 17 %.

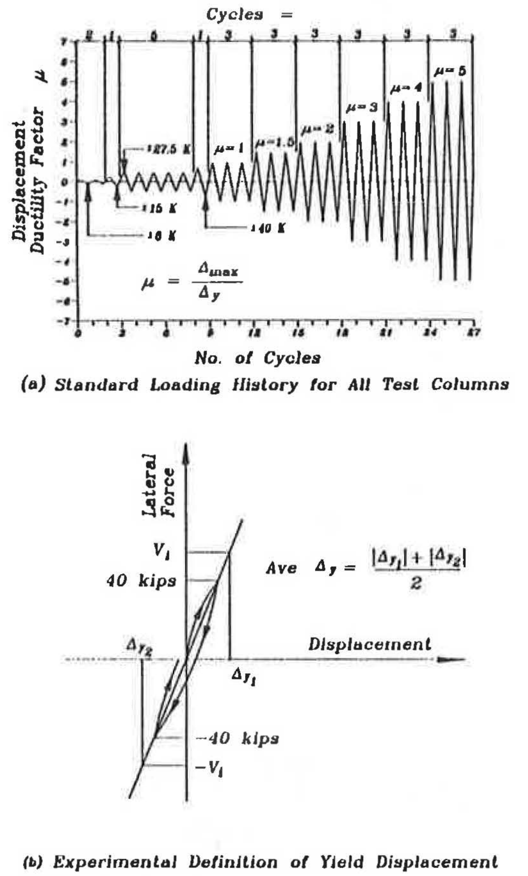
Longitudinal reinforcement for the column consisted of 26 #6 Grade 40 deformed bars; thus representing a longitudinal steel content of 2.53%. Yield strength for the #6 bar averaged 45.7 ksi (315 MPa). Transverse reinforcement consisted of circular hoops (#2 Grade 40 plain bars) placed at 5 inches (127 mm) centers uniformly up the column. The corresponding confining steel ratio is 0.18%. The hoops were spliced in the cover concrete with a lap length of 12 inches (305 mm). Typical reinforcement details for a test column is shown in Fig.4. The design represented a 60 inches (1524 mm) diameter prototype column reinforced with 32 #14 longitudinal bars and #4 circular hoops at 12 inches (305 mm) centers.

Design variations between columns are summarized in Table 1. Column 1, 2, 5 and 6 were built with lap splices of 20 times the longitudinal bar diameter in the potential plastic hinge region. Column 3 and 4 were reinforced with continuous longitudinal bars which were anchored with 90 degree hooks in the footing. Steel jackets for the columns were fabricated from 3/16 inch (4.76 mm) thick A36 hot-rolled steel. A 1/4 inch (6.35 mm) gap was provided between the column and jacket and was pressure-injected with water/cement grout. Typical compressive strength of 2 inch (51 mm) diameter grout cylinder was between 2000 and 2500 psi (14 and 17 MPa) at an age of 14 days. To ensure that the jacket does not bear against the footing when in compression a vertical gap of 1 inch (25 mm) was provided between the jacket and footing. The length of jacket was chosen to be 48 inches (1219 mm) to ensure that the moment demand immediately above the jacket does not exceed 75% of the uncased flexural capacity. The first pair of columns were constructed with a 1960's footing design using only straight reinforcement (two orthogonal layers of 24 #6 bars each) in the bottom region of the footing. The footing was supported on 1 inch (25.4 mm) high rigid pile-blocks. A strong footing detail was used in the remaining four columns after footing shear failure was noted in column 2. Reinforcement for the strong footing was redesigned to include top and bottom layers of #8 bars bent at both ends, 6 pairs of #8 diagonal bars placed close to the column/footing joint and #4 spiral at 2.5 inch (64 mm) pitch within the joint. Instead of using rigid pile-blocks, the footings were uniformly supported on a thin layer of hydrostone and clamped against the test floor. A partial retrofit approach was undertaken in column 5 to limit the amount of enhancement in flexural capacity which was noted in columns 2, 4 and 6. A thin sheet of Styrofoam (1/4 inch or 6.35 mm thick) was added between the column and the grout infill to allow a controlled dilation of cover concrete at large lateral displacement. Complete loss of cover concrete was prohibited by the presence of steel casing. The program also investigates the possible use of steel jacket for post-earthquake repair of bridge columns. Column 1 was fitted with a steel jacket after initial test (indicated as 1-R in the text matrix) and retested using the same load history. Loose cover concrete around the splice region of the main reinforcement was removed before installing the steel jacket. In order to provide better seal against grouting pressure, the jacket was extended to the top of the footing without any vertical gap. The weak footing was strengthened by external prestressing to a total of 300 kips at mid-height of the footing and in the direction of lateral load. Instead of being supported on pile blocks, the repaired column is placed on uniform bearing similar to the strong footing setup.

Table 1: Test Matrix

Test Unit	Column & Footing Details		Remarks
1	$20d_b$ Lap For Long. Bars Without Steel Casing	Weak Footing	Reference
2	$20d_b$ Lap For Long. Bars With Steel Casing	Weak Footing	Full Retrofit
3	Continuous Column Bars Without Steel Casing	Strong Footing	Reference
4	Continuous Column Bars With Steel Casing	Strong Footing	Full Retrofit
5	$20d_b$ Lap For Long. Bars 1/4" Styrofoam Wrap	Strong Footing	Partial Retrofit
6	$20d_b$ Lap For Long. Bars With Steel Casing	Strong Footing	Full Retrofit
1 - R	$20d_b$ Lap For Long. Bars Repaired By Steel Casing	Weak Footing 300 kips Prestress	Repair

All test columns were subjected to the same lateral displacement pattern of increasing magnitude, as shown in Fig.5 (a). The experimental yield displacement Δ_y was determined by extrapolating a straight line from the origin through ± 40 kips which approximately corresponds to the theoretical first yield of extreme tension steel to the ideal capacity V_i which is calculated using the Mander's model for confined concrete [4]. As shown in Fig.5 (b), the average of the two displacements was adopted as the experimental yield displacement.



(a) Standard Loading History for All Test Columns

(b) Experimental Definition of Yield Displacement

FIGURE 5: Standard Load Pattern for Test Column

RESULTS

Columns with Lapped Starter Bars

Column 1 was observed to suffer early bond failure at the lap of longitudinal reinforcement. Rapid strength degradation occurred after displacement to ductility factor μ=1.5. Maximum lateral load of 49 kips (218 kN) was noted during the push cycle to μ = 1.5 and was 97% of the theoretical ultimate capacity V_u. Significant drop in lateral load to 83% of theoretical capacity occurred in the pull direction of the same cycle. Bond failure in the lap was initiated and caused serious strength.degradation under additional cycling (see Fig.6). The strength envelope is seen to degrade asymptotically after μ = 1.5 to the moment resisted purely by the axial load which is estimated to be 19 kips (85 kN). In comparison, column 2 allowed the theoretical flexural capacity to be achieved. There is a 10% increase in theoretical flexural capacity due to confining pressure from the steel jacket. Lateral stiffness of column 2 shows a 19% increase after retrofit. The lateral stiffness of the column is defined as the theoretical flexural capacity (without strain-hardening) divided by the experimental yield displacement. Hysteretic loops for the column were stable up to μ = 3 when footing failure occurred resulting in rapid drop in vertical load carrying capacity (see Fig.7). There was however no sign of bond failure in the lap splices of longitudinal reinforcement in column 2. Hysteretic loops for the pair of columns are shown in Fig.8.

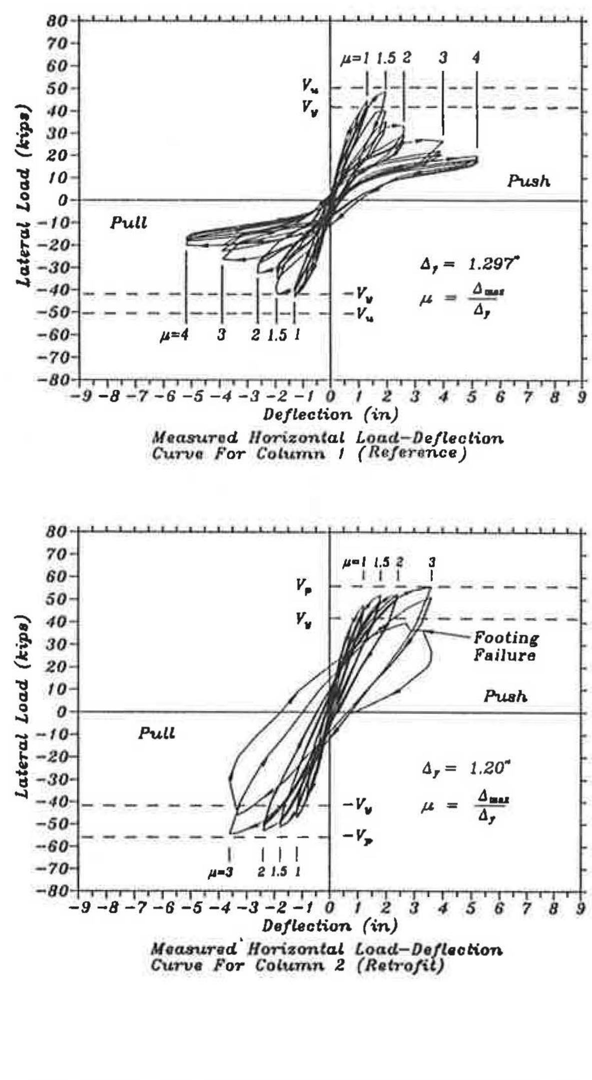


FIGURE 8: Hysteretic Response of Columns with Starter Bar and Weak Footing

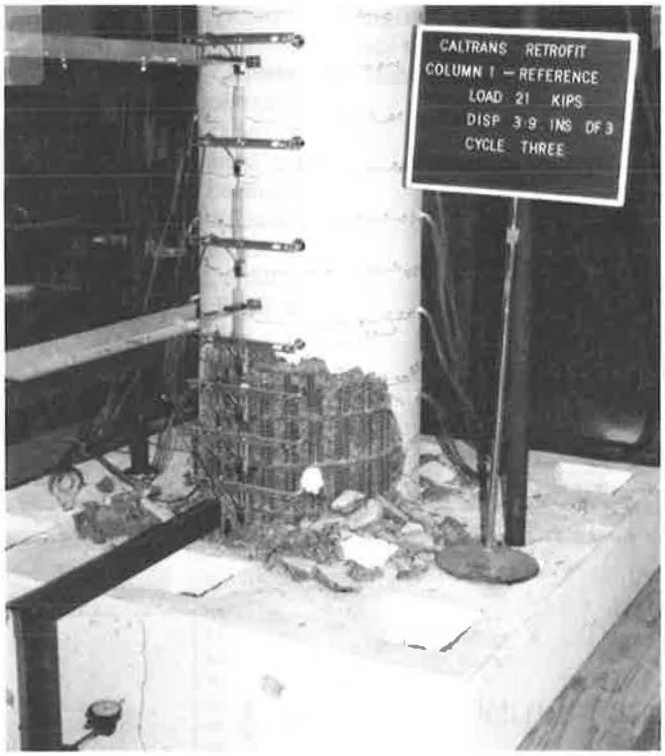


Figure 6: **Bond Failure in Column 1**

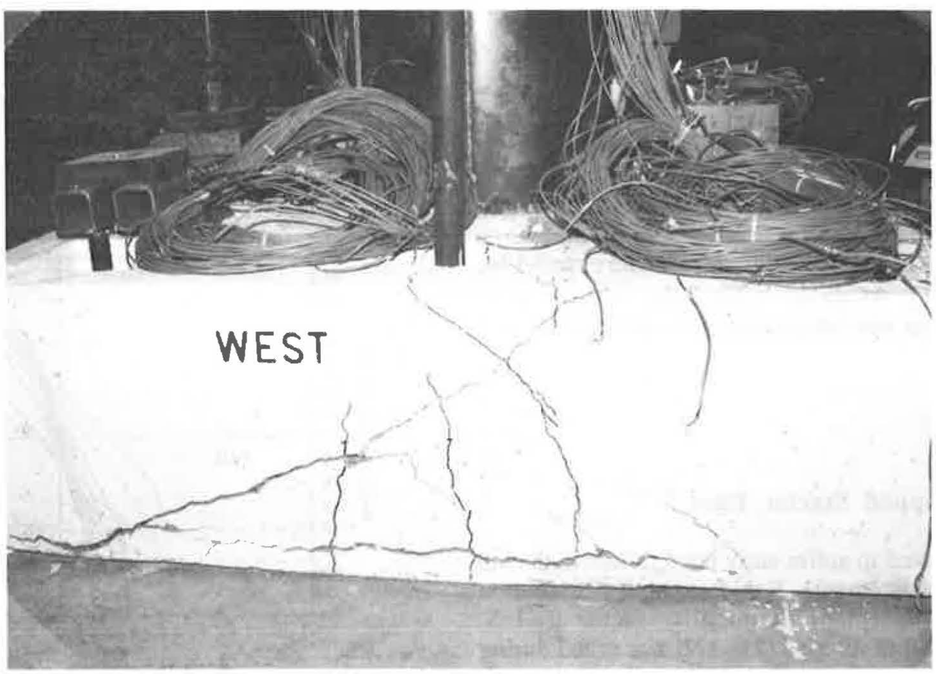


Figure 7: **Joint Shear Failure in Weak Footing - Column 2**

The weak retrofit approach adopted in column 5 uses a soft Styrofoam wrap as cushion to allow dilation of cover concrete and relative slip between main reinforcement and starter bars to occur. Without complete loss of cover concrete, strength degradation is not expected to be as rapid as the uncased column and vertical load carrying capacity can be maintained even after excursion to large lateral displacement. The response of column 5 and column 1 were very similar during the initial stages of loading (see Fig.9). Bond failure was again initiated at $\mu = 1.5$ but subsequent degradation of strength is comparatively more gradual. Theoretical capacity of column 5, as predicted without steel jacket, again could not be achieved.

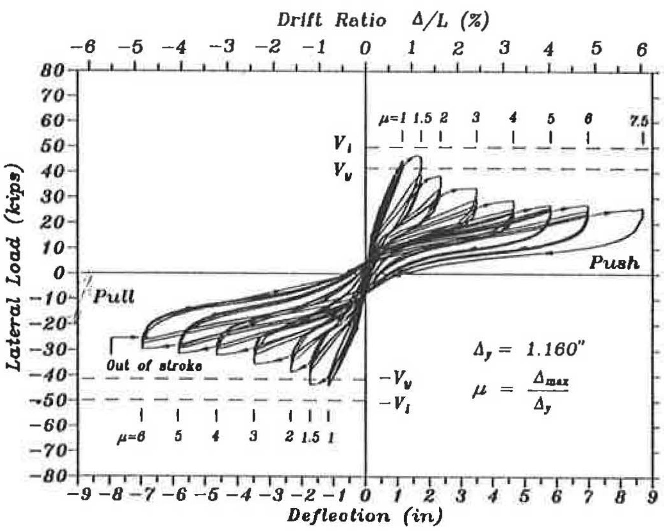
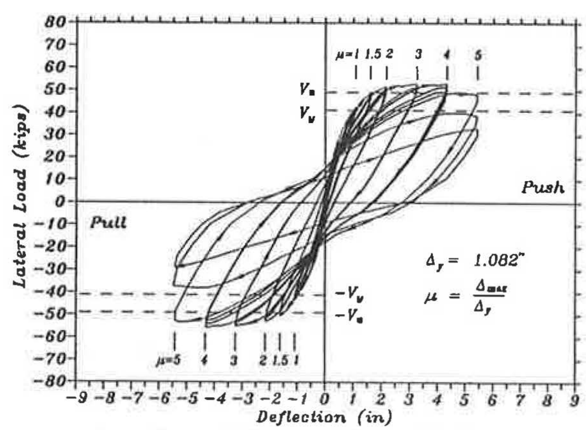


FIGURE 9: Hysteretic Response of Weak Retrofit Column

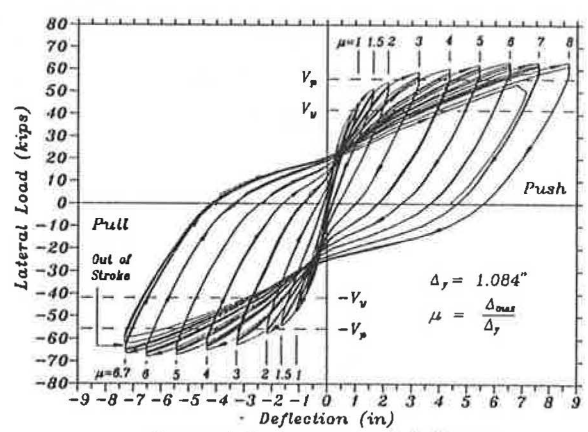
Column 6 was constructed and retrofitted with a steel jacket identical to column 2 except with a strong footing so that response at large displacement would be studied. Because columns 1 and 2 have been supported on 1 inch (25.4 mm) high pile-blocks, the compliance of the footing is reflected in the larger experimental yield displacement when compared with that of column 6 whose footing was placed in uniform bearing. The response of column 6 shown in Fig.10 is very similar to that of column 4 which was constructed with continuous reinforcement. Hysteretic loops for column 6 were stable up to $\mu = 7$ after which extreme tension reinforcement fractured due to low-cycle fatigue (see Fig.11). Bond failure which would otherwise prevail without retrofit was completely inhibited.

Columns with Continuous Reinforcement

The use of continuous reinforcement in column 3 gives a favorable increase in the displacement ductility capacity when compared to column 1. Experimental flexural capacity exceeded the theoretical capacity V_u by 6%. Very little degradation in flexural strength was noted between cycles of the same displacement magnitude except at $\mu = 5$ when failure was initiated by compression buckling of the longitudinal reinforcement. In comparison, column 4 showed significant increase in both the flexural strength and ductility. A maximum displacement ductility factor of 8 was observed in column 4 in the push direction. The displacement in the pull direction was limited by the travel in the actuator to $\mu = 6.7$. Failure of column 4 was caused by low-cycle fatigue fracture of the extreme tension reinforcement as with column 6. Theoretical capacity of $V_p = 55.6$ kips (247 kN) predicted using only the yield stress of the tension reinforcement significantly underestimated the ultimate flexural capacity of the column. Strain-hardening of longitudinal steel was estimated to have occurred at about $\mu = 3$. Theoretical strain-hardened flexural capacity of $V_{sh} = 70$ kips (311 kN calculated using an ultimate stress of 1.5 f_y in the longitudinal steel was only 4% larger than the observed maximum strength. Column 4 showed a 13% increase in the lateral stiffness over column 3. Hysteresis loops for column 3 and 4 are shown in Fig.12.



Measured Horizontal Load-Deflection Curve For Column 3 (Reference)



Measured Horizontal Load-Deflection Curve For Column 4 (Retrofit)

FIGURE 12: Hysteretic Responses of Columns with Continuous Reinforcement and Strong Footing

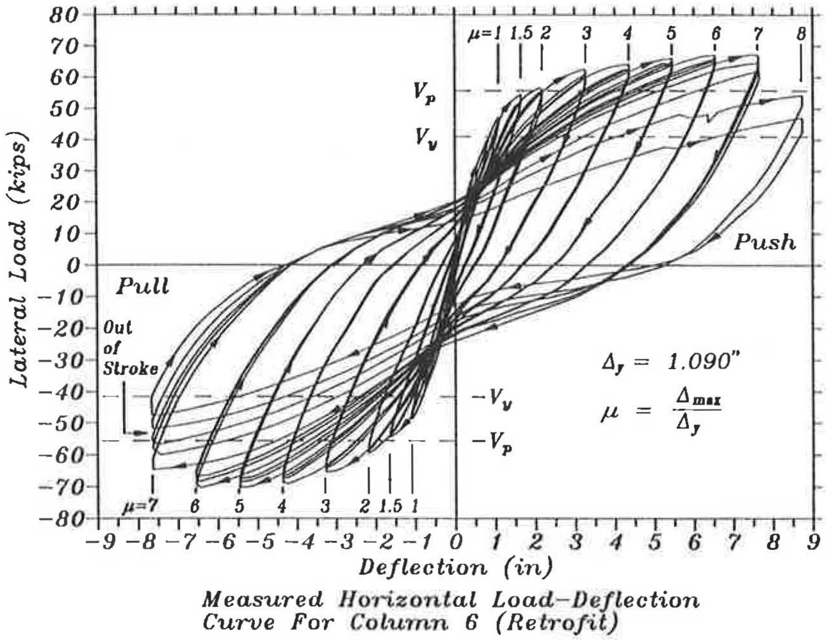


Figure 10: **Hysteretic Response of Column with Starter Bars and Strong Footing**

Figure 11: **Low-cycle Fatigue Fracture of Main Steel in Column 6**

Behavior of Repaired Column

Hysteretic response after repair of column 1 is shown in Fig.13. Compared to first testing without a steel jacket, significant improvement in the cyclic behaviors of repaired column was observed. Theoretical flexural capacity of the original column (50.6 kips or 225 kN) was exceeded by 6% at $\mu = 3$ after which gradual degradation of strength occurred due to development of bond slip in the lap of main reinforcement. There is no observed enhancement of flexural capacity due to strain-hardening of longitudinal steel as evident in the full retrofit case.

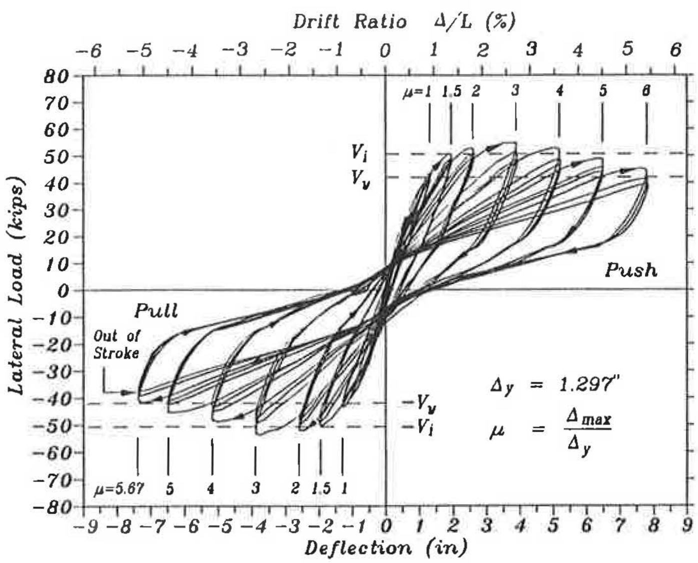


FIGURE.13: Hysteretic Response of Repaired Column

Conclusions

Experimental verification of strength and ductility enhancement by use of steel jacket is presented in the paper. A lap length of 20 times the longitudinal bar diameter is insufficient to develop yield stress of the longitudinal bar. Rapid strength degradation due to bond failure can be expected in these type of columns without retrofit. The introduction of fully grouted steel jacket as a retrofit measure is effective in providing confinement to the concrete. The cover concrete is completely contained within the steel casing eliminating bond failure. A ductile mode of flexural failure with good energy dissipation can be achieved by steel jacketing. Lateral stiffness increase due to fully grouted steel jacket is in the range of 10 to 20%. Due considerations must however be made to ensure comparable footing strength is available when the columns have been retrofitted with steel jackets. Full grouting of the jacket encourages penetration of large inelastic strain into the footing which can cause brittle footing shear failure. Post-earthquake repair of bridge columns using steel jackets can be expected to restore or even improve the seismic performance of columns with lap-splice details.

Acknowledgments

The research described in this paper has been funded by the California Department of Transportation and the Federal Highway Administration under Grants RTA599267 and F885D06. The helpful comments of Guy Mancarti and James Gates are gratefully acknowledged. The comments and conclusions in this paper are those of the authors alone and do not necessarily reflect the views of CalTrans or the FHA.

References

1. Priestley, M.J.N., *"Damage of the I-5/I-605 Separator in the Whittier Earthquake of October 1987,"* Earthquake Spectra, 4 (2): 389-405, 1988.
2. Fung, G.G., R.J. Lebeau, E.D. Klein, J. Belvedere and A.F. Goldschmidt, *"Field Investigation of Bridge Damage in the San Fernando Earthquake,"* Technical Report, Bridge Department, Division of Highways, California Department of Transportation, Sacramento, California, 1971.
3. Mander, J.B., M.J.N. Priestley and R. Park, *"Observed Stress-Strain Behavior of Confined Concrete,"* Journal Struct. Div., ASCE, 114 (8): 1827-1849, August 1988.
4. Mander, J.B., M.J.N. Priestley and R. Park, *"Theoretical Stress-Strain Model for Confined Concrete,"* Journal Struct. Div., ASCE, 114 (8): 1804-1826, August 1988.
5. Priestley, M.J.N. and R. Park, *"Strength and Ductility of Bridge Substructures,"* Road Research Unit Bulletin 71, New Zealand National Roads Board, Wellington, New Zealand, 1984.
6. Park, R.J.T., *"Seismic Performance of Steel-Encased Concrete Piles,"* Research Report 87-5, University of Canterbury, Christchurch, New Zealand, May 1987.