# RESEARCH INTO SEISMIC RETROFIT OF REINFORCED CONCRETE BRIDGE COLUMNS

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## SUMMARY

Structural deficiencies in flexural and shear strength, and in ductility capacity of reinforced concrete columns of Californian bridges have required the development of effective and economic retrofit solutions. The paper describes relevant research at the University of California San Diego, and presents design recommendations based on rather extensive test results.

## INTRODUCTION

Deficiencies in expected seismic performance of California bridge columns designed in the 1950's and 1960's, when the great U.S. freeway expansion occurred, have been apparent since the San Fernando Earthquake of 1971 [4], which caused damage to, or collapse of, a large number of recently designed bridges. More recent experience, in the 1987 Whittier earthquake [8] and the 1989 Loma Prieta earthquake [2], have confirmed these deficiencies, and lent an urgency to the need for developing effective retrofit solutions.

Deficiencies in reinforced concrete column performance may be categorized as follows:

(1) **Inadequate flexural strength.** This is generally the result of short lap splices of longitudinal flexural reinforcement at the base of the column. Lap lengths were typically 20 bar diameters, even for the #14 (44.5 mm dia.) and #18 (57 mm dia.) bars commonly used for bridge column construction. Although design practice of the 1950's and 1960's specified lateral force coefficients of about 0.06g, strength of columns without lap splices is generally adequate, since very conservative elastic design procedures were adopted. Typical flexural strength is about 0.25g, unless degraded by lap slices or other substandard detailing.

(2) **Flexural ductility.** Transverse reinforcement of older bridge columns is almost universally 12.7 mm dia. bars at 305 mm centres, regardless of section shape or size. Normally only peripheral ties or hoops were provided, and ends were anchored by lap splices in the cover concrete. The available ductility is therefore limited by that appropriate for unconfined sections with unrestrained compression reinforcement.

(3) **Shear strength.** The reinforcement details described above clearly provide little shear capacity in plastic hinge regions. In shorter columns flexurally restrained at each end, it is common to find predicted shear strength to be much less than predicted flexural strength, creating a propensity for brittle shear failure.

(4) **Joint shear strength.** The joint regions between column and footing at the base, and column and cap beam at the top, have traditionally been ignored in the design process. Performance in recent earthquakes show these regions to be particularly susceptible to brittle failure.

(5) **Anchorage.** Development lengths of column reinforcement in supporting members typically do not satisfy current code requirements, particularly at the column top. Although there have been some examples of anchorage failures, the problem is probably less critical than indicated by current assessment approaches.

A major research programme at the University of California San Diego, funded by the California Department of Transportation (CALTRANS) was started in 1987 to address the above problems. The project is currently active, and expected to continue at least until 1993. To date, extensive testing for flexural strength, shear strength and ductility has been carried out, with limited examination of joint problems. The research, and its findings to date, are briefly summarized in the following.

### Retrofitting for Flexural Strength and Ductility

Tests have been carried out on large-scale column models of both circular and rectangular section shape, to determine the characteristics of 'as-built' columns, and the effectiveness of various retrofit schemes. The main retrofit technique investigated has been the use of external steel jackets over the

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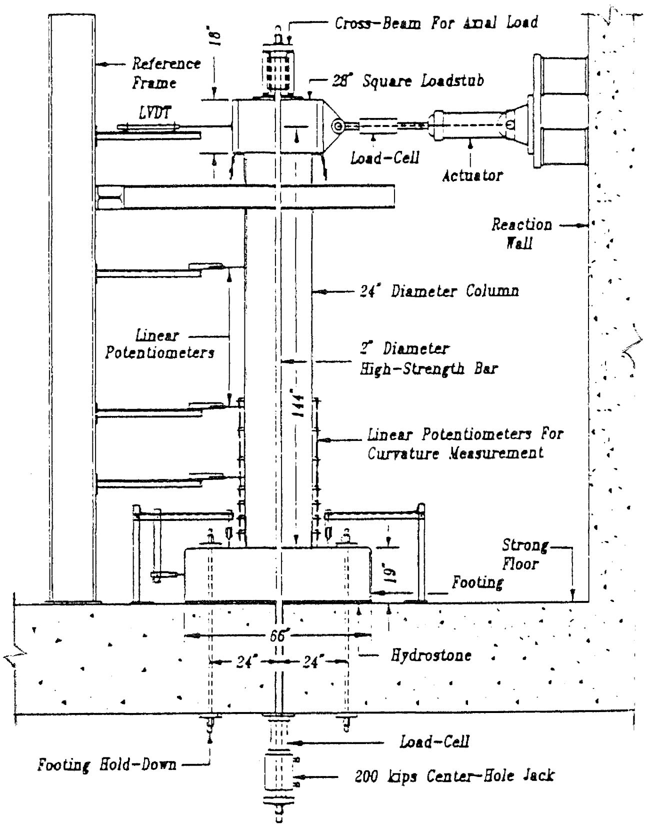



**Figure 1** **Test set up for flexural column tests ( 1 in. = 25.4 mm).**

potential plastic hinge region. For circular columns, the jacket is rolled from mild steel into two half shells to a radius typically 12 mm larger than the column radius (prototype scale). The steel jackets are positioned around the column and welded up the vertical seams, following which the gap between column and jacket is grouted with a pure cement grout. A 50 mm gap is left between the jacket and the supporting member (i.e., the footing or cap beam) in order to avoid the jacket acting as compression reinforcement at high ductilities, thus enhancing flexural strength and causing the potential for secondary problems.

The steel jacket is intended to act as a highly efficient form of confinement, restraining the longitudinal reinforcement from buckling, and enabling high concrete compression strain to be achieved. The concept was developed from research by Park et al.[6] on ductility of steel shell piles.

For rectangular columns, the jacket is rolled to an elliptical shape to provide continuous confinement. Plating with a rectangular jacket is comparatively ineffective, even when vertical and transverse stiffness are provided. The gap between the elliptical jacket and rectangular column is filled with ordinary concrete.

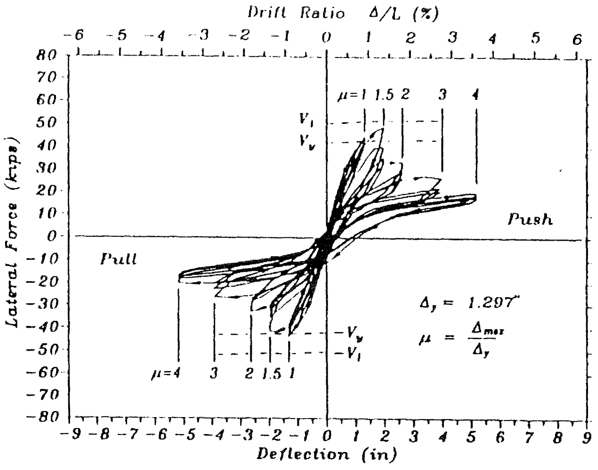
In both circular and rectangular columns, where only flexural action requires retrofit, the jacket is typically required to extend to the point where the moment has decreased to 75% of the value at the critical section.

Tests have been carried out on 3.66 m tall model columns with a section of 610 mm dia., or 711 mm x 480 mm rectangular section loaded axially to about 0.18 $f'_c A_g$, and loaded laterally as vertical cantilevers to gradually increasing displacements. Fig.1 shows the test set up.

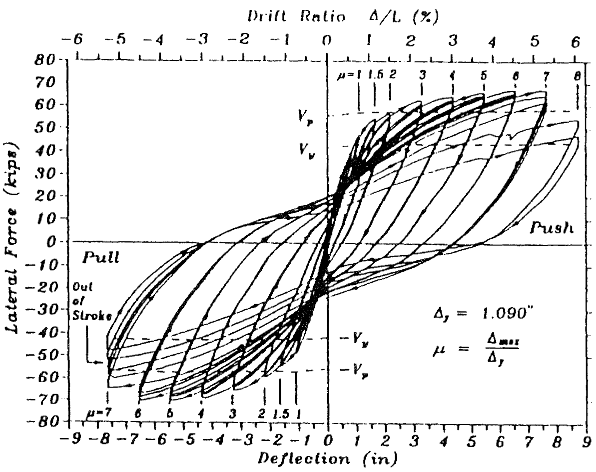
Vertical reinforcement for the column has been 19 mm grade 275 deformed bars, with a typical reinforcement ratio of $\rho_\ell$ = 0.025. Peripheral hoops or spirals of 6 mm bars at 125 mm centres are provided. Most columns have included 20 $d_b$ lap splices of longitudinal reinforcement at the column base, with the bar splices side by side, since this conditions is critical. The steel jackets have had a thickness of 4.8 mm, giving, for the circular columns, a diameter to thickness ratio of 127.

Fig.2 compares hysteresis loops for 'as built' and retrofitted circular columns, and for 'as built' and retrofitted rectangular columns tested in the strong direction. As built columns suffered bond failures at the lap splices and were unable to develop theoretical flexural strength, though strains approaching yield strain were recorded in the starter bars. Strength degraded rapidly to the value corresponding to flexural capacity provided by the axial load alone.

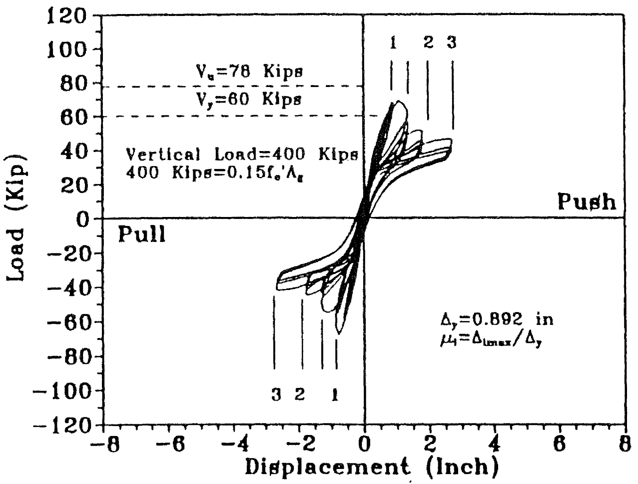
Columns retrofitted by steel jackets performed extremely well, with stable hysteresis loops being achieved at ductilities of the order of $\mu\Delta$=8. Considering the high aspect ratio of these columns, this performance is very satisfactory, corresponding to lateral drift ratios exceeding 5%. No bond degradation of the lap splices occurred, and final failure was by low cycle fatigue fracture of the longitudinal reinforcement. Performance was at least as good as reinforced concrete columns confined and constructed to current design requirements.



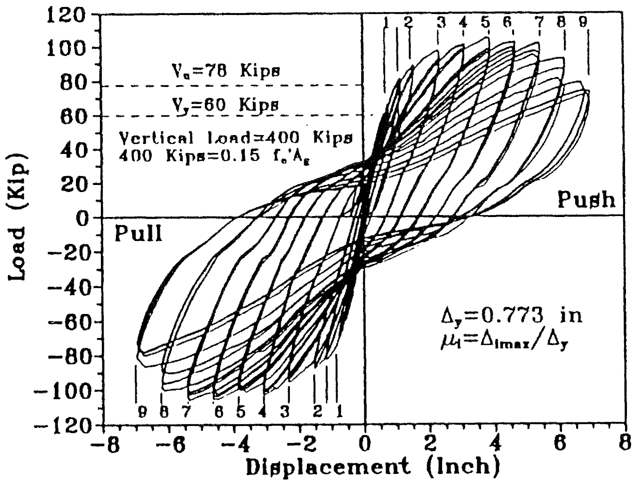

(a) Circular column, as built




(b) Circular column, retrofitted




(c) Rectangular column, as built




(d) Rectangular column, steel jacket retrofit.

**Figure 2** Lateral force-displacement hysteresis loops for flexural columns with lap-spliced longitudinal reinforcement (1 Kip = 4.45 kN, 1 in. = 25.4 mm).

On the basis of these and similar tests involving other retrofit solutions, described subsequently, the performance of as built and retrofitted lap-spliced columns can be explained with reference to Fig.3. For splice failure to occur, vertical cracks between starters and continuous bars, and a peripheral crack inside the plane of the bars must develop [9], as shown in Fig.3.

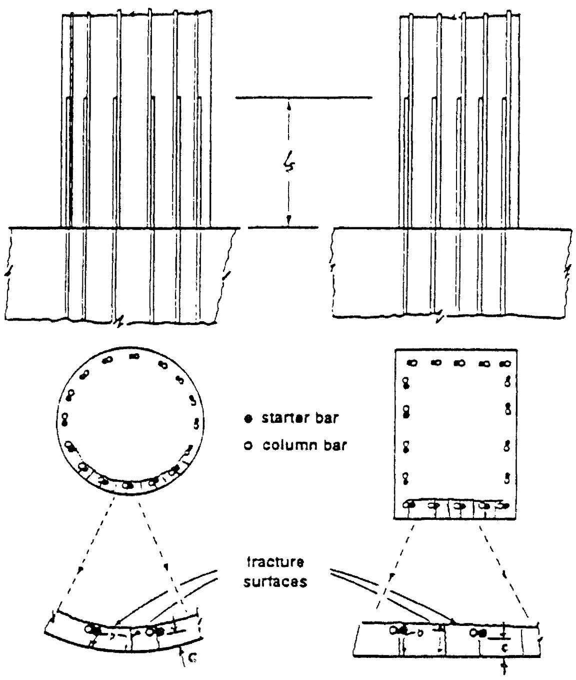
The strength of the lap splice may be found from the tension capacity of the concrete $f_t$, on the net crack area. Thus for a circular column, with n bars spliced around a diameter $D'$, and with splice length $\ell_s$, the tension force expected in a bar at splice failure is

$$T_b = f_s A_b = f_t \left[ \frac{\pi D'}{2n} + 2\,(d_b + c) \right] \ell_s \qquad (1)$$

where c is the cover, and $d_b$ the bar diameter.

Eqn.1 will only hold provided the concrete in the region of lap splice has not been subjected to longitudinal compression strains exceeding about 0.002, since vertical microcracking would develop, degrading the tension strength.

Provided adequate clamping pressure is provided across the potential failure cracks splice failure should be averted. The UCSD tests indicate that the radial dilation strain is also an important factor. A design requirement has been recommended that the lateral confining stress provided by the retrofit measure should be developed by a radial dilation strain not greater than $\xi_d$ = 0.001. By comparison with Eqn.1, allowing a coefficient of friction on the splitting surfaces of $\mu$ = 1.4, and requiring the lateral confining stress to provide adequate force to develop 1.4 $f_y$ (i.e. close to ultimate stress), the required confinement pressure is




(a) **Circular column** (b) **Rectangular column**

**Figure 3 Lap-splice failure mechanism**

$$f_\ell = \frac{A_b f_y}{\left[\frac{\pi D'}{2n} + 2(d_b + c)\right] \ell_s} \tag{2}$$

Data from the UCSD tests have been in close agreement with predictions from the above equations.

Steel jacket requirements to satisfy ductility requirements may be assessed by considering the jacket to be equivalent to continuous hoop reinforcement. Ultimate compression strains are calculated using the energy balance method of Mander et al. [5]. The action of the confining jacket tends to compress the length of the equivalent plastic hinge zone compared with conventional column designs, and a design value of

$$\ell_p = g + 0.044 f_y d_b \tag{3}$$

seems appropriate where g is the gap between the end of the confining jacket and the critical section (typically 50 mm), and $f_y$ and $d_b$ are the yield stress (MPa) and diameter of the longitudinal column reinforcement. Eqn.3 implies that plastic rotation capacity will be independent of column height.

The compressed plastic hinge results in significant increases in tension strains in longitudinal reinforcement compared with conventional ductile design, and ductility is generally limited by low cycle fatigue fracture of the reinforcement. Cumulative damage models [7] provide a reasonable method for assessing the appropriate effective ultimate tensile strain [3], but an approximate steel strain limit of 0.75 $\xi_{su}$ may be used, where $\xi_{su}$ is the strain at maximum stress.

### Retrofit for Shear Strength

The steel jacket retrofit concept has also proved to be highly effective in enhancing shear strength. Columns to the same section size as the flexural columns have been tested in double bending using the test set-up shown in Fig.4. "As-built" circular and rectangular columns have been tested to determine the contribution of concrete shear mechanisms to total shear strength, and to provide insight on the influence of flexural ductility on the strength of the concrete shear mechanism. Results from the tests have indicated shear capacities significantly exceeding those predicted by existing code shear strength equations, but in good agreement with predictions by a method developed by Ang et al. [1] based on cantilever shear tests. A modified form of Ang's equation for brittle shear failure strength has been proposed [9], namely

$$V_u = 0.29\sqrt{f_c'}\, A_e + V_s + 0.2P \tag{4}$$

where $V_s$, the truss mechanism component may be expressed for circular columns as

$$V_s = \frac{\pi}{2} A_{sp} f_{yh} \frac{D'}{s} \cot\theta \tag{5}$$

and P is the axial compression load on the column. In Eqn. 5, $\theta$ is the angle of the principal diagonal compression strut to the column axis, generally taken as 45° in design codes. However, provided longitudinal reinforcement is not terminated in the column midregion, a value of 30° seems appropriate from the UCSD tests.

Eqn. 4 only applies for sections with ductility demand less than $\mu_\Delta = 2$. In plastic hinge regions, the concrete component of shear strength is degraded linearly at ductilities higher than $\mu_\Delta$=2, to a base value of $V_c = 0.1\sqrt{f_c'}$ for $\mu_\Delta \geq 4$.

Fig.5 compares hysteresis loops for circular and rectangular shear columns 'as built' and retrofitted with a full height steel jacket. It will be seen that the steel jacket has successfully converted the 'as built' shear failure to ductile flexural response with extremely stable hysteresis loops to high displacement ductility levels.

### Alternative Column Retrofit Strategies

Two other retrofit strategies have been investigated for improving flexural and shear strength of circular columns [10]. The first involves active confinement provided by prestressing wire wrapped onto the column under tension while the second utilizes a fibreglass/epoxy jacket under hoop tension provided by grouting between the jacket and the column with grouting pressures as high as 1.75 MPa. This technique provides a measure of active confinement to the column, which can be supplemented by additional layers of unstressed fibreglass/epoxy in the most critical regions, such as the column end region.

Both techniques have proved to be highly effective for improving flexural response of circular columns with lapped starter bars. Only the fibreglass/epoxy semi-active jacket has been thus far tested for shear. It has performed as well as a steel jacket. Fig.6 shows hysteresis loops for circular flexural and shear columns retrofitted with fibreglass/epoxy jackets. Comparison with Fig.2 and Fig.5 shows equivalent response to columns retrofitted with steel jackets.

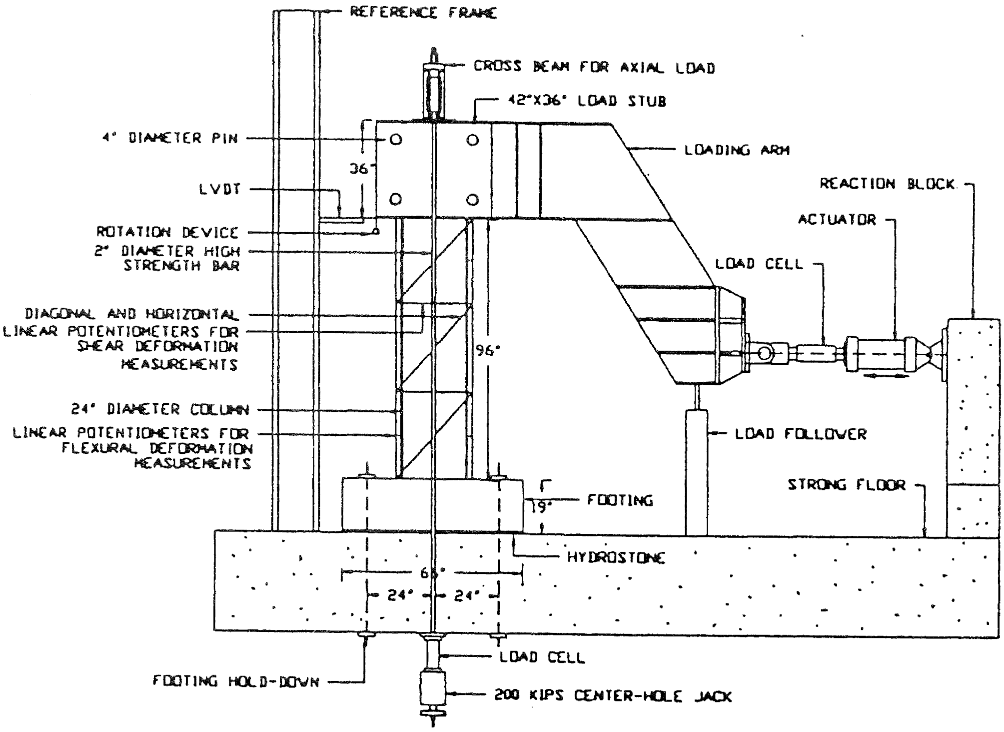



Figure 4 Shear test set-up (1 in. = 25.4 mm)

### Retrofit for Joint Shear Strength

The Loma Prieta earthquake emphasized the fact that bridge joint regions have traditionally received inadequate design attention. Fig.7 shows a test set up designed to provide loading to a column/cap beam knee joint assembly to model seismic actions under transverse response. The inclined actuators ensure that the beam and column in the joint region are subjected to the correct combination of axial force, bending moment and shear force. Fig.8a shows reinforcement details of a 1/3rd scale model of the joint region of a bent from the I-980 connector, which suffered a joint shear failure in the Loma Prieta earthquake. When tested, the model developed a very similar pattern of cracks to that observed in the prototype and suffered a joint shear failure with rapid strength degradation, as shown in Fig.8b.

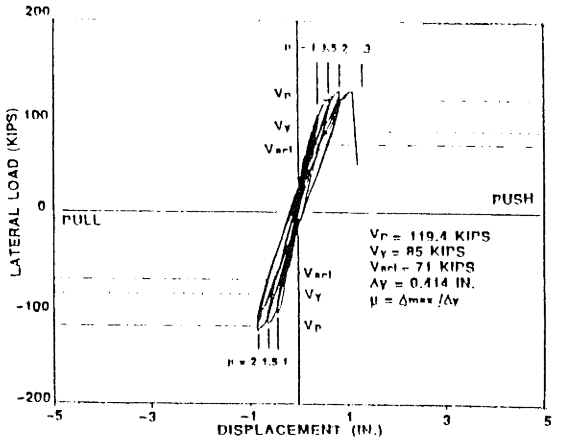
The concrete of the model joint region was removed, and a retrofit solution involving increased horizontal and vertical joint shear reinforcement, and increased joint width was implemented. Fig.8c shows a photograph of the joint reinforcement for the retrofitted model. The redesign of the joint was in accordance with principles familiar to New Zealand building designers, and incorporated in the N.Z. Concrete Design Code [11]. Under test, the retrofitted unit performed very well with the joint remaining elastic, and plastic hinges forming in the column under closing moment, and in the beam under opening moment. Hysteresis loops are shown in Fig. 8d. Final failure was by fracture of the beam bottom reinforcement under low cycle fatigue. That this occurred at a displacement ductility of only $\mu_\Delta = 4$ may be a cause for further concern.

The joint shear test programme has only just commenced at UCSD. Additional tests involving circular columns, and retrofit methods by external concrete jackets, rather than by complete joint replacement will be investigated.

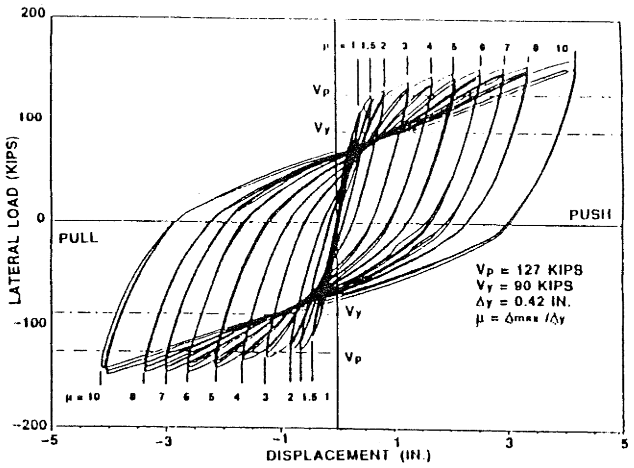
### Future Tests

Test programmes started later in 1991 to investigate retrofit methods for substandard footings, and for anchorage requirements for development of column reinforcement. In this context it is worth noting that confinement effects may produce greatly improved anchorage conditions than that used as the basis of code equations. For example the bottom steel bars, and the column bars of the knee joint in Fig.8c are anchored by straight extensions of rather short length into the joint. Despite this, and the cyclic nature of the load pattern, progressive bond failure did not occur, and the full tensile strength of the bottom reinforcement was developed.

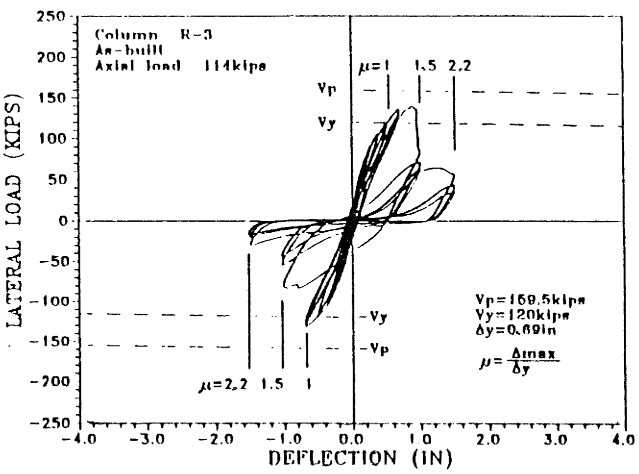
At the time of writing, a major test of a half scale model of a retrofit scheme for the San Francisco double deckers was in the final stage of preparation for testing. This model, weighting approximately 100 tonnes, is based on a retrofit concept developed at UCSD.



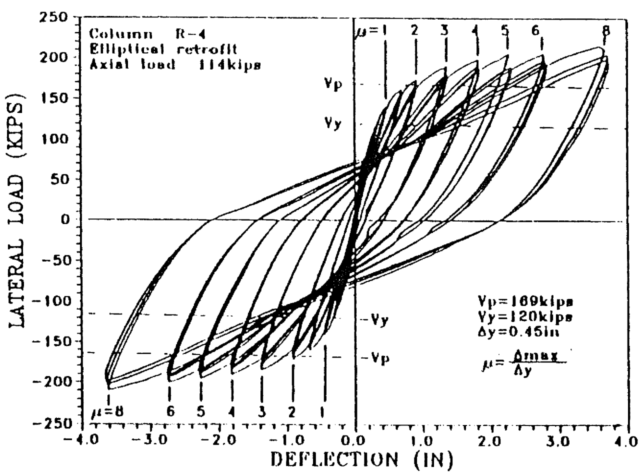

(a) Circular column, as built




(b) Circular column, retrofitted

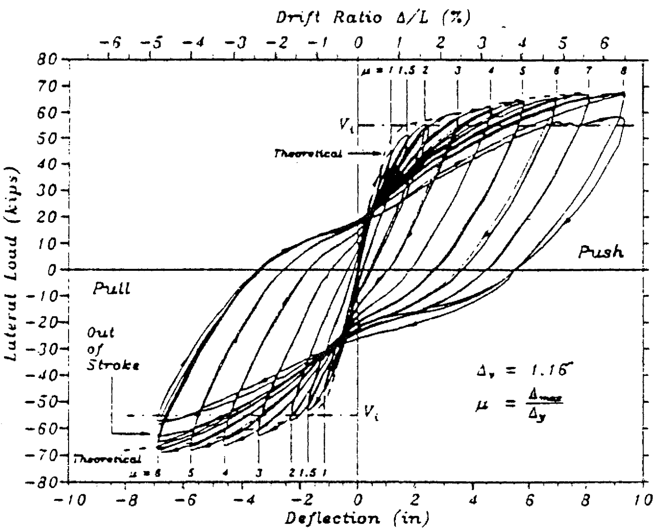



(c) Rectangular column, as built

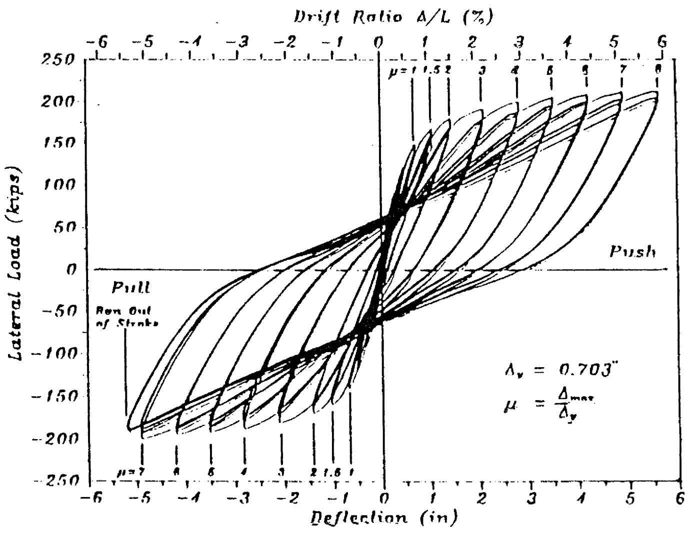



(d) Rectangular column, retrofitted

Figure 5 Lateral force-displacement hysteresis loops for shear columns (1 Kip = 4.45 kN, 1 in. = 25.4 mm)




(a) 'Flexural' column




(b) 'Shear' column

Figure 6 Lateral force-displacement hysteresis loops for circular columns with fibreglass/epoxy semi- active jacket retrofit.

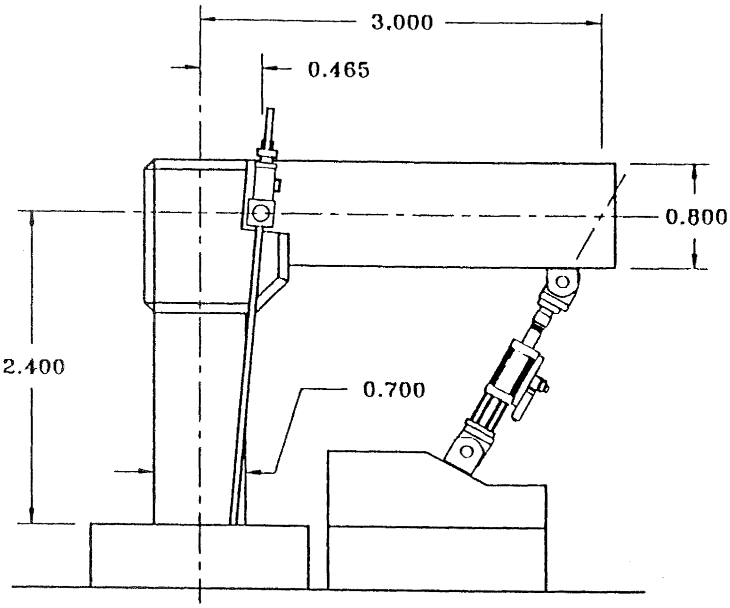
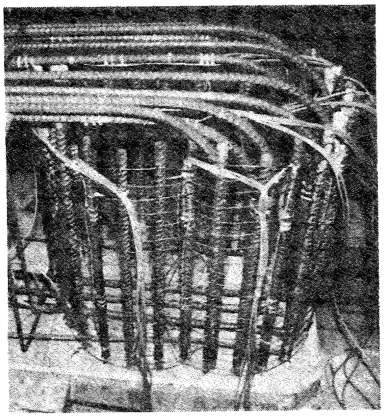
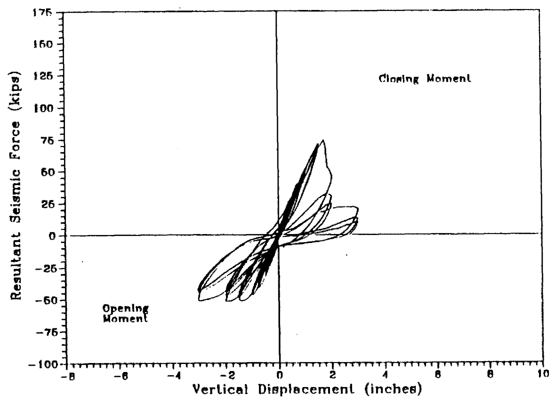


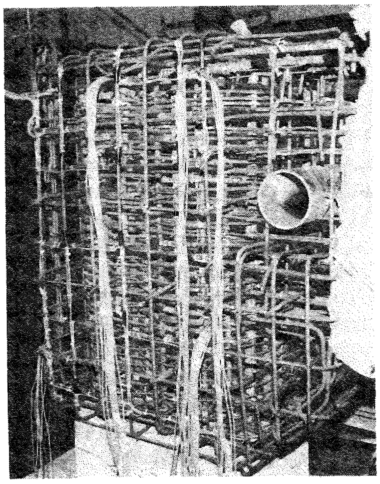

Figure 7 Knee joint test set up.



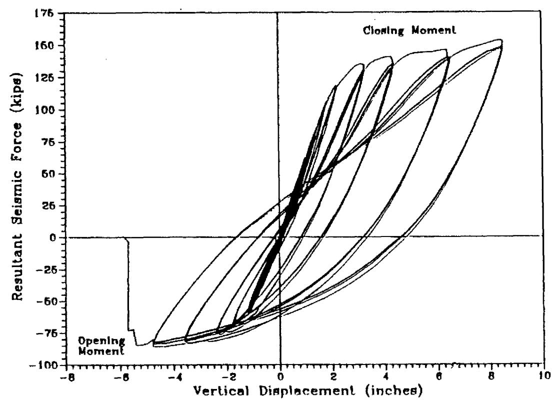
(a) As-built joint reinforcement




(b) As-built hysteresis loops



(c) Retrofitted joint reinforcement




(d) Retrofitted hysteresis loops

Figure 8 Testing of a knee joint model
(1 Kip = 4.45 kN, 1 in. = 25.4 mm)

## CONCLUSIONS

The realization that many of California's 24,000 bridges are incapable of surviving a major earthquake, has created a sense of urgency in the need to develop and test retrofit techniques for different structural inadequacies. Currently this research, centred on the facilities at UCSD, is about 60% complete, and has already produced retrofit methods and design approaches that have been implemented in the field. Design approaches have been verified for retrofitting to improve flexural strength of lap splices, flexural ductility, and shear strength of columns. Research is underway on retrofitting joint regions, and additional research on anchorage problems is planned in the near future.

## ACKNOWLEDGEMENTS

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## REFERENCES

1. Ang, B.G., M.J.N. Priestley and T. Paulay, "Seismic Shear Strength of Circular Reinforced Concrete Columns", ACI Structural Journal, Vol. 86, No.1, January-February 1989, pp. 45-60.

2. Buckle, I.G. et al, "Bridge Structures", Earthquake Spectra - Loma Prieta Earthquake Reconnaissance Report. Supplement to Vol.6, May 1990, pp. 151-187.

3. Chai, Y.H., M.J.N. Priestley and F. Seible, "Retrofit of Bridge Columns for Enhanced Seismic Performance", Proceedings of the U.S. - Japan Workshop on Retrofitting Bridges, December 15 - 22, 1990, Tsukuba, Japan.

4. Fung, G.G. et al., "Field Investigation of Bridge Damage in the San Fernando Earthquake", Technical Report, Bridge Department, Division of Highways, California Department of Transportation, Sacramento, California, 1971.

5. Mander, J.B., M.J.Nigel Priestley and R. Park, "Theoretical Stress-Strain Behaviour of Confined Concrete", ASCE Journal, Vol.114, No.8, August 1988, pp. 1804-1826.

6. Park, R.J.T., M.J.N. Priestley and W.R. Walpole, "The Seismic Performance of Steel Encase Reinforced Concrete Piles", Bull. NZNSEE, Vol. 16, No.2, June 1983, pp. 123-140.

7. Park, Y.J. and A.H.S. Ang, "Mechanistic Seismic Damage Model for Reinforced Concrete", Journal of Structural Division, ASCE, 111(4): 722-739, April 1985.

8. Priestley, M.J. Nigel, "Damage of the I-5/I-605 Separator in the Whittier Earthquake of October 1987", Earthquake Spectra, Vol.4, No.2 389-405.

9. Priestley, M.J.Nigel, "Seismic Assessment of Existing Concrete Bridges", in Seismic Assessment and Retrofit of Bridges. (Eds.) M.J. Nigel Priestley and F. Seible, Structural Systems Research Project, Report No. SSRP-91/03, University of California San Diego, July 1991, pp. 84-149.

10. Priestley, M.J.Nigel, and F. Seible, "Design of Seismic Retrofit of Bridges", in Seismic Assessment and Retrofit of Bridges, (Eds.) M.J.Nigel Priestley and F. Seible, Structural Systems Research Project, Report No.SSRP-91/03, Univers-ity of California San Diego, July 1991, pp. 197-250.

11. Standards Association of New Zealand, "Code of Practice for the Design of Concrete Structures", NZS3101:1982, S.A.N.Z., Wellington, 1982, 283 pp.