

RESEARCH REPORT

SEISMIC LOAD TESTS ON REINFORCED CONCRETE COLUMNS STRENGTHENED BY JACKETING

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STRENGTHENED BY JACKETING

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ABSTRACT

Four reinforced concrete column units were tested subjected to simulated seismic loading to investigate repair and strengthening techniques. The as-built columns were 350mm square and contained low quantities of transverse reinforcement as was typical of building columns designed and constructed in the pre-1970s. The column units represented the column region between the midheights of successive storeys. A stub was present at the midheight of each unit to represent a portion of the two-way beams and slab at the beam-column joint. Two column units were tested, repaired and strengthened by jacketing and retested. The other two column units were strengthened by jacketing and tested. The jacketing consisted of a 100mm (3.94 in) thickness of added reinforced concrete. The new longitudinal reinforcement was placed through the floor slab. Two arrangements of transverse reinforcement in the jacket were investigated. The as-built columns displayed low available ductility and significant degradation of strength during testing, whereas the jacketed columns behaved in a ductile manner with higher strength and much reduced strength degradation. The retrofit of columns using reinforced concrete jackets was found to be successful but labour intensive.

Keywords: building columns, concrete jacketing, ductility, flexural strength, reinforced concrete columns, retrofitting, seismic design.

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INTRODUCTION

Seismic design procedures have advanced considerably since about 1970. The main developments have been in the understanding of the non-linear dynamic response of structures, the introduction of capacity design procedures, and the methods for detailing reinforcement in concrete structures to achieve the ductile behaviour necessary to survive severe earthquakes.

Retrofitting of structures has been undertaken in several earthquake-prone countries after structural damage caused by strong earthquakes^{1,2,3} or because existing structures were required to comply with more recent code provisions ^{4,5}. Deficiences often found in typical existing moment resisting frames are inadequate shear strength of beams, columns and beam-column joints, and inadequate flexural strength and ductility of columns^{6,7}.

Several techniques for the repair and strengthening of structural elements such as reinforced concrete columns have been suggested in the literature^{1,8,9,10}. However there has been limited guidance for designers. As a result the techniques have been used in earthquake-prone countries with design based mainly on engineering judgement^{4,5}. A review of the literature shows that experimental and analytical research is required to provide designers with information regarding the seismic behaviour of structures repaired and strengthened by different retrofit techniques.

One retrofit technique for buildings has involved the jacketing of columns. This approach has the advantage that the resulting increase in the lateral load resistance of the building is distributed throughout the structure, and therefore that new foundations, or significant strengthening of existing foundations, may be avoided¹⁰.

This paper reports the results of an experimental study of the improvement in seismic

behaviour of reinforced concrete columns repaired and/or strengthened by concrete jacketing. The as-built columns tested were typical of those constructed for buildings in the pre-1970s. The tests involved both the as-built columns and the columns strengthened by concrete jacketing with added longitudinal and transverse reinforcement.

DETAILS OF COLUMN TEST UNITS

The prototype column

A typical reinforced concrete column from the lower storey of a seven storey moment resisting frame constructed in New Zealand in late 1950s is shown in Fig. 1. The columns in some regions of this building have inadequate flexural strength to prevent plastic hinging of columns occurring during a severe earthquake. Also, the transverse reinforcement in the columns is inadequate for shear and confinement according to current seismic codes. Although built in New Zealand, the column in Fig.1 may be typical of many of this vintage constructed in other countries of the world.

The as-built Units S1 to S4

Four reinforced concrete column units, referred to as Units S1, S2, S3 and S4, were constructed at 7/8th scale to represent the as-built prototype prototype column shown in Fig. 1. The column cross sections were 350 mm (13.8 in) square and the column unit had a height of 3.3 m (10.8 ft), representing the column region between the midheights of successive storeys of the frame. Typical dimensions and reinforcing details of the as-built column units are shown in Fig. 2. Eight longitudinal bars were distributed evenly around the perimeter of the column cross section and the transverse reinforcement consisted of sets of overlaping square hoops as shown in Fig. 2. The stub at the mid-



Figure 1 - Dimensions and Reinforcement Details for Typical Column of a Moment Resisting Frame Designed in the Late 1950s.





height of the column units represented a small portion of the two-way beams and slab at the beam-column joint. The longitudinal and transverse reinforcement of the column units were plain round bars (typical of the 1950s) of Grade 300 ($f_y \ge 44$ ksi) steel and the concrete of column units was normal weight with a design concrete compressive strength f_c' of 20 MPa (2,900 psi). The concrete in the column units was cast with the columns in the horizontal position.

Table 1 lists details of the reinforcement for the as-built Units S1 to S4. Table 2 lists details of concrete compressive cylinder strengths and the axial load ratios applied to columns during testing. Table 1 shows that the quantities of confining reinforcement in the as-built column units were very low compared with the quantities required by the ACI code¹². However the ratio of the theoretical nominal shear strength of the as-built column units computed using the ACI code¹² approach to the shear force required to develop the theoretical nominal flexural strength was about 1.3.

The jacketed Units SS1 to SS4

The as-built Units S1 and S4 were first damaged by simulated seismic load testing and then jacketed to become Units SS1 and SS4, respectively. The as-built Units S2 and S3 were jacketed without first testing to become Units SS2 and SS3, respectively.

Units SS1 and SS2 had 100 mm (3.94 in) thick concrete jackets containing eight new longitudinal bars bundled into the corners of the jacket and new square hoops as shown in Fig.3a. Units SS3 and SS4 had 100 mm (3.94 in) thick concrete jackets containing twelve new longitudinal bars distributed around the perimeter of the cross section of the jacket and new sets of overlapping square and octagonal hoops as shown in Fig. 3b.

The longitudinal reinforcement in the jackets were deformed bars from Grade 430 (f,

Table 1. Details of Reinforcement in	Column	Test	Units
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		Longitudinal Reinforcement				Transverse Reinforcement			
Unit	Part of Column	d _b mm	f, MPa	Ρ t %	d _b mm	f _{yh} MPa	s _h ^(a) mm	ρ _s %	ρ _{s,code} (b)
\$1, \$2, \$3,\$4	As-built column	20	325	2.05	6	350	265	0.13	0.10
SS1	Column jacket ^(c)	16	502	1.36	10	340	95	0.35	0.38
SS2	Column jacket ^(c)	16	502	1.36	10	340	95	0.35	0.40
SS3	Column jacket (c)	12	491	1.28	10	330	72	0.82	1.00
SS4	Column jacket (c)	12	491	1.28	10	330	72	0.82	0.79

(a) Spacing in potential plastic region

(b) $\rho_{s,code}$ = quantity of transverse confining reinforcement required in potential plastic hinge regions by ACI Code¹².

(c) Units SS1 and SS4 were repaired and strengthened; Units SS2 and SS3 were strengthened.

Note: 1mm = 0.0394 in, 1 MPa = 145 psi.

Unit	Concrete Compressive cylinder strength at stage testing		$\frac{P}{f_c A_g}$
	Age, days	Strength f _c , MPa	
S 1	110	29.5	0.2
S4	104	25.9	0.2
SS1	152 ^(a)	32.9 ^(a)	0.1
SS2	75 ^(a)	34.0 ^(a)	0.1
SS3	77 ^(a)	19.4 ^(a)	0.1
SS4	30 ^(a)	25.2 ^(a)	0.1

Table 2. Compressive Strength of Concrete in Column Test Units

(a) Data for the concrete jacket of the column.

Note: 1MPa = 145 psi

 \geq 62 ksi) steel, and the transverse reinforcement were plain round bars from Grade 300 (f_y \geq 44 ksi) steel. The concrete used for the jackets of these column units was normal weight and was cast with the columns in the vertical position. Before placing the concrete jackets, the surface of the concrete of the as-built columns was lightly roughened to an amplitude of about 2 to 3mm (0.08 to 0.12 in) by chipping, and in the case of the previously damaged columns of Units S1 and S4 all loose concrete was removed. Fig. 4a, b and c show for Unit S1 the damaged region of the as-built column above the slab after the initial seismic load testing, the damaged region of the as-built column after removing the loose concrete, and the column with the new reinforcement before placing the concrete jacket, respectively. The final placement of concrete for the jacket around the column below the floor slab was through holes made in the slab through which the new longitudinal column bars passed. Care was needed to ensure that the concrete was adequately compacted and that the concrete in the jacket reached the underside of the siab.







(a) Damaged Region of As-Built Column After Testing

(b) Damaged Region of As-Built Column After Removing Loose Concrete



(c) New Reinforcement Placed Before Casting the Jacket



(d) Method of Attaching New Ties in Beam-Column Joint Region

Figure 4 - Initial Damage and Method of Repair and Strengthening of As-Built Unit S1

Tables 1 and 2 list for Units SS1 to SS4 the details of the reinforcement and the concrete compressive cylinder strengths. The axial load ratios applied during the tests are also shown. The design of longitudinal and transverse reinforcement in the concrete jackets complied with the requirements for ductile columns designed by capacity design for seismic loading according to the New Zealand concrete design code NZS 3101¹¹, except that in the columns of Units SS1 and SS2 the horizontal spacing of the tied longitudinal bars exceeded the code permitted maximum spacing of 200 mm (7.9 in).

Table 1 shows that the quantities of confining reinforcement in the jackets of Units SS1 to SS4 were generally less than that required by the ACI code¹². However the ratio of the theoretical nominal shear strength of the jacketed column units computed using the ACI code¹² approach to the shear force required to develop the theoretical nominal flexural strength was about 1.5 for Units SS1 and SS2 and about 2.7 for Units SS3 and SS4.

In the beam-column joint regions the longitudinal column bars in the jackets were laterally restrained by ties, which in the case of Units SS1 and SS2 were welded to bolts anchored in the concrete of the as-built units (see Fig.4d). In Units SS3 and SS4 the lateral restraint to the longitudinal column bars in the beam-column joint regions was applied more positively by hoops which were made up by bars passed through holes drilled horizontally through the concrete of the beams and welded in place to form hoops.

TESTING OF THE COLUMN UNITS

Simulated seismic loading

The column units were tested subjected to simulated seismic loading. Quasi-static

cyclic lateral loading H was applied to the stub at the midheight of the unit through a loading frame, and a universal testing machine was used to apply a constant axial compressive load P through steel rollers and end plates to each end of the column (see Fig. 5). For the as-built columns the axial compression load ratio $P/A_g f_c$ was equal to 0.2, which was typical of the lower storey columns of the building investigated. In the repaired and/or strengthened units this ratio was equal to about 0.1 (see Table 2).

In all tests a cycle of lateral loading to $0.75H_{ACI}$ was initially applied, where H_{ACI} is the calculated lateral load associated with the nominal theoretical flexural strength M_i being reached at the critical sections of the column, computed using the ACI¹² rectangular compressive stress block for the concrete with an extreme fibre concrete compressive strain of 0.003, the measured concrete compressive cylinder strength, a strength reduction factor ϕ of unity, and the measured stress-strain relationship for the longitudinal reinforcement. Note that the New Zealand concrete design code¹¹ uses the same assumptions for flexural strength calculations as the ACI code¹². The lateral displacement at first yield Δ_y was found from the stiffness at a lateral displacement Δ'_y measured at the central stub at $0.75H_{ACI}$ or first yielding of the longitudinal reinforcement, whichever was less, extrapolated linearly to H_{ACI}^{13} . This amounted to multiplying Δ'_y by 1.33 to obtain Δ_y . Because of the significant softening of the lateral load versus displacement relationship measured during the tests on the as-built Units S1 and S4, caused mainly by inadequate bond, Δ_y in these units was defined as the central stub displacement measured at 0.75H_{ACI}.

The applied cyclic loading in the inelastic range was displacement controlled. The column units were subjected to two loading cycles to each of $\mu_n = \pm 1, \pm 2, \pm 3, \pm 4$, etc, where μ_n is the nominal displacement ductility factor defined as Δ/Δ_y , where Δ is the lateral displacement of the central stub.







Figure 7 - Effect of Unsymmetrical Plastic Hinge Rotation on Displacements of Column Unit.

Instrumentation

The horizontal displacement and the rotation of the central stub were measured by a set of three linear potentiometers. Additionally, twelve pairs of potentiometers were used to measure the average section curvatures over 80 and 160 mm (3.15 and 6.3 in) gauge lengths in the plastic hinge regions adjacent to the central stub. The pairs of potentiometers in the gauge lengths immediately above and below the column stub were seated directly against the faces of the stub, and hence these readings included deformations of the column due to bond slip of the longitudinal reinforcement in the stub and yield penetration of this reinforcement into the stub.

Also, electrical resistance strain gauges were attached in pairs at various locations on the hoops and on the longitudinal reinforcement in the plastic hinge regions.

PERFORMANCE OF THE AS-BUILT COLUMN UNITS

Load versus displacement response

Fig. 6 shows the experimental lateral load versus lateral displacement hysteresis loops measured for Units S1 and S4 representing the as-built column. Also shown is the nominal ideal theoretical ultimate lateral load H_{ACI} calculated using the ACI code¹² approach previously described. This theoretical load is plotted as dashed lines which reduce with increase in displacement due to the P- Δ effect. Fig. 6 shows that unlike well confined columns, where significant flexural overstrengths have been measured¹⁴, the measured maximum moments of Units S1 and S4 were almost equal to the nominal theoretical strengths calculated including the P- Δ effect.

Fig. 6 shows that Units S1 and S4 demonstrated a significant reduction of strength after they reached the measured maximum moments which ocurred at μ_n equal to





Figure 6 - Measured Lateral Load Versus Lateral Displacement Hysteresis Loops for As-Built Column Units.

approximately 3. This was the stage of significant crushing of cover and core concrete. The damage was concentrated mainly in the plastic hinge region above the central stub in the case of Unit S1 (see Fig.4a) and in the plastic hinge regions above and below the central stub in the case of Unit S4.

Fig. 6 also shows values for the real displacement ductility factor μ_r , which includes the effect of the rotation of the central stub due to the plastic hinge deformations concentrating either above or below the stub. As is shown in Fig. 7, μ_r can be calculated as $\mu_r = (\Delta + \theta h)/\Delta_y$ where θ is the measured rotation of the stub and h is the distance from the centre of the stub to the pin at the end of the column. As is seen in Fig. 6, the value for μ_n of 3 for Units S1 and S4 corresponds to values of μ_r that ranged from 3.6 to 3.9.

A measure of the ductility of the column units is the available displacement ductility factor, μ_a , defined¹³ for four loading cycles (that is, eight loading runs) as

$$\mu_{a} = \Sigma \mu / 8 \tag{1}$$

where $\Sigma \mu$ is the cumulative displacement ductility factor for loading runs in which the lateral load did not reduce to less than 80% of the maximum applied lateral load. Using this definition, for Unit S1 there were four loading runs to $\mu_n = 2$ and four loading runs to $\mu_n = 3$, giving $\mu_a = (4x2+4x3)/8 = 2.5$. For Unit S4 there were four loading runs to $\mu_n = 2$ and three loading runs to $\mu_n = 3$, giving $\mu_a = (4x2+3x3)/8 = 2.1$.

The New Zealand concrete design code NZS 3101¹¹ specifies that structures with "adequate ductility" should reach a lateral displacement of at least 4 to 6 times the displacement at first yield during four loading cycles, without significant reduction in strength. It is evident that the measured available displacement ductility factors for Units S1 and S4 fell well short of those code specified values for ductile structures. Also, the measured ductilities for Units S1 and S4 fell short of those measured in previous research projects conducted at the University of Canterbury^{14,15} during quasi-static cyclic loading tests on reinforced concrete columns having low axial loads and containing greater quantities of transverse confining reinforcement.

The maximum interstorey drift reached by Units S1 and S4 before the lateral load reduced to less than 80% of the maximum applied lateral load was about 1.9%. The interstorey drift is obtained by dividing the interstorey horizontal displacement by the storey height. Interstorey drift has been suggested as a suitable index for the level of deformation imposed on test structures or structural subassemblages¹⁶. However caution must be adopted in the use of this index in tests because the imposed deformation needs to be related to the stiffness of the structure and the displacement ductility factor¹³. Noting that Units S1 and S4 were relatively flexible, the interstorey drift of 1.9% attained by these two units before substantial strength degradation is relatively small compared with the values of at least 2 to 3 % obtained in previous tests on stiffer reinforced concrete columns designed according to the New Zealand concrete design code¹¹ conducted at the University of Canterbury^{14,15,17}.

Measured strains and curvatures

Fig.8 shows the variations of longitudinal concrete strain on the surface of the core concrete in the plastic hinge regions of Unit S1 as calculated from potentiometer readings. Fig.9 shows the variation of longitudinal steel strain in the bars of Unit S1 as measured by electrical resistance strain gauges. The differences between the concrete and steel strains shown by the comparison of Figs. 8 and 9 is due to bond degradation



Figure 8 - Longitudinal Strains Measured on Core Concrete by Potentiometers on Unit S1.





Figure 9 - Longitudinal Strains Measured on Bars by Strain Gauges in Unit S1.



Figure 10 - Measured Curvature Profiles for Unit S1.



Figure 11 - Measured Lateral Load Versus Column Curvature Above Central Stub for Unit S1.



Figure 12 - Theoretical Lateral Load Versus Column Curvature Above Central Stub for Unit S1.



Figure 13 - Measured Strains on Transverse Reinforcement of Hoop Sets Nearest the Central Stub of Unit S1

occurring between the concrete and the plain round longitudinal bars during the tests. Similar trends in the measured strains were found for Unit S4.

Evidence of bond degradation is also given by the curvatures obtained from potentiometer readings for Units S1 shown in Fig.10. It can be seen that most of the inelastic curvature in Unit S1 was concentrated in a plastic hinge length of about $0.5h_c$, where h_c is the column depth. Similar results were found for Unit S4. This concentration of curvature was evidently due to bond degradation leading to one or two main cracks, particularly in the 80 mm (3.15 in) gauge length commencing at 160 mm (6.3 in) from the face of the stub.

The lateral load versus column curvature hysteresis loops measured for Unit S1 are shown in Fig.11, where the column curvatures were obtained from the above 80 mm (3.15 in) gauge length. For comparison, the lateral load versus column curvature response predicted by cyclic moment-curvature theory¹⁸ are plotted in Fig. 12. The comparison shows that the experimental flexural strength of Unit S1 was predicted reasonably well by the moment-curvature analysis, but that the experimental column curvatures were greatly underestimated. This would be because the theory did not include the effect of the bond degradation between the plain round bars and the concrete.

The measured averaged strains of the hoop sets nearest the central stub of Unit S1 (see Fig.2) are shown in Fig.13. The hoop strains in the damaged region seldom reached the yield strain, even in the final stages of testing. Unit S4 gave similar results.

PERFORMANCE OF THE REPAIRED AND/OR STRENGTHENED COLUMN UNITS Load versus displacement response

Fig.14 shows the experimental lateral load versus lateral displacement hysteresis loops for the retrofitted Units SS1 to SS4. The measured lateral load versus displacement hysteresis loops for the four jacketed columns indicated good energy dissipation and only a little reduction in strength up to the end of testing. It was observed that each as-built column behaved monolithically with its jacket during the tests. The increase in stiffness, strength and ductility of the jacketed columns can be observed by comparing Figs. 6 and 14. The comparison indicates, for example, that the strength and stiffness of the jacketed Unit SS1 were about three times those for the as-built Unit S1. The ductility achieved by the retrofitted columns was at a level satisfactory for ductile structures. The maximum interstory drifts reached was about 2.8%.

The experimental loops shown in Fig.14 for Units SS1 and SS4, which were damaged before retrofitting, are similar to those for Units SS2 and SS3, which were not damaged



Figure 14 - Measured Lateral Load Versus Lateral Displacement Hysteresis Loops for Jacketed Column Units



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Figure 14 - Measured Lateral Load Versus Lateral Displacement Hysteresis Loops for Jacketed Column Units (continued) before retrofitting. This suggests that the previous damage to the as-built columns of Units SS1 and SS4 did not significantly influence the performance of the retrofitted units.

Also, the experimental loops shown in Fig.14 for Units SS1 and SS2 which had longitudinal bars bundled into the corners of the jackets (see Fig.3a), and for Units SS3 and SS4 which had longitudinal bars distributed around the perimeter of the jacket (see Fig.3b), were not significantly different. Hence the bundling of bars in the corners of the jackets of Units SS1 and SS2 had no detrimental effect on the seismic performance of those units, in spite of the fact that these bundles were 438mm (17.2 in) apart and hence exceeded the permitted New Zealand code¹¹ maximum spacing between tied column bars of 200mm (7.9 in). Similar results for bundled column bars have also been found in cyclic lateral loading tests of jacketed reinforced concrete columns conducted at the University of Texas¹⁶.

In addition, these tests indicated that the greater quantity of transverse reinforcement in the jackets of Units SS3 and SS4 than in the jackets of Units SS1 and SS2 (see Table 1 and Fig.3) resulted in no significant improvement in the seismic behaviour of Units SS3 and SS4. That is, for these jacketed columns with relatively light axial loads of $0.1f_c^tA_g$ the lateral load tests showed that the quantity of confining reinforcement recommended by the ACI Code¹² is unnecessarily high. The conservative nature of the ACI recommended quantity of confining reinforcement for columns with small axial load levels has been discussed previously¹⁹.

In the test on Unit SS1 the bundled longitudinal bars buckled in the column above the central stub at a nominal displacement ductility factor μ_n of about 6 and eventually fractured (see Fig. 14a and 15a). Although most of the damage was concentrated above the central stub of this unit, some damage was also observed below the stub. The test on Unit SS2 was terminated when a significant reduction of lateral load capacity occurred as a result of buckling of the bundled longitudinal bars in the beamcolumn joint region at a μ_n of about 6 (see Fig. 14b). These bars lost their restraint against buckling when the anchor bolts, to which the ties in the jacket in the beamcolumn joint were welded, pulled out of the concrete of the as-built unit. The more reliable lateral restraint obtained from welded hoops passing through holes drilled in the beams of the beam-column joint region, used for Units SS3 and SS4, is therefore preferred.

The damage to Units SS3 and SS4 was concentrated in the columns below the beamcolumn joints and the longitudinal bars buckled and eventually some fractured at μ_n values of about 6 (see Fig.15b).



(a) Unit SS1







Figure 16 - Measured Curvature Profiles for Unit SS1.



Figure 17 - Measured Lateral Load Versus Column Curvature Above Central Stub for Unit SS1.



Figure 18 - Theoretical Lateral Load Versus Column Curvature Above Central Stub for Unit SS1.

Measured strains and curvatures

Curvatures obtained from potentiometer readings for Unit SS1 are shown plotted in Fig.16. Fig.17 shows the experimental hysteresis loops for lateral load versus column curvatures for Unit SS1, where the column curvatures were obtained from the readings of the potentiometers above the central stub seated directly against the face of the stub. The lateral load versus column curvatures predicted by the cyclic moment-curvature theory of Mander et al¹⁸ are plotted in Fig. 18. The cyclic moment-curvature theory included the effect of yield penetration of a $32\sqrt{d_b}$ length of longitudinal bar into the stub, where d_b is the diameter of the longitudinal bar in mm. Comparison of Figs. 17 and 18 shows that the flexural strength of Unit SS1 was predicted reasonably well by the theory, but that the theory underestimated the experimental curvatures. For more accurate results, bond slip in the column should also be considered in the theoretical predictions.

The measured steel strains on the instrumented hoop sets in the jackets of the plastic hinge regions of the four retrofitted columns indicated that these strains seldom reached the yield strain, even in the final stages of testing.

CONCLUSIONS

1. The seismic load tests on two column units, representing reinforced concrete columns designed and constructed in New Zealand in the 1950s, showed that columns designed to early seismic codes have very low available ductility. The columns tested were 350mm (13.8 in) square, contained plain round longitudinal bars, and had transverse reinforcement consisting of 6mm (0.24 in) diameter hoops at 265mm (10.4 in) centres. During quasi-static cyclic lateral loading tests, which simulated seismic loading, available displacement ductility factors of approximately 2 were found in these column units.

Evidence of bond degradation between the plain round longitudinal bars and the surrounding concrete was also observed, which resulted in a softening of the measured lateral load versus displacement relationships.

2. The two previously tested and damaged as-built column units were retrofitted by adding reinforced concrete jackets. In addition, two further as-built column units were retrofitted by adding reinforced concrete jackets without being first subjected to simulated seismic loading. The jackets consisted of 100mm (3.94 in) thick new concrete containing new longitudinal and transverse reinforcement. The surface of the as-built columns had been lightly roughened by chipping before the jackets were placed. The new longitudinal reinforcement was placed either bundled in the four corners of the jacket with single square hoops as transverse reinforcement, or distributed around the jacket with square and octagonal hoops as transverse reinforcement. In the first case the bundles of longitudinal bars were 438mm (17.2 in) apart, and in the second case the longitudinal bars were no more than 198mm (7.8 in) apart. Results of the simulated seismic load tests showed that the strength and stiffness of the jacketed columns were up to 3 times those of the as-built columns. During quasi-static cyclic lateral loading tests, with imposed nominal displacement ductility factors of up to 6, very good energy dissipation and only a small reduction in strength was observed. These tests also showed that the effect of previous damage to the as-built columns, and the two different reinforcing details used in the jacketed columns, had no significant influence on the overall seismic performance of the jacketed columns.

3. The results of this investigation indicate that jacketing with new reinforced concrete significantly improves the stiffness, strength and ductility of typical reinforced concrete columns constructed according to early seismic codes. However, as was found in the investigation, this technique of retrofitting is very labour intensive.

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REFERENCES

- "Repair and Strengthening of Reinforced Concrete, Stone and Brick-Masonry Buildings", Building Construction Under Seismic Conditions in the Balkan Regions, UNDP/UNIDO PROJECT RER/79/015, United Nations Industrial Development Organisation, Vienna, 1983, p231.
- Endo T et al ,"Practices of Seismic Retrofit of Existing Concrete Structures in Japan", Proceedings of the Eight World Conference on Earthquake Engineering, Vol 1, San Francisco, 1984, pp 469-476.
- Aguilar J, Juarez H, Ortega R, and Iglesias J, "The Mexico Earthquake of September 19, 1985. Statistics of Damage and of Retrofitting Techniques in Reinforced Concrete Buildings affected by the 1985 Earthquake", Earthquake Spectra, Vol 5, No 1, California, February 1989, pp 145-151.
- Sugano S, "Seismic Strengthening of Existing Reinforced Concrete Buildings in Japan", Bulletin New Zealand National Society for Earthquake Engineering, Vol 14, No 4, December 1981, pp 209-222.
- Jara M, Hernandez C, Garcia R, and Robles F. "The Mexico Earthquake of September 19, 1985. Typical Cases of Repair and Strengthening of Concrete Buildings", Earthquake Spectra, Vol 5, No 1, California, February 1989, pp 175-193.

- Brunsdon D R, and Priestley M J N, "Assessment of Seismic Performance Characteristics of Reinforced Concrete Buildings Constructed Between 1936 and 1975", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol 17, No 3, September 1984, pp 163-184.
- Park R "Structural Concrete Developments in Buildings", Proceedings of Silver Jubilee Conference, New Zealand Concrete Society, Wairakei, New Zealand, October 1989, pp 9-25.
- Hayashi T, Niwa H and Fukuhara M, "The Strengthening Methods of the Existing Reinforced Concrete Buildings", Proceedings of the Seventh World Conference on Earthquake Engineering, Vol 4, Istanbul, 1980, pp 89-97.
- Bett B J, Klingner R, and Jirsa J, "Lateral Load Response of Strengthened and Repaired Reinforced Concrete Columns", Structural Journal, American Concrete Institute, Vol 85, No 5, September-October 1988, pp.499-508.
- Rodriguez, M and Park, R, "Repair and Strengthening of Reinforced Concrete Buildings for Earthquake Resistance", Earthquake Spectra, Vol 7, No 3, California, August 1991, pp 439-459.
- "Code of Practice for the Design of Concrete Structures, NZS 3101:1982,"
 Standards Association of New Zealand, Wellington, New Zealand, 1982.
- "Building Code Requirements for Reinforced Concrete (ACI 318-89)", American Concrete Institute, Detroit, 1989.
- Park R, "Evaluation of Ductility of Structures and Structural Assemblages from Laboratory Testing", Bulletin of the New Zealand National Society for Earthquake Engineering, Vol 22, No 3, September 1989, pp.155-166.

- 14. Priestley M J N and Park R, "Strength and Ductility of Concrete Bridge Columns Under Seismic Loading", Structural Journal, American Concrete Institute, Vol 84, No 1, January-February 1987, pp 61-76.
- 15. Soesianawati M T, Park R, and Priestley M J N, "Flexural Ductility of Reinforced Concrete Columns with Low Axial Load and Limited Transverse Reinforcement", Proceedings of Pacific Conference on Earthquake Engineering, Vol 1, Wairakei, New Zealand, August 1987, pp 201-212.
- 16. Alcocer S and Jirsa J, "Assessment of the Response of Reinforced Concrete Frame Connections Redesigned by Jacketing" Proceedings of Fourth US National Conference on Earthquake Engineering, Vol 3, California, May 1990, pp 295-304.
- 17. Tanaka H and Park R, " Effectiveness of Transverse Reinforcement with Alternative Anchorage Details in Reinforced Concrete Columns", Proceedings of Pacific Conference on Earthquake Engineering, Vol 1, Wairakei, New Zealand, August 1987, pp 225-235.
- Mander J B, Park R and Priestley M J N, "Seismic Design of Bridges Piers" Research Report 84-2, Department of Civil Engineering, University of Canterbury, February 1984, p 483.
- Park, R, "Detailing of Transverse Reinforcement in Concrete Columns for Ductility", Proceedings of the 4th United States National Conference on Earthquake Engineering, Vol 2, Palm Springs, May 1990, pp 1037-1046.

NOTATION

 A_{sh} = total effective area of transverse confining reinforcement in direction of column under consideration

 A_{g} = gross area of column section

ď	Ξ	bar	diameter
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- f'_e = compressive cylinder strength of concrete
- f_v = yield strength of longitudinal reinforcing steel
- f_{vh} = yield strength of transverse reinforcing steel
- h = shear span of column
- $h_c = column depth$
- h" = width of core of column measured to outside of the peripheral hoop
- H = lateral load
- H_{ACI} = lateral load associated with the theoretical nominal flexural strength of column calculated using the ACI 318 method and assuming a strength reduction factor $\phi = 1$
- P = compressive load on column
- s_h = centre to centre spacing of hoop sets
- Δ = horizontal displacement
- Δ'_y = horizontal displacement measured at 0.75 H_{ACI} or first yielding of longitudinal reinforcement, whichever is less
- Δ_{v} = horizontal displacement at first yield
- ρ_t = ratio of area of longitudinal steel to gross area of column
- $\rho_s = A_{sh}/s_h h''$
- $\rho_{s,code}$ quantity of confining reinforcement in potential plastic hinge regions of columns recommended by ACI 318-89.
- θ = rotation of column stubs

 $\mu_a = \Sigma \mu/8$

 μ_{n} = nominal displacement ductility factor = Δ/Δ_{y} μ_{r} = real displacement ductility factor = $(\Delta + \theta h)\Delta_{y}$ $\Sigma \mu$ = cumulative nominal displacement ductility factor

 ${\cal F}_{i}$

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