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Studies on repair and strengthening methods of damaged reinforced concrete columns

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Abstract

Jacketing method with reinforced concrete (RC), steel plates or carbon fiber (CF) sheets has been widely used to repair or strengthen the RC columns damaged by the 1995 Hyogoken-Nanbu earthquake. The purpose of experimental studies reported in this paper is to confirm those repair or strengthening effects. To investigate the shear strength and ductility of RC columns repaired or strengthened by jacketing, eight column specimens were tested under constant axial compressive load and cyclic shear forces. The main conclusions obtained from the test were summarized as follows. (1) Shear strength and ductility of the repaired columns, in which the concrete remained crushed and the longitudinal bars remained buckled, can be restored over the pre-damaged level. (2) The maximum shear strength of the column repaired or strengthened by jacketing with RC, steel plates or CF sheets can be calculated by the formulas proposed in this paper.

Keywords: Evaluation of damage level; Repair work; Jacketing method; Carbon fiber sheets; Column test

1. Introduction

The Hyogoken-Nanbu earthquake which occurred on 17 January 1995 caused unprecedented damage to a large number of reinforced concrete (RC) buildings [1]. The damaged buildings had to be restored for reuse. Table 1 shows the criteria for restoration of damaged RC buildings. The terms "restoration", "repair" and "strengthening" in the table are defined as follows:

Restoration: to repair and strengthen damaged buildings so that they can be used again.

Repair: to improve the deteriorated structural performance of damaged buildings back to their original levels.

Strengthening: to improve the deteriorated structural performance of damaged buildings beyond their original levels.

Fig. 1 shows the evaluation of damage level of RC buildings.

The restoration design for damaged buildings had been performed according to many kinds of existing manuals and guidelines. Examples of such guidelines edited by Takenaka Corporation are:

1. *Table of techniques and procedures for repair*: a list for easy selection of repair techniques suited to the damage condition of each member in RC buildings (example; Figs. 2 and 3).
2. *Repair work sheets*: a sheet showing details of repair work, selection of standard materials and construction procedures.

The jacketing method of RC columns with RC and steel plates is adopted in the "Repair work sheets". The structural performance of RC columns repaired by jacketing has been verified by past researches. In this paper, however, an experimental study on repair and strengthening of RC columns is newly carried out due to the following reasons.

1. To restore the heavily damaged columns, in which the concrete remained crushed and the longitudinal bars buckled, by jacketing.
2. To adopt the restoration method with new materials, that is, carbon fiber (CF) sheets and shrinkage-compensating mortar.

Building damage by the 1995 Hyogoken-Nanbu earthquake required strengthening the existing buildings constructed before 1981 so that no damage or light damage by a large earthquake is expected. Strengthening

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Table 1
Criteria for restoration [2,3][a]

Damage level	Light	Minor	Medium	Major	Collapse
Seismic intensity in JMA scale					
Lower than V	○	△	×	×	×
V	○	○	△	×	×
Higher than V	○	○	○	△	×

[a] ○: restoration by repair; △: restoration by repair or strengthening; ×: restoration by strengthening or demolition.

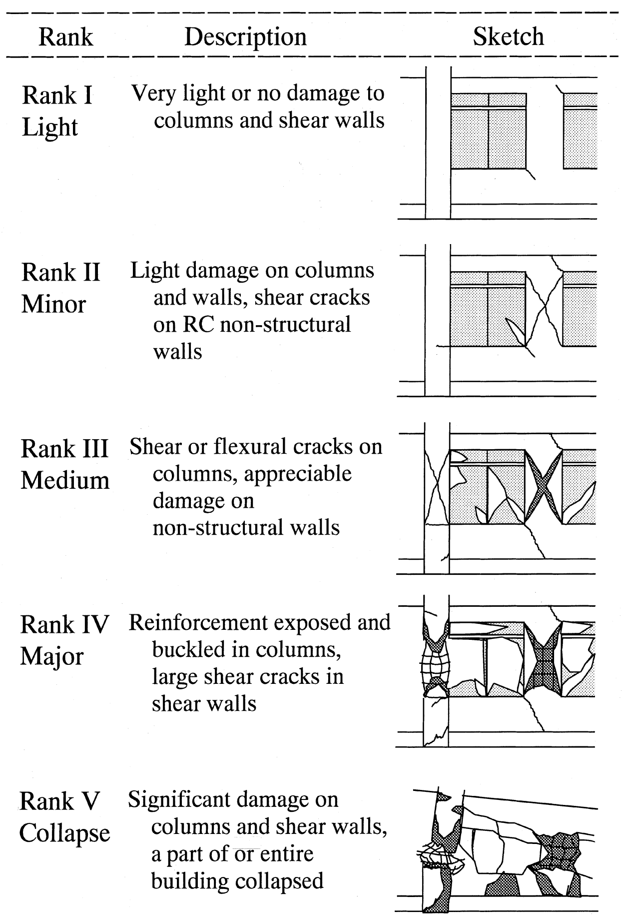


Fig. 1. Evaluation of damage level [4].

methods by jacketing with steel plates, CF sheets and preformed carbon fiber reinforced plastic (CFRP) plates are adopted. Test results on structural performance of RC columns strengthened by jacketing are also presented in this paper.

2. Column shear test

This experimental study was conducted to investigate the shear strength and behavior of repaired RC columns heavily damaged by an earthquake and strengthened RC columns. The methods of repair or strengthening in this test are RC jacketing, steel plates jacketing, CF sheets jacketing, preformed CFRP plates jacketing and shrinkage-compensating mortar grouting, which were adopted for actual repair or strengthening.

2.1. Specimens

Eight column specimens of an approximately 1/2 scale were tested. Table 2 summarizes the specimens. Fig. 4 shows the cross-sections of specimens and shapes. The column height of all specimens was 900 mm and the original cross-section was 350 mm in width $(B)\times 350$ mm in depth (D). The critical moment-to-shear span ratio (M/QD) was 1.29.

Specimen C1 is an original existing column before damage. The longitudinal reinforcing bars of specimen C1 were 4-D16 (reinforcement ratio: $Pt = Ar/(B \cdot D) =$ 0.65%, Ar: cross-sectional area of bars). Shear reinforcing bar (hoop) was 2-D6 @100 (shear reinforcement ratio: $Pw = Aw/(B \cdot s) = 0.18\%$, Aw: cross-sectional area of hoops, s: spacing of hoop).

Specimens C2 and C3 represent the restored columns which were assumed to be heavily damaged by the Hyogoken-Nanbu earthquake. Specimen C2 is a RC column jacketed with welded wire fabric, in which the crushed concrete remained within the reinforcing cage, and the cross-section was enlarged by the placement of high-fluidity concrete. For specimen C3, additional longitudinal reinforcing bars were arranged outside the buckled longitudinal reinforcing bars while the crushed concrete still remained in the column. The cross-section of specimen C3 was enlarged by placing steel plates along the perimeter of the column. High-fluidity concrete was grouted in the gap between steel plates and crushed concrete. The cross-section of specimens C2 and C3 was 450 mm in width×450 mm in depth $(M/QD = 1.0)$.

Cover concrete of specimens C4 and C5 was replaced with concrete and shrinkage-compensating mortar, respectively, leaving the crushed concrete inside the columns.

Specimen C6 was jacketed with steel plates along the perimeter of the original existing column.

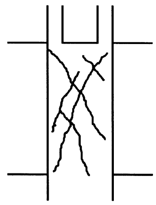
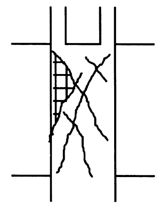
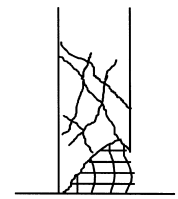
Damage rank	Rank III or less	Rank IV	Rank V
Sketch of damage			
Repair method	C10 Repair of cracks	C20 Grouting mortar with pressure	C30 Reinforcement of existing section
	C11 Repair of crack and partial loss of concrete	C21 Casting concrete, grouting mortar	C31 Jacketing with steel plate, grouting mortar
	C12 Repair of crack and partial loss of concrete	C22 Jacketing with welded wire fabrics and mortar	C32 Jacketing with steel plate, grouting mortar
	C13 Repair of partial loss of concrete C14 C15	C23 Jacketing with steel plate, grouting mortar	C33 Jacketing with steel plate, C34 Addition of vertical reinforcement, grouting mortar

Fig. 2. An example of guidelines for repair work [3].

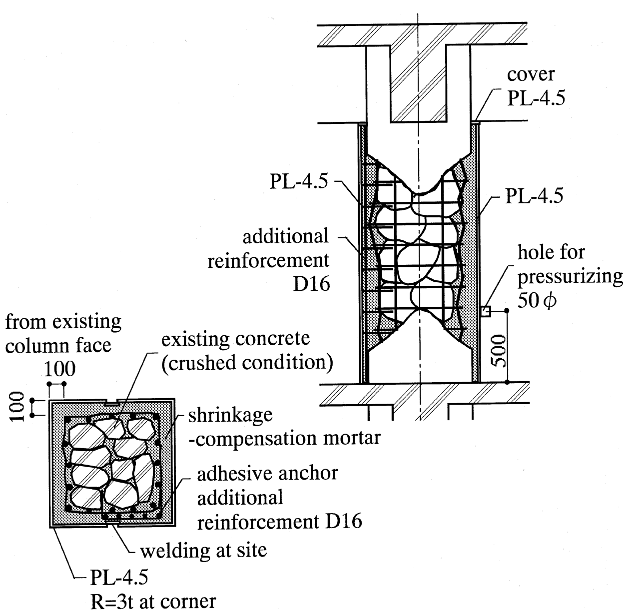


Fig. 3. An example of guidelines for repair work [3].

Specimen C7 was jacketed with two layers of CF sheets (fiber content in one layer: 300 g/m^2) to provide the strength equivalent to that of specimen C6 jacketed with steel plates.

Specimen C29 was jacketed with preformed CFRP plates. The gap between preformed CFRP plates and column concrete was filled with shrinkage-compensating mortar. The preformed CFRP plates, shown in Fig. 5, was equivalent to the CF sheets in terms of the strengthening effect.

For specimens C2–C5, in which the crushed concrete was left within the columns, the actual damage in the columns, where cracks reached the inside of the column core, was modeled by piling up pre-crushed concrete randomly in the hoop of the column after the installation of reinforcement (Fig. 6). In specimens C3 and C6, steel plates for jacketing were used as forms for concrete placement.

2.2. Materials

Mechanical properties of the concrete and reinforcing bars used for the specimens are listed in Tables 3 and 4, respectively.

For all of the specimens, D16 and D6 deformed bars (nominal diameters were 16 and 6 mm, respectively) were used as longitudinal reinforcing bars and shear reinforcements, respectively. In specimens C3 and C6, steel plates of 2.3 mm in thickness were used for jacketing.

2.3. Loading

Fig. 7 shows the test setup. Cyclic shear force was applied to the column while the axial compressive load was held constant. Cyclic shear force was controlled by deflection angle of the column, $R = \delta/h$; where δ is the measured horizontal displacement, and h is the column height ($h = 900$ mm). The axial compressive load applied to all the specimens was 865 kN. This axial load corresponded to the axial stress of 30% of the concrete compressive strength of specimen C1.

Table 2
Test program

Specimen no.	Cross-section	Damage degree	Repair/strengthening method
C1	350 × 350	Not damaged	Not repaired (standard specimen)
C2	450 × 450	Crush of concrete	Crushed concrete remaining
			Shear reinforcement by welded wire fabric
			Placement of high-fluidity concrete
C3	450 × 450	Crush of concrete	Crushed concrete remaining
		Buckling of longitudinal bars	Buckled longitudinal bars remaining
			Addition of longitudinal bars
			Jacketing with steel plate (thickness = 2.3 mm)
			Placement of high-fluidity concrete
C4	350 × 350	Crush of concrete	Placement of concrete
C5		Crush of concrete	Placement of shrinkage-compensating mortar
C6		Not damaged	Jacketing with steel plates (thickness = 2.3 mm)
C7		Not damaged	Jacketing with carbon fiber sheets
C29	410 × 410	Not damaged	Jacketing with performed CFRP plates

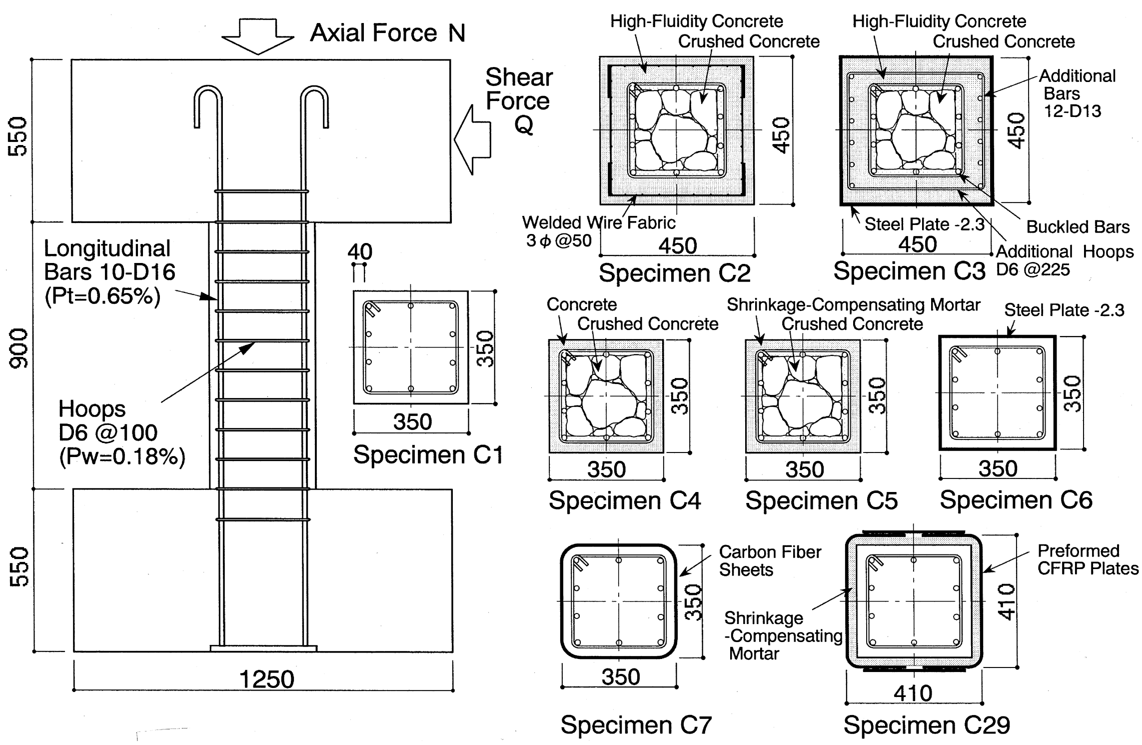


Fig. 4. The cross-sections of specimens and shapes (units; mm).

3. Results of experiments

3.1. Comparison among specimens C1, C2 and C3

Fig. 8 shows the crack patterns of specimens C1 and C2 at the deflection angle $R = 10/1000$ rad. Fig. 9 shows the shear force–deflection hysteresis loops. The vertical axis represents the measured shear force, Q, and the horizontal axis represents the deflection angle, R.

In specimen C1, an original existing column specimen, flexural–shear cracks occurred at $R = 2.5/1000$ rad. Subsequently, diagonal shear cracks extended all over the column face with increase of deflection. The maximum strength was reached at $R = 4.1/1000$ rad.

A similar crack process was also observed for specimen C2. Control of crack width by welded wire fabric caused shear force to increase until the deflection angle, R, reached 9/1000 rad. Then the welded wire fabric

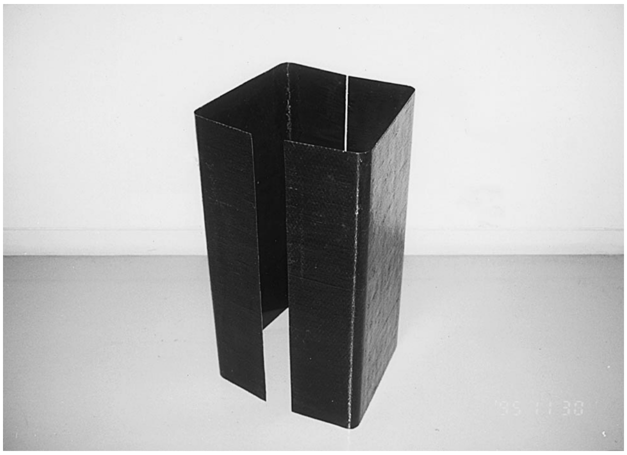


Fig. 5. Preformed CFRP plates.

Table 3
Mechanical properties of concrete[a]

Classification	σ_B (N/mm^2)	Ec (N/mm^2)	σ_t (N/mm^2)	Specimen
Concrete	25.1	23 100	2.28	C1, C4, C6, C7
Concrete	24.1	21 500	2.47	C29
Concrete[b]	28.8	24 600	–	C2, C3, C4, C5
Concrete[c]	40.2	27 900	2.40	C2, C3
Mortar	57.0	24 900	–	C5
Mortar	55.1	21 900	4.05	C29

[a] σ_B: Measured compressive strength; Ec: secant modulus at $\sigma_B/3$; σ_t: Measured tensile strength.
[b] Concrete for crushed concrete.
[c] High-fluidity concrete.

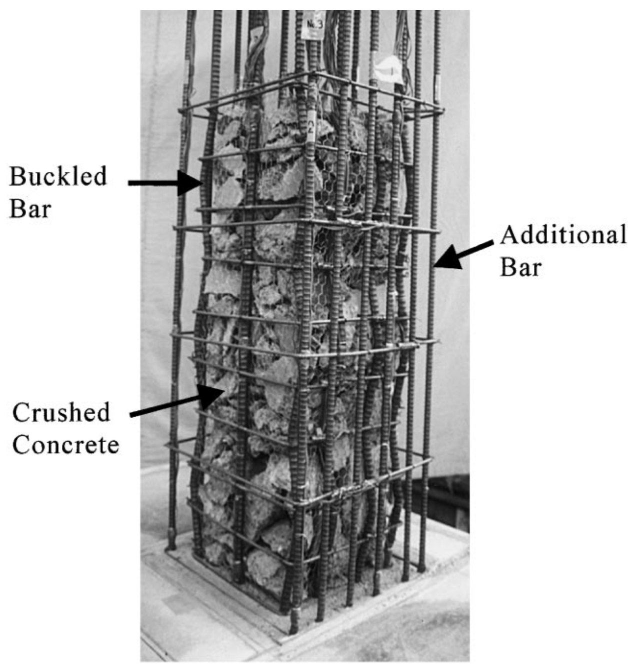


Fig. 6. Arrangement of specimen C3.

Table 4
Mechanical properties of bars and Plates[a]

Type of bars		σ_y (N/mm^2)	Er (N/mm^2)	Specimen
Longitudinal reinforcing bar	D16	373	190 000	C1–C7
	D16	453	188 000	C29
Transverse reinforcing bar	D6	302	194 000	C1–C7
	D6	329	191 000	C29
Welded wire fabric	3ϕ	614	196 000	C2
Steel plate	$t = 2.3$ mm	392	206 000	C3, C6

[a] σ_y: tensile yield strength; Er: modulus of elasticity.

fractured along shear cracks, and the shear force suddenly decreased.

In specimen C3, the column concrete was confined by steel plates jacketing, resulted in the increase of the maximum shear strength and ductility. The maximum shear strength was reached at $R = 15/1000$ radian. The steel plates then buckled outward, and finally cracks occurred in the steel plates in the corners of the column.

A comparison of hysteresis loops for these specimens shows that specimens C2 and C3 have larger shear strength and ductility than specimen C1. It was, therefore, confirmed that the shear strength and the ductility of the damaged column can be enhanced remarkably by jacketing with RC or steel plates.

3.2. Comparison among specimens C1, C4 and C5

Fig. 10 shows the crack patterns of specimens C4 and C5. In specimen C4, many shear cracks were observed. The shear cracking area of specimen C5 was not as large as C4. Fig. 11 shows the comparison of the envelopes of shear force–deflection hysteresis loops. It was found that C1 and C4 presented almost the same load-deflection relations. This may be because the post-cast concrete used in specimen C4 filled the gaps of the crushed concrete. This specimen C4, therefore, has performance equivalent to or better than that of the original column specimen C1 even without an increase of the cross-section.

In specimen C5, the maximum shear strength was large because compressive strength of shrinkage-compensating mortar was large. However, diagonal shear crack was accompanied by quick reduction in

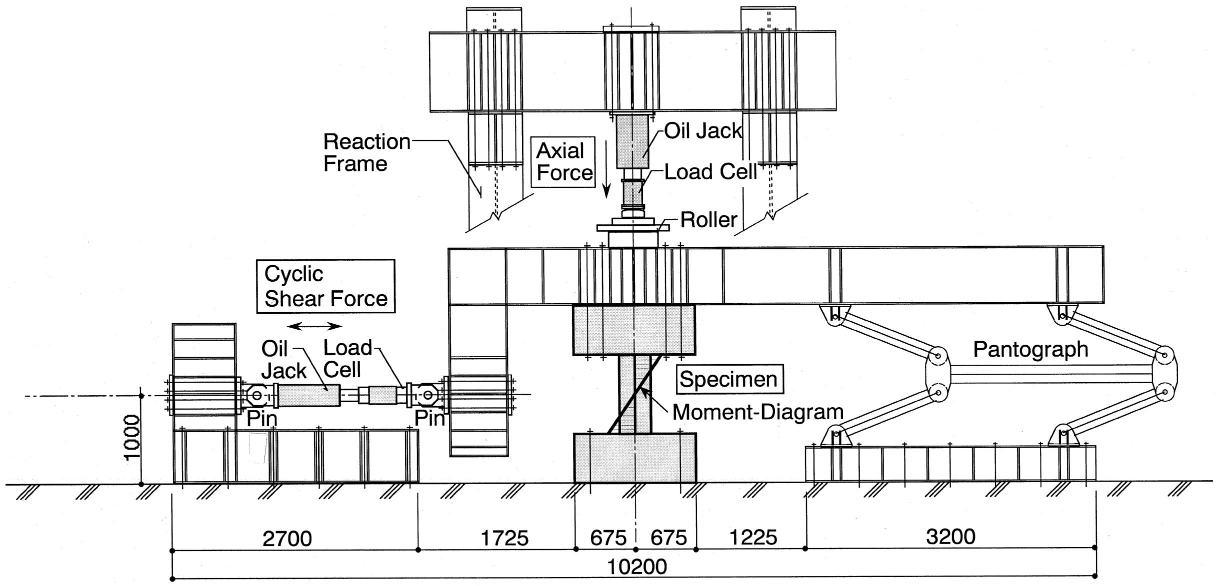


Fig. 7. Details of test setup (units; mm).

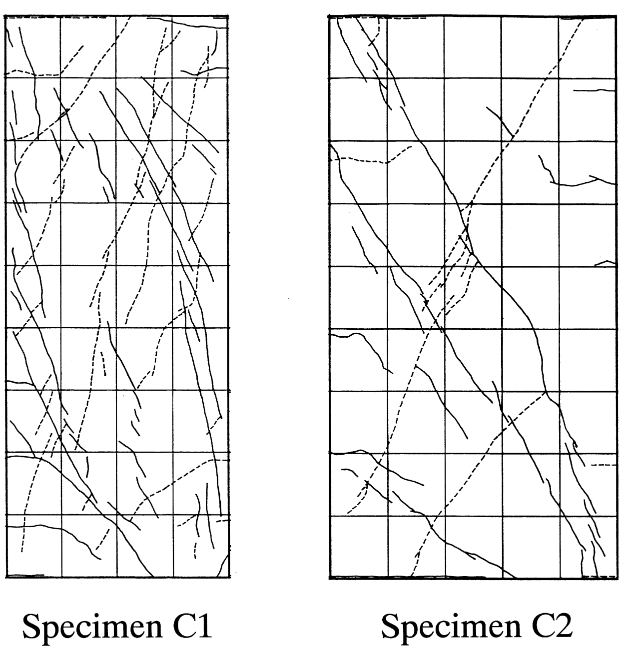


Fig. 8. Crack patterns at $R = 10/1000$ rad.

shear force. Such a rapid deterioration of shear strength may have occurred because shrinkage-compensating mortar could not transmit the shear force through the aggregate interlocking.

3.3. Comparison among specimens C6, C7 and C29

A comparison of envelope curves of specimens C6, C7 and C29 is shown in Fig. 12. The ductility of specimens C6, C7 and C29, jacketed with steel plates, CF sheets and preformed CFRP plates, respectively, had been enhanced substantially owing to the confining effect of concrete. In specimen C6, out-plane buckling of steel plates at the top and bottom ends of the column became significant, and shear force gradually decreased. The specimen, however, maintained its axial force until the deflection angle, R, of 50/1000 radian. In both specimens C7 and C29, CF sheets or preformed CFRP plates broke in the corners of the column, and shear force suddenly decreased because of the loss of confining effect on concrete.

Envelopes for specimens C6 and C7 are almost the same. Steel plates and CF sheets have similar strengthening effects. Specimen C29 shows larger shear strength than specimens C6 and C7, because of larger cross-section.

4. Review of maximum shear strength

Table 5 shows the comparison between measured and calculated maximum shear strength of specimens. Shear force at the maximum flexural strength, *Qmu*, and ultimate shear strength, *Qsu* were calculated by the formula shown in Table 5.

Equivalent compressive strength of concrete, σ_B, for the specimens containing crushed core concrete was calculated by the following equation.

$$\sigma_B = (\sigma_{B_1} \cdot A_1 + \sigma_{B_2} \cdot A_2)/(A_1 + A_2),$$

A_1 is the area of core concrete (250×250 mm^2), A_2 the area of post-cast concrete or post-cast mortar, B the column width, D the column depth

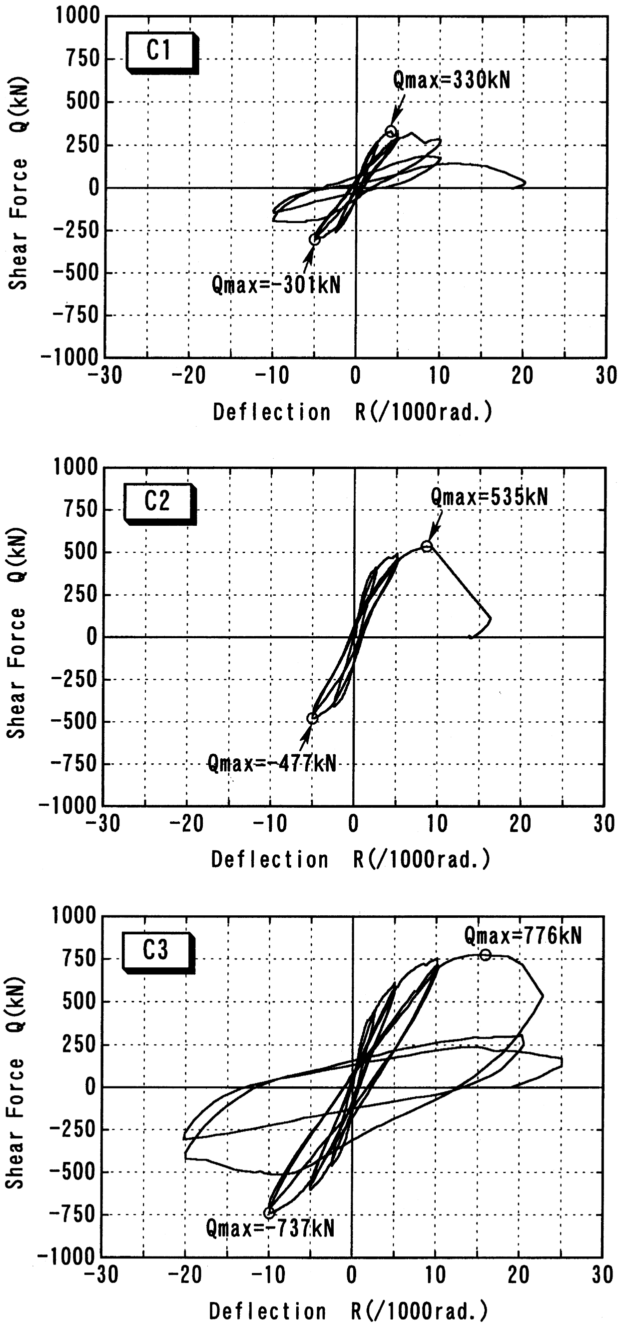


Fig. 9. Shear force–deflection hysteresis loops.

$$A_1 + A_2 = B \cdot D,$$

σ_{B_1} the compressive strength of concrete for crushed concrete shown in Table 3, σ_{B_2} is the compressive strength of post-cast concrete or mortar.

Tensile strength of CF sheet for shear strength calculation, σ_{CF}, was set at 2/3 of standard maximum tensile strength ($\sigma_{CF} = 1618$ N/mm^2).

The measured maximum shear strength of specimen C1, an original column specimen, fairly agreed with the calculated shear strength.

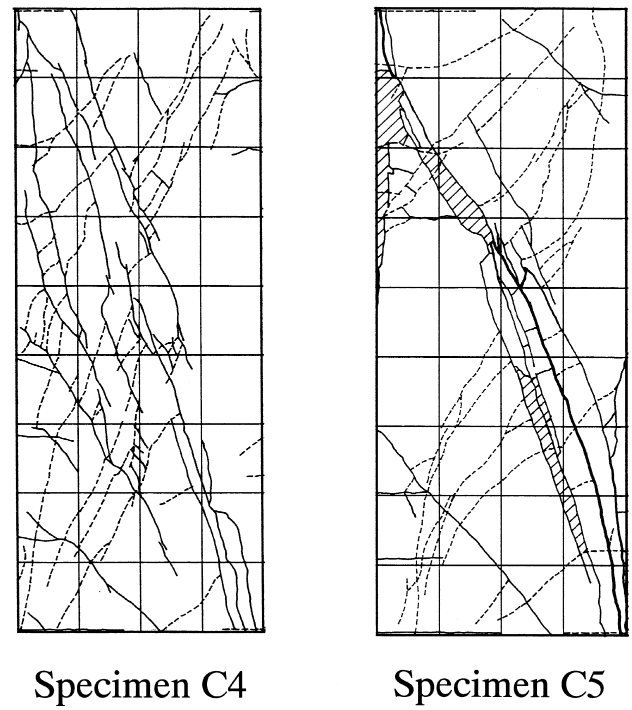


Fig. 10. Crack patterns at $R = 10/1000$ radian.

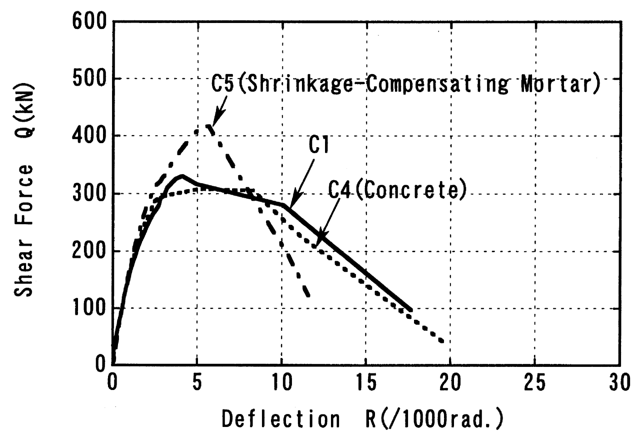


Fig. 11. Envelopes of shear force–deflection hysteresis loops.

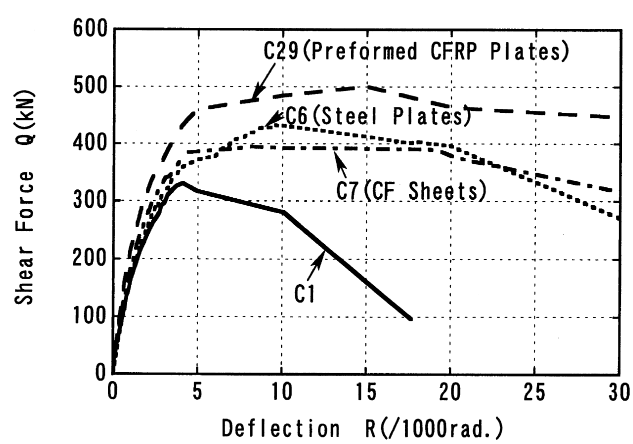


Fig. 12. Envelopes of shear force–deflection hysteresis loops.

Table 5
Comparison of maximum shear strength

Specimen no.	σ_B (N/mm^2)	$P_{we} \cdot \sigma_{ye}$ (N/mm^2)	Axial load, N (kN)	Maximum shear strength		
				Measured, eQu (kN)[a]	Calculated, cQu (kN)[b]	Measured/ calculated
C1	25.1	0.551	865	330	286	1.16
C2	34.7	0.811	865	535	466	1.15
C3	34.7	4.610	865	776	665	1.17
C4	27.0	0.551	865	308	293	1.05
C5	41.7	0.551	865	417	347	1.20
C6	25.1	5.706	865	431	418	1.03
C7	25.1	3.639	865	394	379	1.04
C29	32.6	3.149	887	512	490	1.04

[a] $eQu = \max(pQu, nQu); pQu, nQu$: Measured maxium shear strength in positive loading and negative loading, respectively.
[b] $cQu = \min(Qmu, Qsu)$; $Qmu = 2 \cdot Mu/h$ (kgf); $Mu = 0.8at \cdot \sigma_y \cdot D + 0.5N \cdot D(1 - N/(B \cdot D \cdot \sigma_B))$ (kgf · cm); $Qsu = \{0.068Pt^{0.23}(\sigma_B + 180)/(M/Qd + 0.12) + 2.7\sqrt{P_{we} \cdot \sigma_{wye}} + 0.1N/B \cdot D\}B \cdot j$ (kgf); $P_{we} \cdot \sigma_{wye} = P_{w_1} \cdot \sigma_{wy_1} + \sum P_{w_2} \cdot \sigma_{wy_2}$ (kgf); $Pt = at/(B \cdot D)$; B, D, d, h are the section width, depth, effective depth and height of column, respectively (cm); at area of tensile longitudinal bars (cm^2); M/Q the moment–shear span (cm); P_{w_1} the original shear reinforcement ratio; P_{w_2} the additional shear reinforcement ratio (welded wire fabric, steel plate, and CF sheets); σ_B is the compressive strength of concrete (kgf/cm^2); σ_y the yield strength of longitudinal bars (kgf/cm^2); σ_{wy_1} the tensile strength of original shear reinforcement (kgf/cm^2); σ_{wy_2} is the tensile strength of additional shear reinforcement (kgf/cm^2); 1 kgf = 9.80665 N, 1 kgf/cm^2 = 0.0980665 N/mm^2.

The measured maximum shear strength of specimens C2 and C3, was 1.15 and 1.17 times larger than the calculated shear strength, respectively. From these facts, the calculation estimated well the maximum shear strength of the repaired columns by jacketing. The comparison between measured and calculated shear strength of specimen C3 shows that the buckled longitudinal reinforcing bars restrained by steel plates jacketing may have contributed to the resistance to flexure and shear.

In actual placement, shrinkage-compensating mortar had a greater efficiency in filling the gaps of crushed concrete than ordinary concrete. For both specimens C4 and C5, although repaired without enlarging the cross-section, the measured maximum shear strength was larger than the calculated shear strength.

The measured maximum shear strength of specimens C6, C7 and C29 agreed well with the calculated shear strength. It was found that the amount of steel plates and CF sheets to be applied for reinforcement could be calculated in almost the same manner as ordinary shear reinforcements.

5. Conclusions

As a result of experiments on structural performance of repaired RC columns with damage and strengthened RC columns, the following conclusions were obtained.

1. Shear strength and ductility of the repaired columns, in which the concrete remained crushed and the longitudinal bars remained buckled, can be restored over the level of the pre-damaged columns.
2. Shear strength of column having rehabilitated cover concrete with shrinkage-compensating mortar was restored back to the original level. Shear strength, however, suddenly decreased after the maximum strength was reached. This may be because shrinkage-compensating mortar can hardly transmit the shear force by aggregate interlocking.
3. The ductility of jacketed columns with steel plates, CF sheets or preformed CFRP plates sufficiently increased.
4. The maximum shear strength of the column repaired or strengthened by jacketing with RC, steel plates, CF sheets or preformed CFRP plates can be calculated by the proposed formulas. Even when the crushed concrete remained in the column, the maximum shear strength, calculated by substituting an equivalent concrete strength obtained according to the area ratio between crushed and post-cast concrete, agreed well with experimental results.

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