-Technical paper-

EXPERIMENTAL AND ANALYTICAL INVESTIGATION OF SEISMIC RETROFIT FOR ONE-SIDED WING-WALL RC COLUMNS

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ABSTRACT: Three seismic retrofit techniques for shear critical one-sided wing-wall RC columns are proposed in this paper. Three different retrofitted test specimens in addition to a standard one with shear span to depth ratio of 2.0 for column only were tested under the reversed cyclic lateral forces and a constant axial load (axial force ratio=0.2 for column only) simultaneously. Two simplified methods have been attempted to calculate the flexural strength. The effectiveness of the proposed retrofit techniques is evaluated through both experimentally and analytically.
KEYWORDS: Wing-wall RC column, seismic retrofit, high strength steel bar, prestress, angle, steel plate.

1. INTRODUCTION

In Japan, the wing-wall section is popular for low-rise to medium height RC buildings, such as, school building, police station, hospital etc. This wing-wall section is widely used particularly for external column besides the window. But, with the addition of wing-walls to original column section, the input of shear force is considerably increased. As a result, if the transverse reinforcement is insufficient, the undesirable brittle shear failure is likely to happen. Therefore, it is necessary to asses the seismic performance of RC wing-wall column as well as the requirement of retrofit for the existing RC buildings.

It is well known fact that the strength and ductility of RC columns, which are vulnerable to seismic excitation can be extremely enhanced by transverse confinement that also acts as shear reinforcement. Considering this fact and based upon the investigation of previously proposed retrofit method for RC column utilizing high strength steel bar prestressing by Yamakawa & Kurashige et al [1], three seismic retrofit techniques for shear critical one-sided wing-wall RC columns are proposed in this paper. The objective of this paper is to verify the seismic performance of the proposed retrofit techniques against brittle shear failure with ensuring ductile flexural response. Finally, two simplified methods have been attempted to calculate the flexural strength for the one-sided wing-wall RC columns.

2. TEST PLAN

In order to ascertain the effectiveness of the proposed retrofit techniques, four one-sided wing-wall RC column specimens were tested under the combination of cyclic lateral forces and a constant axial load simultaneously. Each specimen consisting of a square column (depth=250mm, height=1,000mm) and a concentric wing-wall (thickness = 50 mm) only at one side of column was cast monolithically with two stabs. The specimen R03WO-P0 was the non-retrofitted standard test specimen. In case of test specimen R03WO-P65A, the main square column was retrofitted by pre-tensioned high strength steel bars (diameter=5.4mm) placed on the width side and steel angles placed on the depth side. Here steel angles are used as an alternative of corner blocks. Because, steel angles are economic, convenient and easily available. Corner blocks are such kind of devices (L-shaped with equal leg having length of 75mm, width of 37mm and thickness of 32mm) which are placed at four corners of column to hold the high strength steel bars that employed like a circumferential tie-hoop around the column during retrofitting. The level of prestressing strain of steel bar was about 2450μ (at a stress of 490 MPa) of approximately 1/3 of yield strain (6100μ). In this case, steel bars were inserted into wing-wall by making holes (12.5mm) near the junction of

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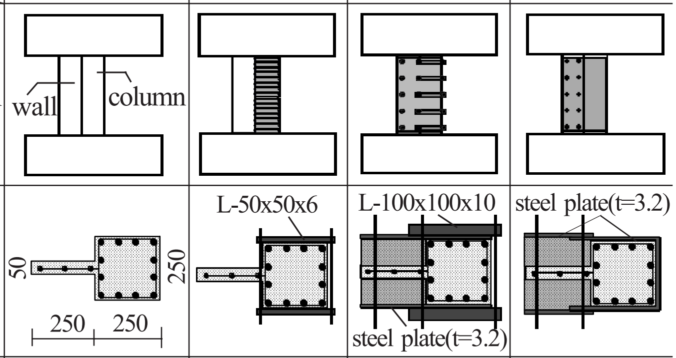
Table 1 Properties of materials

		a (mm^2)	f_y (MPa)	ε_y (%)	E_s (GPa)
Rebar	D10	71	365	0.20	183
Hoop	3.7ϕ	11	391	0.19	205
steel bar	5.4ϕ	23	1220	0.61	200
	13ϕ	133			
Steel plate	t=3.2 mm	1600	279	0.14	200
Steel angle	L-50x50x6 L-100x100x10		250*	-	200*

Notes : * = assumed values,
a = cross sectional area,
f_y= yield strength of steel,
ε_y= yield strain of steel,
E_s=Young's modulus of elasticity.

Table 2 Retrofit details of test specimens (unit : mm)



	R03WO-P0	R03WO-P65A	R03WO-SA	R03WO-S
Elevation				
Cross section (mm)				
Steel bar	-	5.4ϕ-@65	13ϕ-@200	13ϕ-@200
Pretension strain level	-	2450μ	1250μ	1250μ
Common details	$N/(bD\sigma_B)$=0.2, M/(VD)=2.0, σ_B=25.9 MPa, Add. concrete=68.5 MPa, Rebar:12-D10 (p_g=1.36%), Hoop:3.7ϕ-@105 (p_w=0.08%), Wall reinforcement:3.7ϕ-@105 single, Wall thickness=50.			

main column and wing-wall. Another specimen R03WO-SA was retrofitted like a sandwich and converted to a rectangular shape. During retrofitting, thin steel plates were embedded on both sides (depth side) of column up to wing-wall and additional non-shrinkage concrete was cast within the embedded steel plates (500 x 980 x 3.2mm). After hardening of post-cast concrete, prestress was applied in the steel bars (diameter=13mm) that were penetrated across the wing-wall beforehand as well as in the other steel bars placed on the width side of column opposite to the wing-wall. During prestressing in the steel bars placed on the both sides (width sides) of column, steel angles were also attached on the both sides of the steel plates in the column region. The last test specimen R03WO-S was retrofitted in similar way to the test specimen R03WO-SA. But the main differences were that no angles were used and the steel plate in the form of a one end opened tube was embedded on the three sides of column except the side attached to wing-wall. Another difference was that no steel bars were placed on the column side opposite to the wing-wall side. Epoxy resin was also grouted into the gap between the column surface and the steel plate to eliminate the gap between them. In last two cases, the level of prestressing strain of steel bar was about 1250μ (at a stress of 250 MPa). The mechanical properties of the materials employed in the test specimens are listed in Table 1. Schematic figures of retrofit techniques are presented in Table 2. The scale factor in this experiment program was about 1/2.4 to model a low-rise school building designed according to pre-1971 design code. The axial force ratio ($N/(\sigma_B Db)$) was 0.2 when only column section was considered but in case of column including original wing-wall, that ratio was 0.17 and including wing-wall with additional concrete that was 0.1. The shear span to depth ratio ($M/(VD)$) for column only was 2.0 and for column with wing-wall that was 1.0.

3. EXPERIMENTAL RESULTS AND DISCUSSIONS

The observed cracking patterns of wing-wall columns after the end of test are presented in Fig. 1. Since the specimens R03WO-SA and R03WO-S were covered by thin steel plates on the both sides, the cracking patterns of these test specimens were detected by simply detaching the steel plates and not taking off the additional post-cast concrete after the end of test. The experimental results on the relationship between the shear force V and the story drift angle R, and the variation of average longitudinal strain ε_v along the column axis with the story drift angle are illustrated in Fig. 2. The dotted lines drawn in the V-R curves are the calculated flexural strength of main square column without considering the lateral confinement effect of retrofit, and taking into account the P-Δ effect by AIJ simplified equation. The variation of accumulated absorbed energy (W) with drift angles are presented in Fig. 3.

In non-retrofitted specimen R03WO-P0, shear crack was generated first at R=0.4% and with the increase of drift angle, cracks spread rapidly on both sides of column and wing-wall. Due to poor amount of shear reinforcement, the experimental lateral capacity of this specimen dropped suddenly and at a drift angle right before -1.0%, this wing-wall column failed in a brittle shear manner with the formation of large diagonal crack (crack width=5mm).

The experimental lateral capacity of retrofitted specimen R03WO-P65A reached the calculated flexural strength and at R=0.5%, the longitudinal reinforcement of main column yielded. At R=1.5%, vertical slit appeared along the junction line between the main square column and wing-wall. With the increase of drift angle, this slit widened gradually by the formation of large cracks. Moreover, holes made near the junction of main column and wing-wall to penetrate the high strength steel bars during retrofitting also accelerated this slit formation. Finally, the unified wing-wall column was transformed into two individual columns with sustaining the total axial load in proportion to section area. Again, the shear and flexural strengths of wing-wall were very small to maintain high ductility due to poor amount of longitudinal and transverse reinforcement. So, at large drift angles, the experimental lateral capacity was mainly governed by the square column only. Experimentally, it is observed that pre-tensioned high strength steel bar strains decreased with the increase of drift angle. Since the prestress was applied only in one direction and a small gap (about 5mm) existed between high strength steel bar and column surface, therefore, with the formation of more cracks, confinement effect was reduced and the lateral capacity decreased gradually. From ε_v-R curve in Fig. 2, it is also observed that the average longitudinal strain is compressive due to the spalling of cover concrete and the buckling of rebars at the top and bottom end regions of column. Moreover, after R=3%, lateral capacity lowered below the flexural strength of column only. In the past investigation [2] for the case of column with wing-walls on the both sides either concentrically or eccentrically in which the column only was retrofitted by pre-tensioned high strength steel bars placed on four sides of column utilizing corner blocks (in case of concentric wing-walls) or on two sides of column utilizing corner blocks and steel plate bands (in case of eccentric wing-walls), the lateral capacity maintained over the flexural strength of column only even at large drift angles and also the ε_v-R curves were tensile and sharp. Therefore, it is understood that the utilization of angle as an alternative of corner block is not suitable in respect of maintaining higher ductility.

In case of specimen R03WO-SA, at R=1.0%, flexural crack happened and longitudinal reinforcement yielded with the formation of plastic hinges at top and bottom end regions of column. At R=1%, lateral capacity also reached the maximum value. But, after this drift angle, lateral capacity decreased gradually. This may be due to disintegration between original wing-wall column and post-cast concrete. Moreover, high strength steel bars placed on column side opposite to wing-wall were not attached directly to column surface and hence, cover concrete spalled. Experimentally, it is also observed that pre-tensioned steel bar strains decreased with the increase of drift angle. However, this retrofitted specimen exhibited a relatively good response than the previous one. The ε_v-R curve in Fig. 2 shows that at large drift angles, the average longitudinal strains are tensile, which indicates the rigid rotation of whole specimen within the formed plastic hinges.

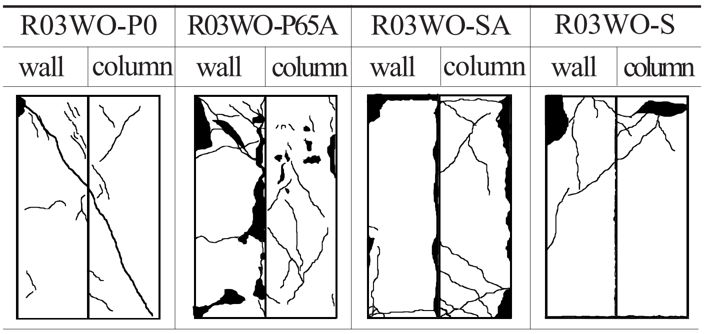
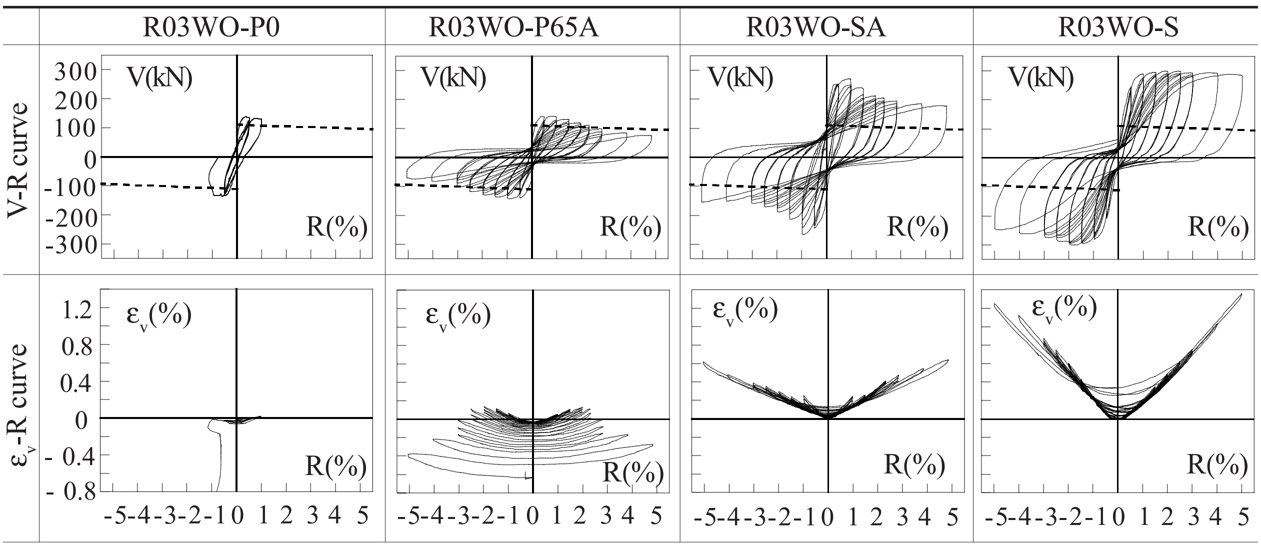


Fig. 1 Observed cracking patterns after the end of test



Note: Dotted lines indicate the calculated flexural strength of column only (M/(VD)=2) by AIJ simplified equation.

Fig. 2 Experimental V-R and ε_V-R relationships

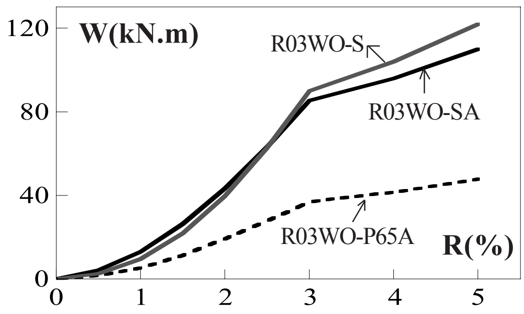


Fig. 3 Accumulated absorbed energy (W)

In other retrofitted specimen R03WO-S, the experimental lateral capacity increased significantly and also maintained even at large drift angles. In this case, the longitudinal reinforcement also yielded at R=1.0% and plastic hinges formed at the top and bottom end regions of column. The ε_v-R curve of this specimen also shows the rigid rotation of whole specimen within the formed plastic hinges. This retrofitted specimen exhibited the best response with respect to sustain high lateral capacity at large drift angles and this kind of retrofit can be used as a reliable retrofit technique for enormous seismic excitation.

In Fig. 3, after R=3%, there are noticeable changes in slopes. Because after this drift angle, the cyclic loading test was carried out for one cycle each instead of three cycles. In Fig. 2, although the lateral capacity of R03WO-S is maintained and the lateral capacity of R03WO-SA is decreased as the drift angle increases, but there is a little difference in accumulated absorbed energy of two specimens as it can be seen from Fig. 3. This is because at drift angles until about 2.5 %, although the lateral capacities of R03WO-SA are lower but the hysteresis loops are wider than that of R03WO-S. Therefore, in the viewpoint of accumulated absorbed energy, it may be concluded that the both retrofit techniques endorse the effective improvement of seismic performance.

4. ANALYTICAL INVESTIGATION

In order to predict the seismic failure behavior analytically, it is necessary to calculate the shear and flexural strength accurately. The shear strength is calculated by AIJ simplified equations [3] based on the truss and arch mechanism. The flexural strength is calculated more precisely by fiber model. As an alternative to calculate the flexural strength, two simple addition methods and ACI method are also employed. The calculation procedures of shear strength by AIJ formula and flexural strength by simple addition methods are illustrated in Fig. 4 and Fig. 5 respectively. In the calculation of flexural strength by ACI method, the actual shape of concrete compressive stress block is replaced by an equivalent rectangle. The rectangle has a mean stress of 0.85 times the crushing strength of concrete and a depth a, where $a/c=\beta_1$=0.85 (c is neutral axis depth) for concrete strength up to 27.6 MPa and β_1 is reduced continuously by 0.05 for each 6.9 MPa of strength in excess of 27.6 MPa. Again, at flexural strength , the unconfined concrete strain of extreme compression fiber is taken to 0.003. Since in all the retrofitted specimens, pre-tensioned high strength steel bars were employed only in one direction and in transformed sandwiched section, the strength of additional concrete (68.5 MPa) is higher than that of original wing-wall column, therefore, it is difficult to predict the effectiveness level of enhancement of concrete strength. The test results show that by retrofitting, the maximum lateral capacity of specimen R03WO-P65A is increased a little than non-retrofitted specimen. So, in such case, it is reasonable to adopt unconfined concrete strength for analysis. Again, the retrofittings of transformed sandwiched specimens are such that the column only of these specimens can be assumed as confined by steel tube and high strength steel bars like a hoop and confined concrete strength (38.6

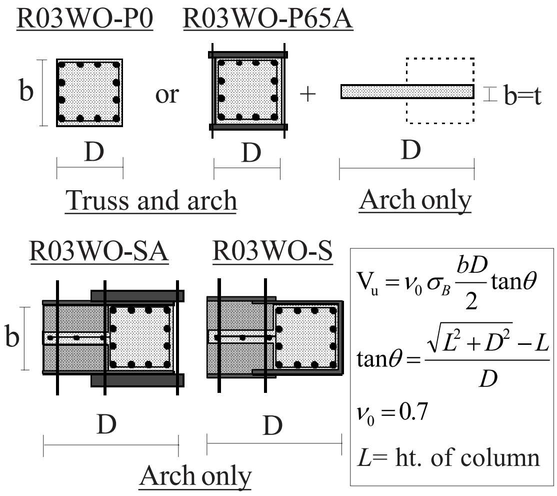


Fig. 4 Calculation methods of shear strength by AIJ formula

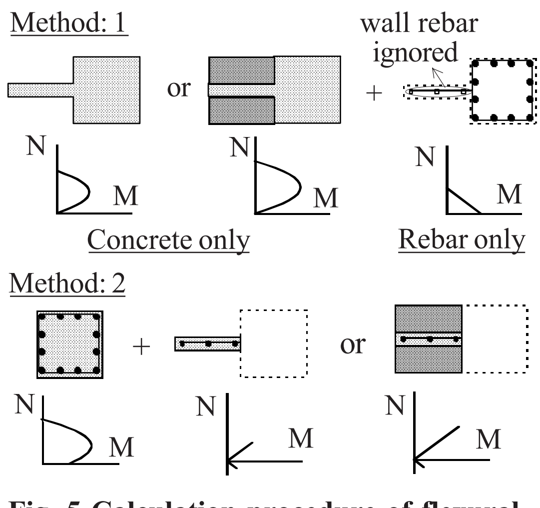


Fig. 5 Calculation procedure of flexural strength by simple addition methods

MPa) thus obtained can be considered for strength of entire converted rectangular section. The stress-strain relationship of concrete is considered according to Mander's model [4]. In Fig. 6, axial force-shear force interaction diagram together with calculated shear strength are presented and also test and analytical results are compared.

For specimens R03WO-P0 and R03WO-P65A, the calculations of shear strengths are proposed in two parts. One is for square column part by truss and arch mechanism, and the other is for wing-wall part by arch mechanism only as shown in Fig. 4. Again, in arch mechanism, the common part of square column and wing-wall is counted twice. Since, the concrete of wing-wall is unconfined, the result is not affected considerably. In case of specimen R03WO-P65A, the exact determination of effective shear strength enhancement due to retrofit by steel angles and pre-tensioned high strength steel bars is not possible. Since high strength steel bars are placed only in one direction and a small gap is existed between column surface and high strength steel bars, there is a great possibility of slipping angles and reduction of pretension forces in steel bars due to spalling of cover concrete. However, since the test result of this specimen shows the ductile flexural response, shear strength improvement can be assured due to this retrofitting. For R03WO-SA and R03WO-S, the shear strength is calculated based on the arch mechanism of concrete and the value of ν_0 (coefficient of effective compressive strength of concrete in shear) is considered 0.7 only due to effect of confinement by steel plates and high strength steel bar prestressing. Here, it may be assumed that steel plates and pre-tensioned high strength steel bars can maintain the rigidity and provide protection against spalling of concrete. Moreover, the retrofit of these specimens may also contribute some shear resistances.

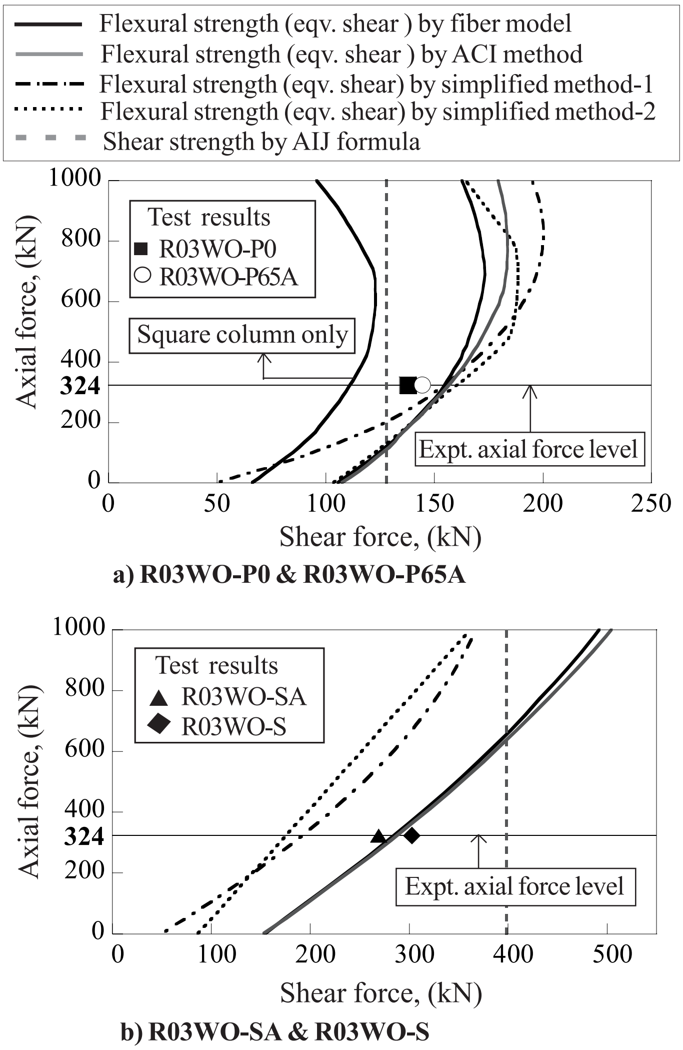


Fig. 6 Calculated flexural strength (eqv. shear force) and shear strength

Since the unified one-sided wing-wall RC column section is asymmetric about the center line of square column section, the contraflexure point of column member is not located at the center of the member. Therefore, the section has two different moment capacity depending on the situation of compression or tension either in column side or in wall side during reversed cyclic loading. So, in this case, the flexural strength of unified section calculated by fiber model, ACI method and simple addition methods for two different situations are converted to equivalent unique shear force. Moreover, due to unsymmetry, the plastic centroid (the centroid of resistance of the section if all the concrete is compressed to the maximum stress and all the steel is compressed to the yield stress with uniform strain over the section) of the unified section is not located at the center of the square column section only. For analysis by fiber model and ACI method, the location of neutral axis at ultimate stage is considered at plastic centroid. According to simplified method-1, the flexural strength of unified section is calculated as the summation of flexural strength of unified concrete section only (without reinforcement) and the flexural strength of rebars in column only (without concrete). In this calculation, for simplicity, the rebars in wing-wall are neglected and in these two separate calculations, the location of neutral axis at ultimate stage is considered at the center of square column. Again, by simplified method-2, the flexural strength of unified section is calculated as the summation of flexural strength of square column only (with reinforcement) and the flexural strength component of wing-wall including or excluding additional concrete. In this calculation, the flexural component of wing-wall is considered as a vector and the location of neutral axis at ultimate stage is also considered at the center of square column [5].

The calculated results by fiber model for specimen R03WO-P0 show that flexural strength exceeds the shear strength, which indicates shear failure behavior. From Fig. 6, it is observed that in case of R03WO-P0 and R03WO-P65A, the test results agree well with the calculated results by fiber model, ACI method and also by both simple addition methods. Moreover, in case of R03WO-SA and R03WO-S, the test results also agree well with the calculated results by fiber model and ACI method, but does not match with the calculated results by both simple addition methods. According to simple addition method-1, the moment capacity of section at zero axial force is assumed as the moment capacity of rebars only and concrete does not contribute any moment. Moreover, in this method, at different axial forces, all rebars are assumed yielded. But practically, moment capacity is influenced by the combined interaction between concrete and steel as well as by section geometry and reinforcement arrangement. Again, according to simple addition method-2, the largeness of wing-wall component vector also influences the axial force-moment interaction diagram. Since the areas of wing-wall part of specimens R03WO-SA and R03WO-S are large compared to that of specimens R03WO-P0 and R03WO-P65A and as explained before, the wing-wall section is asymmetric and also the location of neutral axis at ultimate stage is considered at the center of square column. Therefore, in case of R03WO-SA and R03WO-S, the both simplified methods are very much conservative. However, as an alternative, ACI method can be well applied in the aspects of design and assessment.

The final goal in the seismic design is to prevent the brittle shear failure with ensuring the ductile flexural response. The design controls of strength and ductility for one-sided wing-wall RC columns are made possible by selecting the retrofit techniques proposed herein. However, it should be noted that the assessment of the proposed retrofit techniques was performed through a limited number of test specimens. Moreover, there is some possibility which disregards the effective utilization of retrofit when the attention is not paid to either plastic hinge formation of beams which is attached to the column with wing-wall or the rotation of base of column. For this reason, the cyclic loading tests of the frame built with these seismic elements will also be required. Therefore, to suggest detailed design recommendation, more experimental investigations are needed to be carried out.

5. CONCLUSIONS

1. Although the angles are economic, convenient, easily available and also serve the purpose of corner blocks, the retrofit method of R03WO-P65A utilizing angles is not suitable in respect of maintaining higher ductility, but the utilization of pre-tensioned high strength steel bars on four sides of column is better.
2. In the viewpoint of accumulated absorbed energy and practical design drift angle (1-1.5%), the retrofit technique of R03WO-SA can be recommended. On the other hand, in the context of strength, ductility and energy absorption, the retrofit technique of R03WO-S can be selected as a reliable retrofit method for enormous seismic excitation.
3. In the viewpoint of design and assessment, the simple addition methods can be applied as an alternative to calculate the flexural strength for original wing-wall column sections. In case of wing-wall section transformed into rectangular sandwich, ACI method can be well applied.

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