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STRENGTH AND DEFORMABILITY OF REINFORCED CONCRETE COLUMNS WITH WING WALLS

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ABSTRACT

A series of tests on reinforced columns with wing walls have been conducted from 2007 to 2009 to investigate seismic performance of the members with an irregular section. The test results on four specimens were reported with method of evaluation for shear and flexural strengths in Japanese design practice. The specimens were one-half scale reinforced concrete columns combined with a wing wall only on one side in the loading direction. The effects of asymmetric horizontal section, reinforcement details and varying axial force on seismic performance were investigated. The ultimate strengths of the members were identified with the observed failure modes. The measured flexural and shear strengths were compared with the calculated strengths based on the conventional design formula as well as new formula in practice proposed by the authors with the results of other test series.

Introduction

Use of columns with wing walls is a simple and cost-effective design option to provide a reinforced concrete building with relatively higher lateral load carrying capacity. The field investigations on the structural damages induced from past earthquakes and laboratory tests indicated that reinforced concrete columns with wing-walls had relatively good seismic performance as earthquake-resistant members increasing column stiffness and strength in reinforced concrete buildings. In Japan, however, wing-walls have not been designed and used very much as structural members in recent reinforced concrete buildings. Instead, the walls have been often separated from the main frames by installing seismic slits between the column and the wall. Since the columns with wing-walls show different seismic behavior from shear walls or independent columns, the evaluation method of its strength and ductility has not been clearly defined in the guidelines for design practice, which makes it difficult to design columns with wing-walls. However, the wing-wall attached to column undoubtedly increases the lateral

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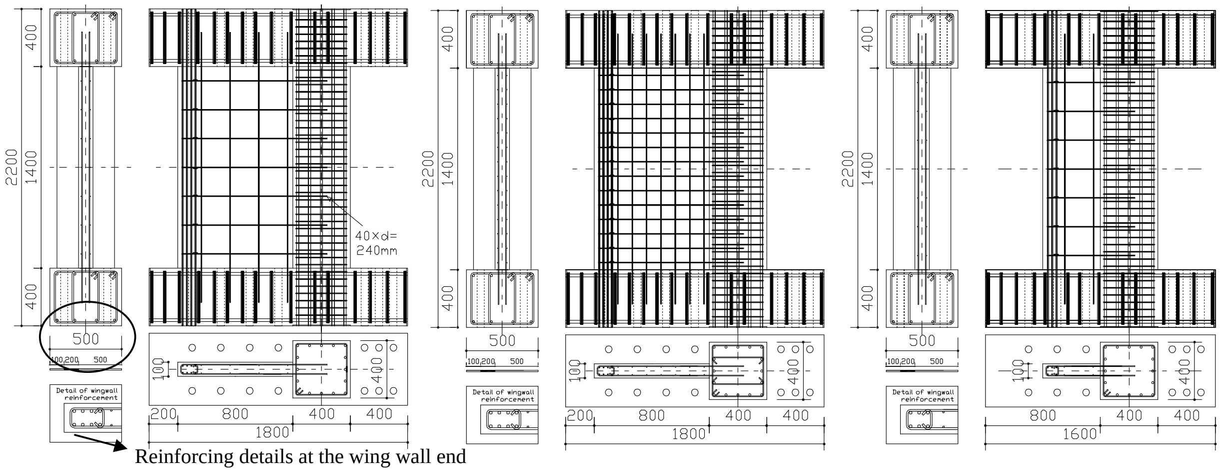
strength of column, therefore, installing seismic slit between column and wing-wall might be inefficient in most cases of low-rise buildings, which could have been designed as strength-based-design. Conventional evaluation methods for columns with wing-walls may be found in Seismic Evaluation Standard for RC buildings by The Japan Building Disaster Prevention Association [JBDPA, 2001] and also in Guidelines for Standard Requirements on Building Structures by The Building Center of Japan [BCJ, 2007]. However, these conventional evaluation methods for ultimate strength and ductility are not rational in mechanical point of view. In addition, these methods are formulated by assuming columns with both-side wing-walls, so the design equations are not basically applicable to columns with one-sided wing-wall.

On the other hand, a simple fail-safe system consisting of relatively strong superstructure and sway-slip foundation could be feasible against extreme motions exceeding the design level. It has been proved from full-scale shake table tests on reinforced concrete school buildings conducted at E-Defense, the world largest three-dimensional shake table, in 2006[Kabeyasawa 2007a,b] that the slip behavior on the flat base slab would occur under very high ground acceleration so that the response of the superstructure could be controlled as minor as insensitive to the level and characteristics of possible extreme motion. Based on the test result, wing-walls will also be useful to give the required capacity to the columns in the fail-safe system so as to remain in minor damages even under the extreme motions.

In this study, four specimens representing reinforced concrete column with a wing wall were tested to investigate seismic performance of the member with irregular section. The specimens were one-half scale columns combined with a wing wall only on one side in the loading direction. The ultimate strengths of the members were identified with the observed failure modes. The measured flexural and shear strengths were compared with the calculated strengths based on the conventional design formula as well as new formula, which had been proposed by the authors [Kabeyasawa 2007c] with the results of the first test series.

Test Methods on Columns with One-sided Wing Wall

The characteristics of specimens are listed in Table 1. The scale of specimens is half or two-thirds to typical sections of full-scale medium-rise buildings in Japan. The section and reinforcement details of each specimen described in Table 1 were planned following the previous tests [Kabeyasawa 2008 and Tojo 2008] on column with wing walls on its both sides. The length of wing-wall in SWT-L and SWT-LW is equal to the sum of lengths of both-side wing-wall tested in 2007 [Kabeyasawa 2008 and Tojo 2008], and the length of wing-wall in SWT-SC and SWT-SV is half to that of SWT-L and SWT-LW. The section of SWT-LW is identical to SWT-L, but the reinforcement ratio of SWT-LW is twice to that of SWT-L except longitudinal reinforcement at the end of wing wall (Fig.1) that is identical between two specimens. On the other hand, SWT-SC and SWT-SV have the same reinforcement ratio to that of SWT-L, but its longitudinal reinforcement at the end of wing-wall is 6-D10 which is smaller than that of SWT-L (8-D10) and same to that of both-sided wing-wall specimens tested in 2007. SWT-SV and SWT-SC have identical section details, but SWT-SV was subjected to varying axial load while SWT-SC was tested under constant axial load. Material properties of reinforcement are summarized in Table 2. The nominal strength of concrete was 24MPa, while 31.6 to 33.3MPa from the material test as shown in Table 1.



(a) SWT-L (b) SWT-LW (c) SWT-SC, SV

Figure 1. Specimen details

Table 1. List of specimens

<table>
<tr><th rowspan="2">Specimen</th><th rowspan="2">Concrete Strength (N/mm^2)</th><th colspan="3">Column (mm)</th><th colspan="5">Wing wall (mm)</th><th colspan="2">Axial Force (kN)</th><th colspan="2">M/Q (mm)</th></tr>
<tr><th>Section</th><th>Main bars</th><th>Hoop</th><th>Length</th><th>Width</th><th>Vertical reinforcement</th><th>Main bars at section end</th><th>Horizontal reinforcement</th><th>+</th><th>-</th><th>+</th><th>-</th></tr>
<tr><td>SWT-L</td><td>33.3</td><td rowspan="4">400 x400</td><td rowspan="4">16-D16 (2.0%)</td><td>2-D6@50 (0.32%)</td><td rowspan="2">800</td><td rowspan="4">100</td><td>2-D6@200 (0.32%)</td><td rowspan="2">8-D10</td><td>2-D6@200 (0.32%)</td><td colspan="2" rowspan="3">800</td><td colspan="2" rowspan="3">1000</td></tr>
<tr><td>SWT-LW</td><td>33.3</td><td>4-D6@50 (0.64%)</td><td>2-D6@100 (0.64%)</td><td>2-D6@100 (0.64%)</td></tr>
<tr><td>SWT-SC</td><td>31.6</td><td rowspan="2">2-D6@50 (0.32%)</td><td rowspan="2">400</td><td rowspan="2">2-D6@200 (0.32%)</td><td rowspan="2">6-D10</td><td rowspan="2">2-D6@200 (0.32%)</td></tr>
<tr><td>SWT-SV</td><td>31.6</td><td colspan="2">400+2Q</td><td>900</td><td>1100</td></tr>
</table>

Table 2. Material properties of steel

Mark	D6	D10	D16
Yield Strength (N/mm^2)	340	363	372
Elastic Young's Modulus ($10^4 N/mm^2$)	17.31	18.58	18.86
Tensile Strength (N/mm^2)	453	512	558

For SWT-L, SWT-LW and SWT-SC, the total constant axial load N applied at the column center is 800kN corresponding to the axial load ratio of 0.2 that is N / (2Ac*Fc) = 0.2, where, Ac is column section area and Fc is compressive concrete strength (24Mpa). And SWT-SV was subjected to varying axial load calculated from N=400+2Q (Q: lateral shear in kN), which simulated the actions in a prototype six-story building. Lateral loading is controlled based on displacement and reversed at the peak drift ratios of ±1/400, ±1/300, ±1/200, ±1/150, ±1/100, ±1/75, ±1/50, ±1/37.5 and ±1/25 with a single cycle of loading per each drift increment. For the specimens subjected to constant axial load (SWT-L, SWT-LW and SWT-SC), shear span (moment-to-shear ratio, M/Q) is kept to be 1000mm (clear height of column is 1400mm).

Therefore, the shear span to depth ratio (M/Qd, d is effective depth of column with wing-wall) of SWT-L and SWT-LW is 0.83 and 1.25 in case of SWT-SC. On the other hand, shear span of SWT-SV subjected to varying axial load was kept to be 900mm (M/Qd =1.13) in positive direction (column compression) and 1100mm (M/Qd =1.38) in negative direction (column tension) as shown in Table 1. The loading method is described in detail elsewhere(Kabeyasawa, 2009).

Conventional Design Equations for Columns with Wing-Walls

The following Equations (1) and (2) for flexure and shear in the guidelines [JBDPA.2001 and BCJ.2007] are based on the idea of the equivalent wall thickness method, where the column with wing-wall section are replaced with the equivalent uniform thickness wall with the same sectional area (Table 4). The problems of these conventional methods may be summarized as: At the end of the wing wall section on which excessive stress is concentrated, the thickness of equivalent section is always overestimated compared to that of actual thickness of wing wall. Since the ends of the wing walls are not confined generally by the transverse reinforcement, it becomes overestimation to substitute into the equivalent wall from e.g. the wall with boundary columns. It is also apparently overestimation to calculate an equivalent transverse reinforcement ratio as the sum of the ratios of column hoops and wall web reinforcement from Equation (3).

$$M_u = (0.9+\beta)a_t\sigma_y D + 0.5ND\left\{1+2\beta-\frac{N}{b_e DF_c}\left(1+\frac{a_t\sigma_y}{N}\right)^2\right\} \quad (1)$$

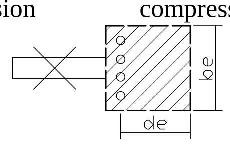
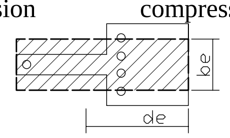
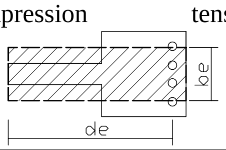
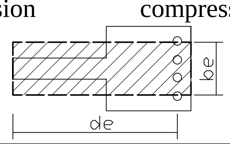
[Nm]

$$Q_{su} = \left\{\frac{0.053p_t^{0.23}(F_c+18)}{M/Qd_e+0.12}+0.85\sqrt{p_{we}\sigma_{wy}}\right\}b_e j_e + 0.1N \quad (2)$$

[N]

$$p_{we} = p_w\left(\frac{b}{b_e}\right)+p_s\left(\frac{t}{b_e}\right) \quad (3)$$

Table 4. Equivalent sections for strength evaluation of columns with wing-walls

Guidelines	(a) [JBDPA, 2001]	(b) [BCJ, 2007]
Positive loading (→)	tension compression 	tension compression 
Negative loading (←)	compression tension 	tension compression 
de	Distance from the center of column tensile bar to edge of compressive zone	Distance from the center of tensile bar group to edge of compressive zone
be	(column section + compressive wing wall section) /(column depth + compressive wing wall length)	Whole section area / whole depth

The wing wall length on the tensile side is disregarded expediently in the equation by JBDPA as shown in Table 4 so that the overestimation due to the equivalent area replacement could be canceled out. However, if the wing wall lengths between left and right side are different or if the wing wall is attached on only one side, the method is not applicable and undefined. The calculation methods for the wing wall length (de) and the wall thickness (be) are different between JBDPA.2001 and BCJ 2007 also shown in Table 4. In addition, simple accumulation of the ratios of the hoops and the web reinforcement by Eq.(3) is apparently invalid and irrational.

Proposed Design Equation in Practice – Cumulative Method with Divided Sections

Considering those problems in the conventional evaluation methods described as above, a modified design formula for columns with wing walls has been proposed(Kabeyasawa, 2007), referred as "*Cumulative Method with Divided Sections.*" This proposed method evaluates the shear strength of column with wing wall by accumulating individual shear strength of wall and column based on conventional practical design equations for shear(Eqs. (7), (8), (9)), where the sections are divided into the direction of the wall length as shown in Fig. 3.

$$Q_{su} = Q_{suw} + Q_{suc} + 0.1N \qquad [\mathrm{N}] \qquad (7)$$

$$Q_{suw} = \left\{ \frac{0.053 p_{twe}^{\ 0.23}(F_c + 18)}{\dfrac{M}{Qd_w} + 0.12} + 0.85\sqrt{p_{wh}\sigma_{why}} \right\} t_w j_w \qquad [\mathrm{N}] \qquad (8)$$

where $p_{twe} = a_{tw} / t_w d_w$, a_{tw}: area of column tensile bars and wing wall vertical reinforcements within 2 layers, M/Q: shear span ($0.5 \le M/Qd_w \le 2$), $d_w = 0.95(D + l_1 + l_2)$, $p_{wh} = a_{wh} / t_w s_w$, $p_{wh} = a_{wh} / t_w s_w$, σ_{why} : yield strength of wing wall reinforcement.

$$Q_{suc} = \left\{ \frac{0.053 p_{tce}^{\ 0.23}(F_c + 18)}{\dfrac{M}{Qd_{ce}} + 0.12} + 0.85\sqrt{p_{cwe}\sigma_{cwy}} \right\} b_{ce} j_{ce} \qquad [\mathrm{N}] \qquad (9)$$

where $p_{tce} = a_{tc} / \{(B - t_w) d_{ce}\}$, a_{tc} : area of column tensile reinforcement, M/Q : shear span ($1 \le M/Qd_{ce} \le 3$), $d_{ce} = 0.95D$, $p_{cwe} = (a_w - p_{wh} t_w s)/(b_{ce} s)$, $p_{cwe} = a_w /(b_{ce} s)$, $b_{ce} = B - t_w$, a_w : hoop area of column, s : hoop spacing, p_{wh}: horizontal reinforcement ratio of wing wall, σ_{cwy}: yield strength of hoop, B: column width, t_w: wing wall width, $j_c = 7/8 \cdot d_{ce}$

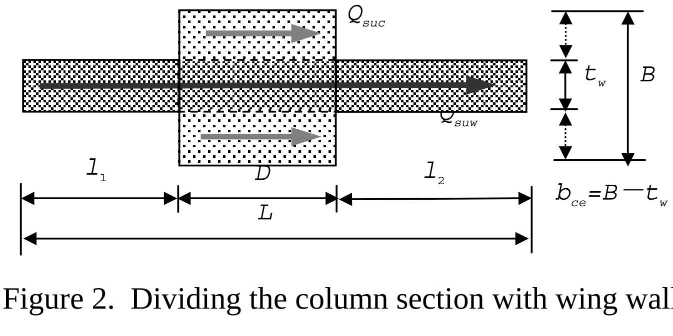


Figure 2. Dividing the column section with wing wall

Test Results on Columns With One-Sided Wing-Wall

Flexural bending and shear cracks of all specimens occurred and progressed from the wing-wall at the loading cycle of +1/400rad. In the negative direction when the wing wall is in compression, the shear cracks were initially developed in the wing-wall and top of column at the loading cycle of -1/400rad. In case of SWT-SV subjected to varying axial load, flexural bending cracks were much wider compared to the other specimens, which might be due to low axial load level. Figure 3 shows the final failure states of each specimen. The failure modes of the specimens are described separately on column and wing-wall in each direction because they are different. The failure mode is determined from yielding sequence of reinforcing bars based on measured strains, such as longitudinal or transverse re-bars of column and horizontal or longitudinal re-bars of the wing wall. The failure mode of the specimen SWT-L may be defined as flexural bending failure in the positive direction (column in compression) with yielding of the longitudinal reinforcement at the end. In the negative direction (wing wall in compression), the wing wall failed in shear. Crushing occurred in the loading cycle of -1/200rad at the bottom of the wing wall. The failure mode of SWT-LW is identical to that of SWT-L, though the specimen did not collapse until the loading cycle of +1/25rad. The failure mode of SWT-SC was similar to that of SWT-LW, which was flexural failure in the positive direction and shear failure in the negative direction. The failure mode of SWT-SV was flexural failure mode in the positive direction and shear failure in the negative direction, which was the same as those observed in other specimens. Although compressive failure of wing wall (crushing) occurred at relatively large drift ratio (1/75rad.) compared to the other specimens, the ultimate failure mode was changed from shear to flexural failure characterized by yielding of the longitudinal re-bars of the column. The failure modes, lateral resistance and axial capacity of each specimen are described with deformation levels in detail elsewhere(Kabeyasawa, 2009).

Hysteretic Relations

The hysteretic relations of each specimen are shown in Fig. 3, where calculated flexural and shear strength are also shown with horizontal lines, where three different equations for the shear strength described as above are used including JBDPA(2001), BCJ(2007), and *Cumulative Method with Divided Sections* (Kabeyasawa, 2007). Higher calculated strengths being out of scale were not illustrated. The strengths of SWT-SV subjected to varying axial load were calculated using the experimental axial load at the peaks. In the positive direction (column compression), all the specimens failed in flexure due to the yield of longitudinal reinforcements at the end of the wing wall section. Ductile behavior with slight strength deterioration was observed after the post-peak region. On the other hand, different failure modes were observed in the negative direction where the column was in tension and the wing wall in compression. Drastic deterioration of the lateral resistance and the loss of the axial load carrying capacity were observed in the specimen SWT-L showing typical shear failure mode. Onset of strength deterioration in the specimen SWT-LW and SWT-SC was initiated soon after attaining the maximum strength followed by crushing of concrete at the edge of the wing wall. The specimen SWT-SV subjected to varying axial load showed higher deformability and lower strength and stiffness than those of SWT-SC subjected to the constant axial load.

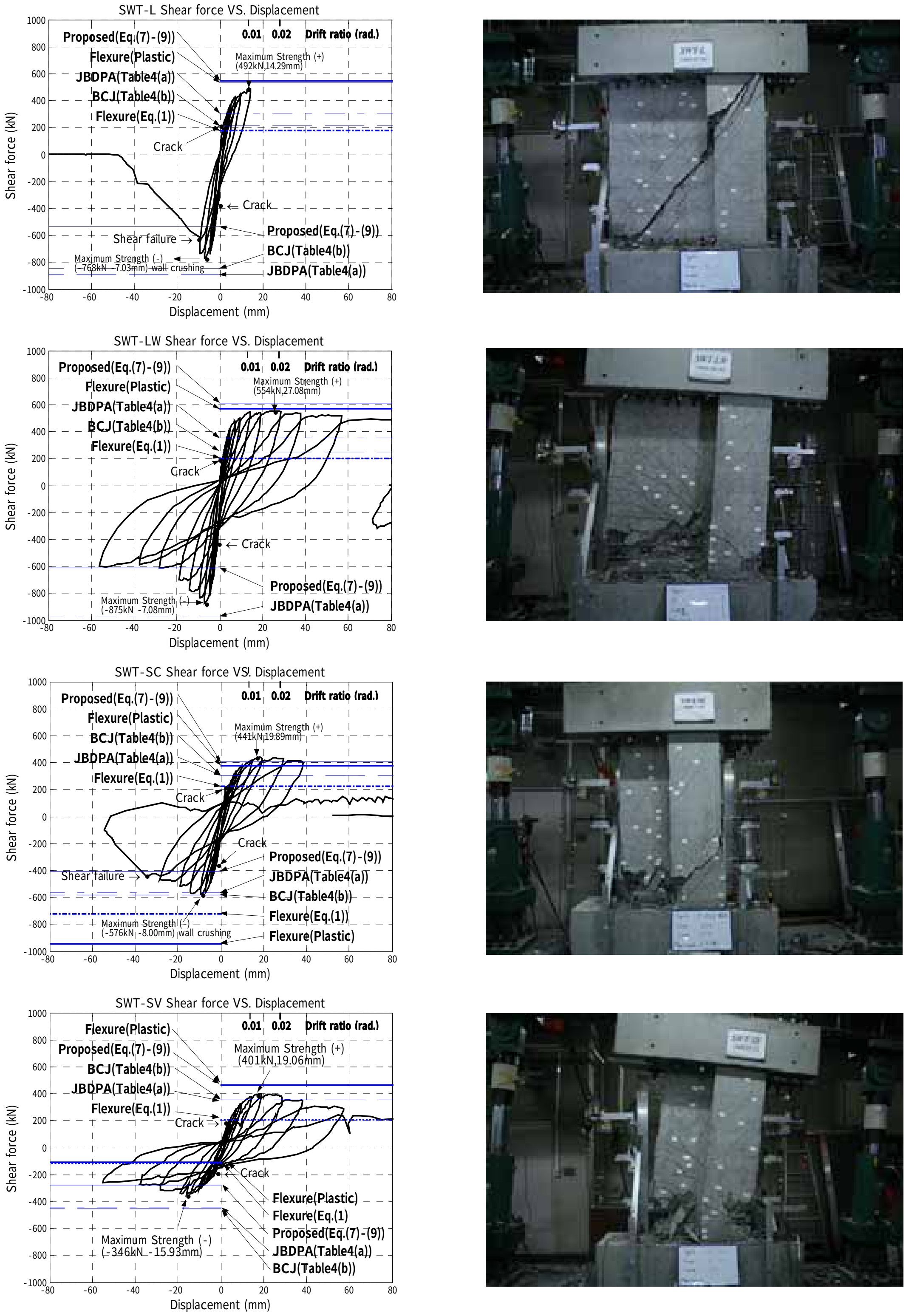


Figure 3. Hysteretic relations and collapsed specimens

Observed and Calculated Ultimate Strengths

Figure 4 compares the observed ultimate strengths of the specimens with the calculated strengths based on both conventional and proposed formula, where the strength of SWT-SV subjected to varying axial load was calculated using the observed axial load at peak. As is described in the figure, in the positive direction where the column is in compression, all the specimens showed flexural failure mode characterized by yielding of longitudinal reinforcement at the end of the wing wall section. On the other hand, both shear and flexural failure indicating concrete crushing or yielding of the column main bars were observed in the negative direction. When the ultimate strength was attained, the failure mode of all the specimens could be categorized into shear failure. From these results, observed maximum strengths in positive direction may be compared with the calculated flexural strengths, and those in negative direction may be compared with the calculated shear strengths.

The white and black markers shown in Fig. 4 indicate shear and flexural failure mode of specimens, respectively, from which the calculated shear strengths of the specimens categorized into the shear failure mode (white markers) are compared with the observed lateral strengths. While the shear strengths calculated from conventional evaluation methods by JBDPA Seismic Evaluation Standard [JBDPA, 2001] and BCJ Guidelines [BCJ, 2007] are almost the same and overestimated the observed shear strength by 1.15~1.3 times, while the shear strength calculated from the proposed formula underestimated the observed ones (i.e., 20% below that of SWT-SV and 30% below those of the other specimens). These results show that the conventional equations, where the wing wall in tension is disregarded so that the overestimation could be canceled out in the estimation, is not conservative in case of the columns with one-sided wing wall. On the other hand, the proposed formula provides conservative estimates by approximately 20~30% of safety margin, which is a little smaller to that of Arakawa's equation obtained for the past tests on independent columns and shear walls.

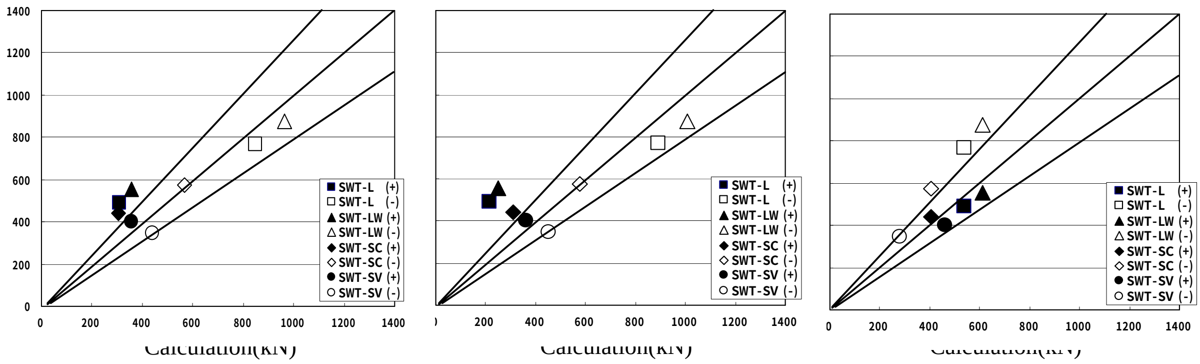


Figure 4. Comparison of observed and calculated shear strengths

In cases of SWT-L and SWT-SC between which the wing wall lengths were different, the shear strength of SWT-L calculated from the conventional equations is much higher than that of SWT-SC compared to the test results, which means that if the wing wall is longer, the higher shear strength is calculated from the conventional equations. On the other hand, the proposed formula could estimate the shear strength with constant safety margin regardless of the wing wall length. The different reinforcement steel ratio between SWT-L and SWT-LW did not affect the ratio of observed shear strengths to calculated ones from both conventional and proposed equations, which means that all the formulas could evaluate the effect of shear reinforcement.

The lateral strengths of the specimens that exhibited flexural failure mode (black marks) are compared with the calculated flexural strengths in Fig. 5. The flexural strengths of all the specimens calculated from Eq. (4) underestimated the experimental results because the longitudinal reinforcements at the end of wing-wall section are not being considered. On the other hand, the flexural strengths calculated by the equation of BCJ 2007 based on the plastic theory considering the whole cross section and reinforcements showed good agreement with the test results.

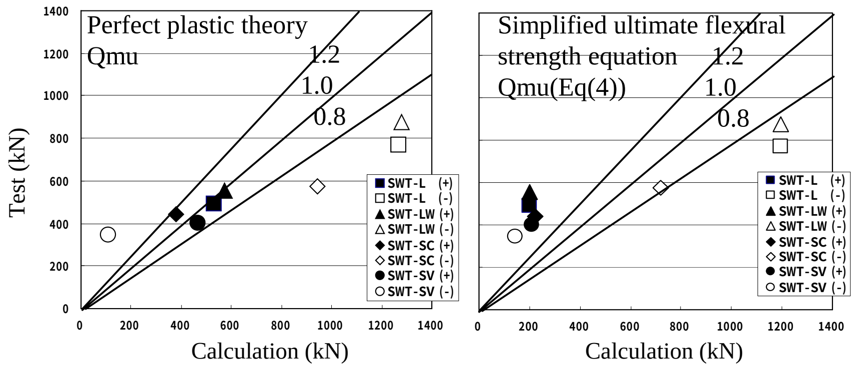


Figure 5. Comparison of calculated and observed flexural strengths

Conclusions

Four columns with one-sided wing wall with different section details and axial loading conditions were tested. The effects of these parameters on the shear strengths and deformation capacities were investigated. The observed strengths are compared with the calculated from the conventional and the proposed design formula for practical evaluation. The following conclusions may be drawn from the tests.

Regarding the effects of the loading directions, different failure modes such as shear and flexural failure were observed commonly among the specimens. In the positive loading direction, when the column was in compression, the longitudinal reinforcement at the end of wing-wall section yielded and the specimen was in flexural failure mode with slight strength deterioration and good deformability. On the other hand, in the negative direction, when the wing wall was in compression, the horizontal reinforcement of the wing-wall yielded first and the specimens were basically in the shear-tension yielding mode. Except for the specimen SWT-L, which failed in shear followed by losing the axial load carrying capacity, the mode of the other specimens (SWT-LW, SWT-SC and SWT-SV) were switched from the shear-yielding to the flexural type of failure in the final stage after crushing of the wing wall to induce the yielding of the column main bars. The specimens SWT-LW with twice reinforcement ratio and SWT-SC with short wing wall showed the flexural failure mode with high deformability in the positive direction. The specimen SWT-SV subjected to varying axial load showed lower stiffness and strength but higher ductility than SWT-SC under the constant axial load.

The shear strengths calculated by the conventional design equation were generally higher than the observed ultimate shear strengths of the column with one-sided wing-wall. The error in estimation was more in cases of SWT-L and SWT-LW with longer wing wall. Proposed formula by the cumulative method with divided sections provided conservative gave fair estimates of the ultimate shear strengths observed in the tests on the columns with the wing wall irrespective of its wing wall length.

Acknowledgement

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