Evaluation on Deformability of Reinforced Concrete Columns with Wing Walls

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SUMMARY:
A series of tests on reinforced columns with wing walls were conducted from 2007 to 2010 at Earthquake Research Institute. Following the shear tests in the first three years, flexural tests were conducted for six specimens in the final year to investigate the flexural deformability of the members. The effects of the moment-to-shear ratios of loading, the reinforcement details and the width and length of the wing walls were investigated. Although the strength decay after yielding was different, all specimens showed ductile and stable behaviour in flexural failure mode up to the maximum loading drift level. The stable behaviour was owing to the inelastic energy dissipation by the wing walls by which the damage to the column was relatively relieved. The observed ultimate strengths and deformations are compared with calculation using the proposed design form practice, by which fair correlations are obtained.

Keywords: flexural theory, ultimate deformation, compressive strain, confinement, damage control

1. INTRODUCTION

Use of columns with wing walls is a simple design option to provide a reinforced concrete frame structure with relatively high column strength. A series of tests on reinforced columns with wing walls were conducted from 2007 to 2010. In the first three years, mainly shear tests have been conducted to verify the method for evaluating the shear strength with the members of irregular section (Kabeyasawa 2008, Tojo 2008). As for the shear strength, the design formula has been proposed (Kabeyasawa, 2007c) and verified through the results of these first shear test series. The method is called as "cumulative method," formulated as the sum of the column and wing walls, by which a fair estimation of shear strength could be provided. Therefore, if the shear strength was made higher than the shear at ultimate flexural strength, then flexural yielding would be expected preceding to the shear failure. In this case, the strength decay would occur at the compressive failure of wing walls, where the first deformability would be limited.

The columns with wing walls may be used in hyper-earthquake resistant systems (Kabeyasawa 2007b, c) as described below, where the overall beam-yielding mechanism is to be ensured. Even in this case, the deformability is required at the column bases in the first story. Therefore, following the shear tests in the first three years, flexural tests were conducted for six specimens in 2010, to investigate the deformability after flexural yielding (Kabeyasawa, 2011). The effects of the moment-to-shear ratios of loading, the reinforcement details and the width and length of the wing walls were varied. The specimens with thin wing walls showed strength decay after the ultimate strength in flexure, due to the compression failure of concrete and buckling of the re-bars at the wall ends. However, this is the first deformability while much larger ultimate deformability can be attained without loss of axial capacity to the end owing to energy dissipation preliminarily by wing walls. The observed ultimate strengths and the first deformability are compared with calculation using the proposed design form in practice, by which fair correlations are obtained.

2. HYPER-EARTHQUAKE RESISTANT SYSTEM

Full-scale shake table tests on reinforced concrete school buildings were conducted at E-Defense, the world largest three-dimensional shake table, from September to November 2006 (Kabeyasawa 2007b, c). A dynamic progressive collapse of the bare frame specimen occurred associated with shear and axial failure of the short columns in case of fixed foundation, while an obvious input loss was observed in case of swaying foundation. Based on the test result, the authors have proposed "hyper-earthquake resistant system," as shown in Figure 1, which is based on a simple fail-safe design concept against extreme motions exceeding the design level, consisting of relatively strong superstructure and sway-slip foundation (Kabeyasawa, 2010a). The slip behaviour would occur only 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. Use of columns with wing walls is a simple but cost-effective design option to provide a superstructure with relatively higher capacity, up to required in the hyper-earthquake resistant system.

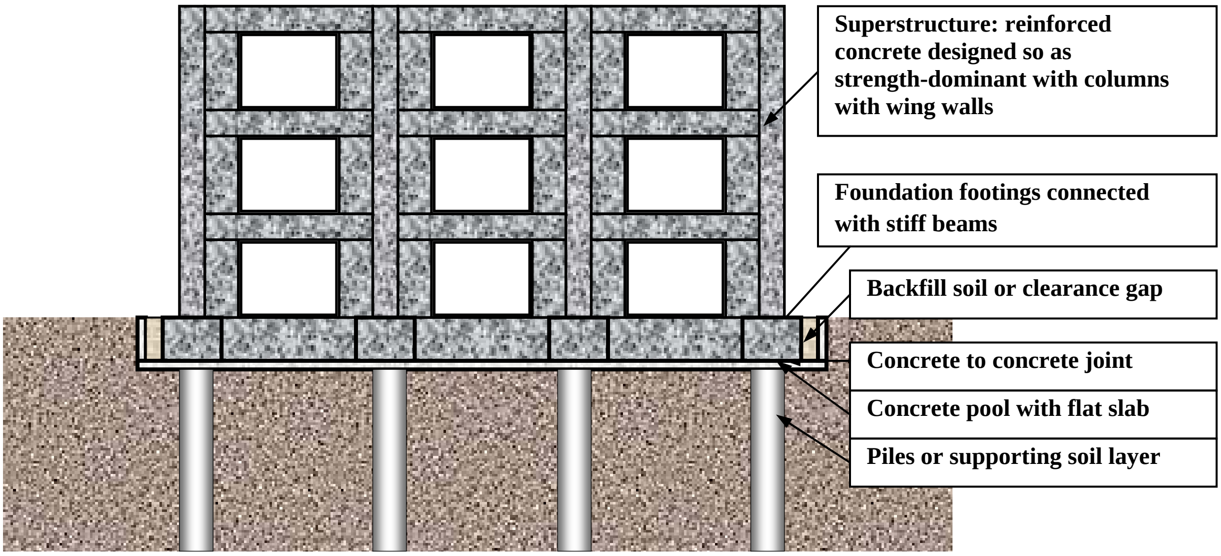


Figure 1. "Hyper Earthquake Resistant System" against extreme motion

3. CUMULATIVE METHOD FOR SHEAR DESIGN IN PRACTICE

Considering the problems in the conventional evaluation methods (JBDPA 2001, BCJ2007) discussed and described elsewhere (Kabeyasawa, 2007a, 2009, 2010b), a modified design formula for columns with wing walls has been proposed, referred as *"Cumulative Method with Divided Sections."* The proposed method evaluates the shear strength of column with wing walls by accumulating individual shear strength of wall and column based on conventional practical design equations for shear(Eqs. (1), (2), (3)), where the sections are divided into the direction of the wall length as shown in Figure 2. The point in the method is to take longer shear span in evaluation of column shear strength.

$$Q_{su} = Q_{suw} + Q_{suc} + 0.1N \tag{1}$$

$$Q_{suw} = \left\{ \frac{0.053 p_{twe}^{\ 0.23}(F_c + 18)}{\dfrac{M}{Qd_w} + 0.12} + 0.85\sqrt{p_{wh}\sigma_{why}} \right\} t_w j_w \tag{2}$$

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.

$$Q_{suc} = \left\{ \frac{0.053 p_{tce}^{\ 0.23}(F_c + 18)}{\dfrac{M}{Qd_{ce}} + 0.12} + 0.85\sqrt{p_{cwe}\sigma_{cwy}} \right\} b_{ce} j_{ce} \tag{3}$$

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, $j_c = 7/8 \cdot d_{ce}$.

The results of verification on the shear strength evaluation with the past test data [Kabeyasawa, 2008, 2009, 2010][Higashi, 1973, 1974, 1974, 1976][Tajiri, 2009][Uehara2010][Iso, 2010] are shown in Figure 3, comparing the conventional method in the current practice of seismic evaluation[JBDPA, 2001] as shown in the figure (a) and the cumulative method as above as shown in the figure (b). It may be concluded obviously from the figures that much better correlations can be obtained from the new formula by cumulative method.

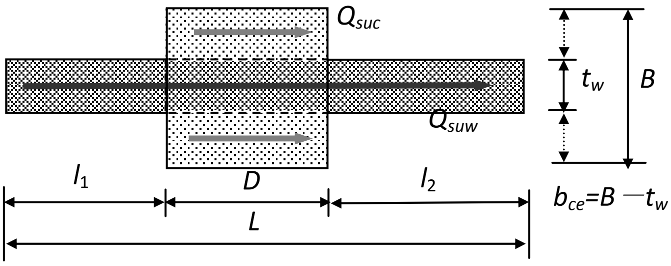
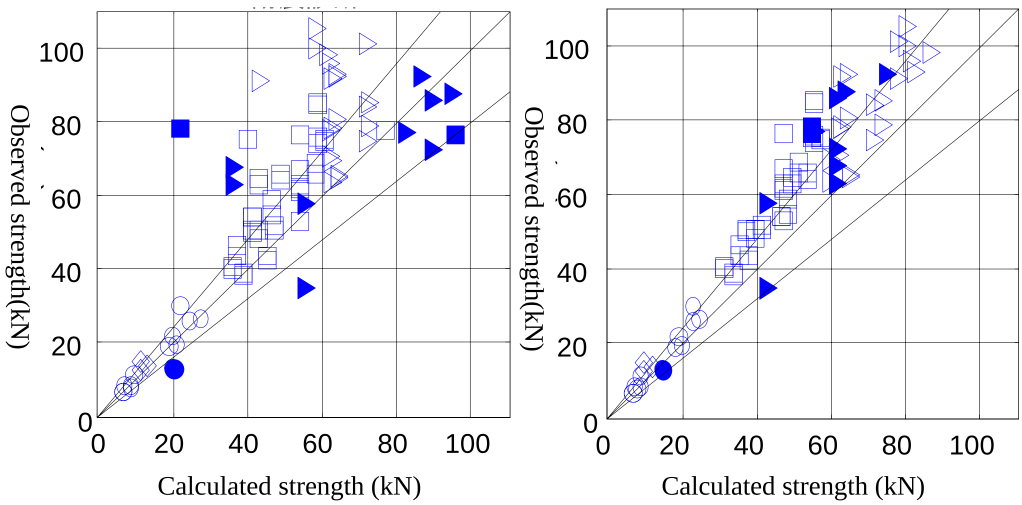


Figure 2. Dividing the column section with wing walls in Cumulative Method



(a) Current method (JBDPA, 2001)　　(b) Cumulative Method (Kabeyasawa, 2007)

Legend for test laboratory and specimen type: ▶:ERI, one-sided, ▷:ERI, both-sided, ■:BRI, one-sided, □:BRI, both-sided, ●:TMU, one-sided, ○:TMU, both-sided.

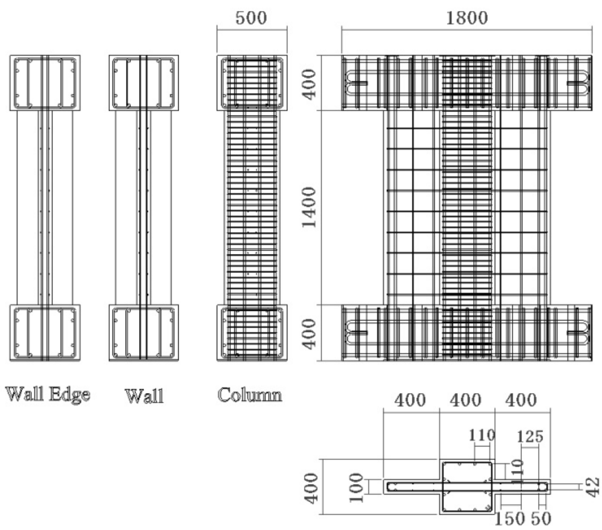
Figure 3. Comparison of observed and calculated shear strengths from past tests

4. TEST SPECIMENS

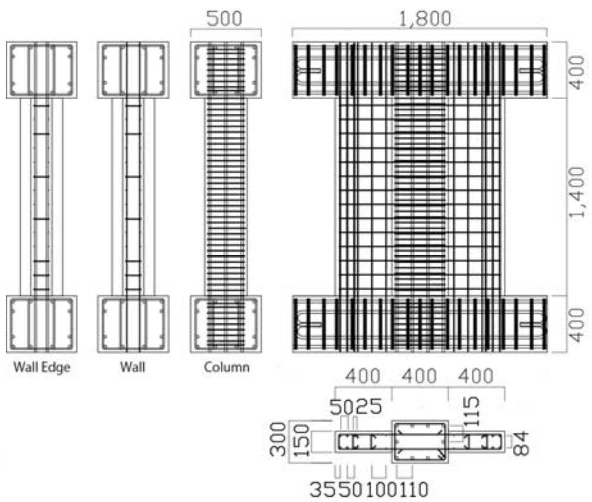
The test specimens and methods for the flexural test in 2010 are described here. The characteristics of six specimens, SWF1 through SWF6 in 2010, are listed in Table 1. The scale of specimens is half or two-thirds of typical sections of full-scale medium-rise buildings in Japan. The section and reinforcement details of each specimen were planned following the previous tests on columns with wing walls for shear tests (Kabeyasawa 2008 and Tojo 2008), as shown in Figure 4.

The specimens SWF1 and SWF2 have identical section and reinforcement details with a specimen in the series, where the thickness of the wing walls was 100mm and the wall length was 400mm attached to both side of the column with the section size of 400mmx400mm. The column main bars are 12-D16 (1.49%) and the hoops were 2-D6@40(0.40%). The shear reinforcement of the wing walls were 2-D6@150 (0.43%) for both horizontal and vertical directions. The longitudinal reinforcing bars of 4-D13 were arranged at the wall edge for flexural resistance. The ratio of the edge bars is much higher than required by the minimum requirement in current design practice of Japan, which is 2-D13 in full-scale. The shear span ratio of loading was changed between SWF1 and SWF2 selected to fail in flexure for both. As for the specimens SWF3, SWF4 and SWF5, the thickness of the wing walls was increase to 150mm. Instead, the column size was reduced to 400mmx300mm, depth of 400mm and width of 300mm in the loading direction, so that the total sectional area was made the same as that of SWF1 and SWF2. The main bars of the column were reduced to 10-D16(1.66%). The amount of the shear reinforcing bars in the wing walls was increased to 2-D6@100 while the ratio was the same as 0.43% for the thickness of 150mm. The wall edge longitudinal reinforcing bars were also increased to 6-D13. The wall edge of the specimen SWF5 was specially confined with closed square hoop of D6 at the spacing of 50mm.

Only the moment-to-shear ratio of loading was changed also between SWF3(SWF5) and SWF4. As for the specimen SWF6, the wall thickness was 100mm but the wall length was made longer to 600mm. The column size was reduced to 400mmx300mm, depth of 300mm and width of 400mm, so that the total sectional area was kept also as the same as the others. The main bars of columns and the longitudinal bars at the wall edges were anchored and welded to the steel plates at the faces of the loading beams while straight anchorage length of 40 times the nominal diameter was taken for longitudinal D6 bars in the wall. The horizontal shear reinforcing bars in the wall were anchored and lapped with splice length of 240mm in the columns and closed at the wall edge. The compressive strength of concrete was slightly varying from 26.1 through 28.5MPa as shown in Table 1, measured from the material test conducted at the age of testing each specimen. The yielding strengths of D6, D10 and D16 were 349, 342 and 351 MPa as shown in the note of Table 1.



(a) SWF1 and SWF2



(b) SWF3 and SWF4

Figure 4. Details of specimens: Column with wing walls

Table 1. List of specimens(SWF1-SWF6)

Name	Concrete Strength (N/mm^2)	Column (mm)			Wing walls on both sides				Axial Load (kN)	M/Q (mm)
		Width x Depth	Main Bars	Hoop	Length (mm)	Thick (mm)	End Re-bars	Shear Re- bars		
SWF1	26.9	400x400	12-D16 (1.49%)	2-D6@40 (0.40% or 0.53%)	400	100	4-D13	2-D6@150	800	2400
SWF2	27.5									1800
SWF3	26.1	300x400	10-D16 (1.66%)		400	150	6-D13	2-D6@100 (0.43%)		2400
SWF4	28.3	300x400					6-D13			1800
SWF5	28.5	300x400					6-D13*			2400
SWF6	27.8	400x300			600	100	6-D13	2-D6@150		3000

*Special confinement detail for end region with closed hoops of D6@50 for the specimen SWF5
#1 Yield strengths of steel, D6, D13, D16: 349, 342 and 351(N/mm^2), respectively
#2 Tensile strengths of steel, D6, D13, D16: 489, 499 and 517(N/mm^2), respectively
#3 Nominal sectional area of reinforcing bars, D6, D13, D16: 32, 127 and 200(mm^2), respectively

The shear strength under lower shear span ratio has been identified by the former shear test series. Also the shear strength could be estimated by a proposed cumulative method [Kabeyasawa, 2008]. Based on the calculation using the former equation, the shear span to depth ratio of loading was selected so that the calculated shear strengths were higher by 1.1 - 1.2 times so that the flexural ultimate strength would be achieved before the shear failure would occur in all specimens. Also the effects of the loading would be comparable among the specimens. The shear span of loading for SWF1 and SWF2 were selected as 1800mm and 2400mm, where the shear span to depth ratio was 1.5 and 2.0, respectively. This was also the case for the two specimens SWF3 and SWF4. The loading condition of the specimen SWF5 was the same with SWF3 so as to investigate the effect of the confinement at the wall edge. As for the specimen SWF6 with total depth of 1500mm, the shear span to depth ratio was selected as 2.0, which was the same as that of the specimens SWF1, SWF3 and SWF5, so the shear span to be 3000mm. The same constant axial load (N = 800 kN) was applied for all specimens. The axial load ratio for the column area was 0.2, that is N / (Ac*Fc) = 0.2, by using the column sectional area Ac of 400mmx400mm and Fc as compressive concrete strength of 25MPa. The axial load ratio for the total area including the wing walls was 0.2/1.5=0.13.

The test set-up at ERI laboratory was used for the loading system as shown in Figure 5. The constant axial load was applied with the two vertical oil jacks, each applying 400kN. Then the lateral load Q was applied by the horizontal oil jack at the height of 2m, while the corresponding varying axial load (DN) in proportion to the lateral resistance was also applied at the top of the specimen with the two vertical oil jacks to maintain the constant target moment-to-shear ratio or the shear span of loading h0 as planned and shown in Table 1 (h0=2+3DN/Q(m)). The three jacks are controlled simultaneously by monitoring the loading conditions. The lateral loading is also controlled based on the displacement amplitudes, which were reversed at the peak drift ratios of ±1/400, ±1/300 each amplified twice and up to +1/12.5 which is limited by the stroke capacity of the horizontal oil jack.

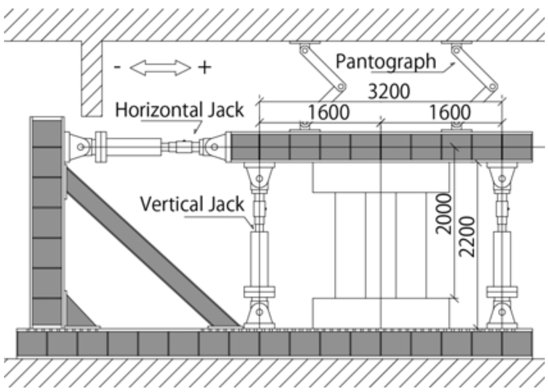
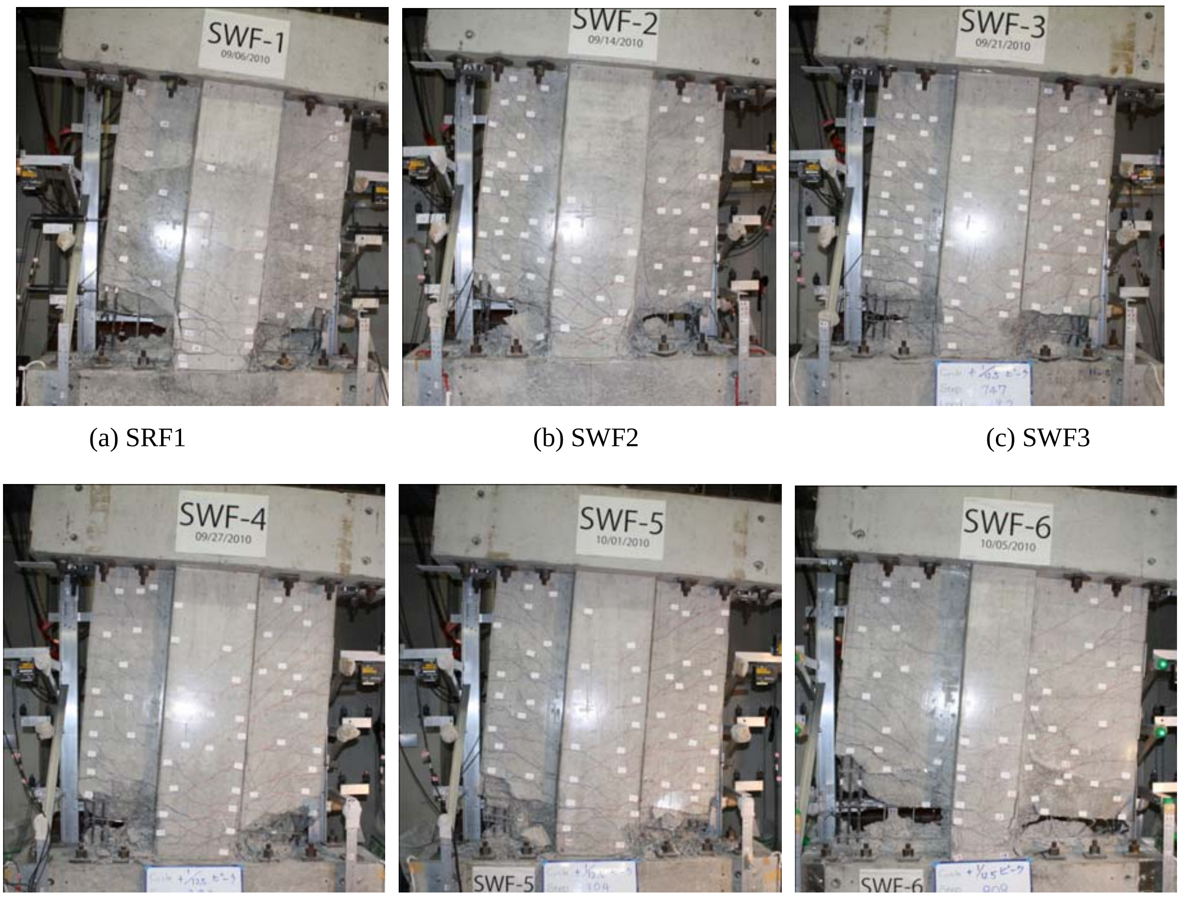


Figure 5. Test set-up at Earthquake Research Institute, University of Tokyo.

5. TEST RESULTS

The six specimens generally failed in flexural mode following the similar process though the deformation levels were different for each specimen. The detailed process of each specimen is not described here, but the behaviour is described briefly in common. Flexural bending and shear cracks occurred and progressed from the wing walls at the first loading cycle of 1/400rad. Longitudinal reinforcement at the wall edge yielded then the maximum strengths were attained at 1/200 to 1/100 (1/50 in SWF5) in a flexural mode. Then apparent or slight strength decay was observed due to the compression failure of concrete and buckling and rupture of the re-bars at the wing wall ends under the deformation amplitudes greater than the maximum. Crushing of concrete was observed in 1/200 to 1/100 cycles of loading in the specimens SWF1, SWF2 and SWF6 with thin wing walls. As for the specimens SWF3, SWF4 and SWF5 with thick wing walls, the slight crushing of cover concrete was observed in the same deformation levels, and the additional damage was much less at the larger deformation amplitudes.

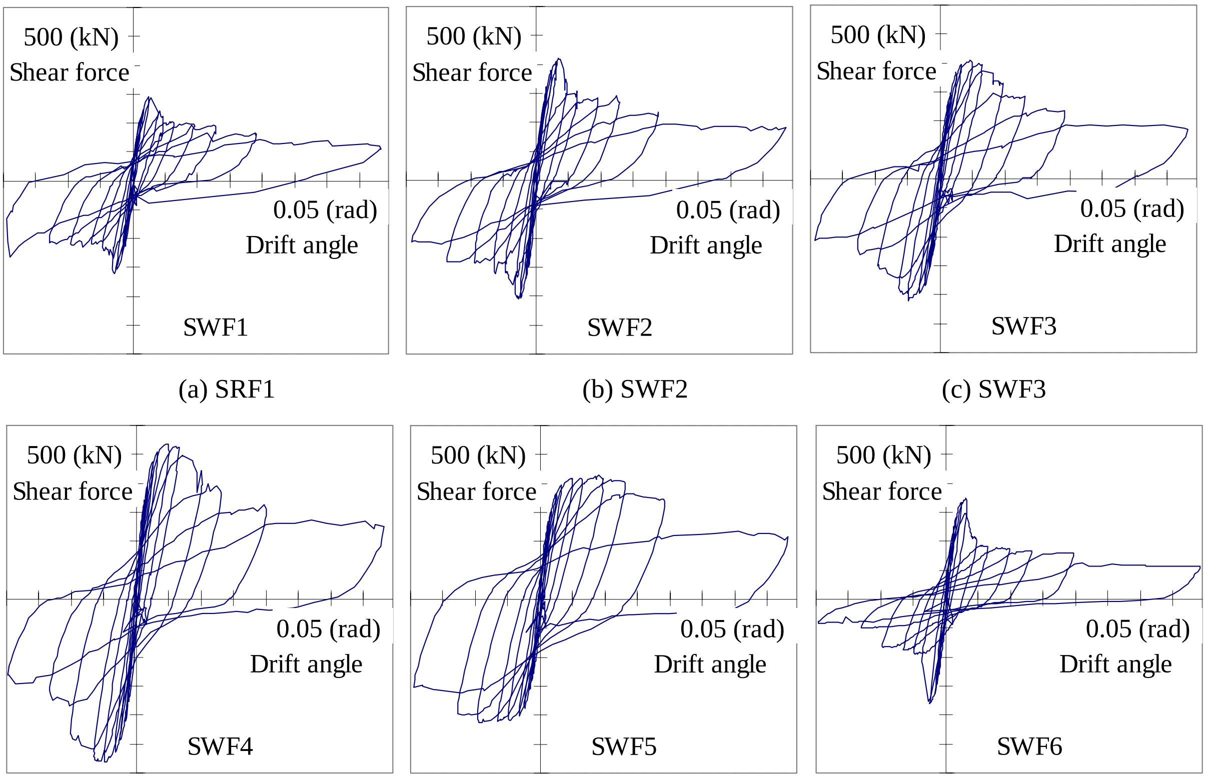
The final failure states are in Figure 6 and the measured peak values are summarized in Table 2, such as the maximum strengths in positive and negative directions with the drift rotations when the strengths were attained. The observed deformability marked with the asterisk (*) in the table was defined at the deformation with the strength decay to 80% of the maximum strengths after the peaks. The observed hysteretic responses of the specimens are shown in Figure 7 between the lateral shear forces and the lateral deformation at the upper loading beam level in terms of rotation angles. Although the strength decay was different, all specimens basically showed ductile and stable behaviour in the flexural failure mode of column after crushing of walls up to the maximum loading drift of 1/12, owing to the inelastic energy dissipation by the wing walls, by which the damage to the column might have been relatively relieved.



(a) SRF1 (b) SWF2 (c) SWF3

(d) SRF4 (e) SWF5 (f) SWF6

Figure 6. Observed failure modes at the maximum loading deformations.



(a) SRF1 (b) SWF2 (c) SWF3

(d) SRF4 (e) SWF5 (f) SWF6

Figure 7. Observed hysteretic responses: lateral load vs displacement in terms of drift rotation angles.

Table 2. Observed and calculated strengths and deformations

Title	SWF1	SWF2	SWF3	SWF4	SWF5	SWF6
Maximum strength positive (kN)	293	421	409	535	427	351
Drift angle at max strength (rad)	0.0052	0.0073	0.0098	0.0098	0.0189	0.0065
Maximum strength negative (kN)	-323	-409	-422	-564	-427	-360
Drift angle at max strength (rad)	-0.0061	-0.0056	-0.0104	-0.0102	-0.0196	-0.0052
Calculated flexural strength (kN)	284	379	347	462	347	322
Maximum strength/calculated	1.14	1.11	1.21	1.21	1.22	1.12
Observed deformability* (rad)	0.0087	0.0096	0.0189	0.0189	0.0385	0.0069
Observed deformability* (rad)	-0.0067	-0.0090	-0.0143	-0.0204	-0.0256	-0.0063
Calculated neutral axis (mm)	293	421	409	535	427	351
Calculated deformation R_u (rad)	0.00811	0.00811	0.0129	0.0129	0.0258	0.00584

* the ultimate deformability is defined at the 80% strength decay from the peak

6. CALCULATION OF ULTIMATE STRENGTHS AND DEFORMATIONS

The ultimate flexural moment resistance of the specimens at the base section was calculated based on plastic flexural theory, using simple design equations derived and proposed for the columns with wing walls as in the forms of (4) through (7):

$$M_u = \sum (a_t \cdot \sigma_y \cdot j_t) \cdot + N \cdot j_N \tag{4}$$

where, a_t and σ_y: area (mm^2) and yield strength (N/mm^2) of tensile longitudinal reinforcing bars, which includes all bars in the tensile regions, though the bars close to the neutral axis may be ignored,
j_t: effective distance between the tensile longitudinal reinforcing bars and the center of compressive concrete stress block ($=d_t - L_{cc}$) (mm), d_t : effective depth of the bars from compressive fiber(mm),
N: constant axial load(N), j_N : axial load ($=L/2 - L_{cc}$) (mm) , A_{cc}: area of compressive concrete block(mm^2), here given by equation (2), ignoring the compressive bars and reduction due to substitute to full-plastic concrete block, which would have compensating effects:

$$A_{cc} = \frac{\sum (a_t \cdot \sigma_y) + N}{F_c} \tag{5}$$

and, the centroid of compressive concrete stress block from the compression fiber (mm) L_{cc} can be given by the following equations (6) or (7):

$$L_{cc} = A_{cc} / (2t_w) \qquad \text{if} \quad A_{cc} \le A_{w1} \tag{6}$$

$$L_{cc} = \frac{A_{w1}}{A_{cc}} \cdot \frac{L_{w1}}{2} + (1 - \frac{A_{w1}}{A_{cc}})(L_{w1} + \frac{A_{cc} - A_{w1}}{2B_c}) \qquad \text{if} \quad A_{cc} > A_{w1} \tag{7}$$

where, t_w and L_{w1}: thickness and length of the compressive wing wall (mm), $A_{w1}=L_{w1}*t_w$,: area of the wing wall, B_c: width of column(mm). The equation (7) is to formulate the centroid of T-shaped compressive region of the wing wall and the column in total area of A_{cc}.

The flexural strengths calculated from above formula for the six specimens are shown also in Table 2 and Figure 5(a), in comparison with the observed ultimate strengths, which is taken as either of the highest values of the strength in the positive and negative loading directions. In case of SWF1 and SWF2, eight bars out of twelve are counted as the tensile bars and the four bars in the compressive side are to be neglected, while all ten column bars are to be considered as tensile bars in cases of SWF3 through SWF6. The ratios of the measured maximum strength to the calculated strength were 1.14, 1.11 and 1.12 for the specimens SWF1, SWF2 and SWF6 with thin wing walls, while they are 1.21 to 1.22 for the specimens SWF3, SWF4 and SWF5 with thick wing walls. The ratios are generally higher in the latter case, probably because the deformation levels at the maximum strengths are larger owing to the stable behavior of the compressive concrete block in the cases of thick walls, so that the effects of strain hardening in the tensile bars were much higher.

The calculation on the ultimate deformability is proposed here and formulated based on a simple flexural model as follows. The ultimate deformation angle R_u is assumed in proportion to the ultimate curvature ϕ_u and the compressive hinge zone length l_h in the form as:

$$R_u = c \times l_h \times \phi_u \tag{8}$$

and, the ultimate curvature may be given using the neutral axis x_n, and the ultimate compressive strain of concrete ε_{cu},, in the form as:

$$\phi_u = \varepsilon_{cu} / x_n \tag{9}$$

The compressive hinge zone length is here assumed in proportion to, for example as observed in the test, taking the twice of the wall thickness as:

$$l_h = 2t_w \tag{10}$$

Then, the ultimate deformation may be written in the form as:

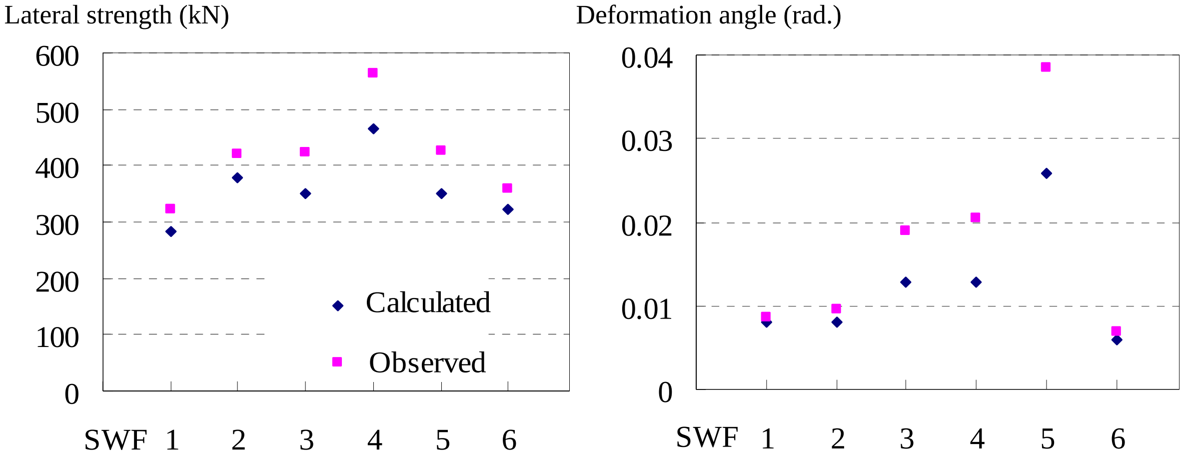
$$R_u = c \times 2t_w \times \varepsilon_{cu} / x_n \tag{11}$$

By taking the inner end of the compressive concrete in the same forms as the equations (5.2), (5.3) and (5.4), the neutral axis x_n can be derived as follows:

$$x_n = 2L_{cc} = A_{cc} / t_w \qquad \text{if} \quad A_{cc} \le A_{w1} \tag{12}$$

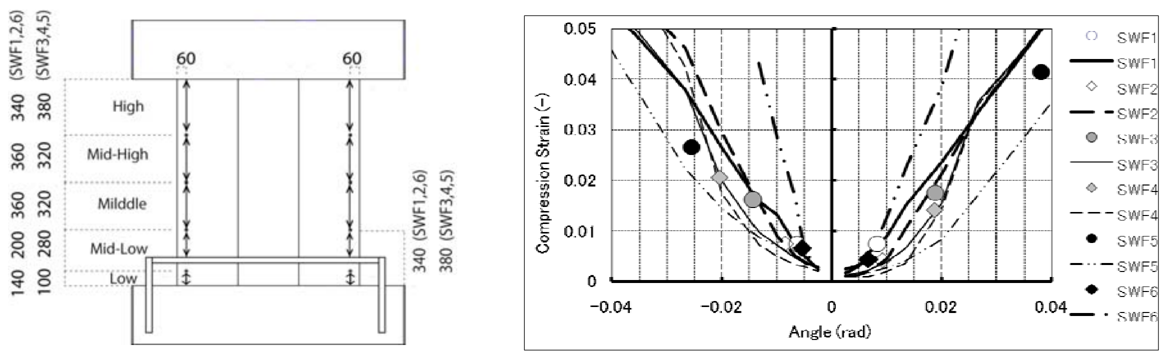
$$x_n = L_{w1} + \frac{A_{cc} - A_{w1}}{B_c} \qquad \text{if} \quad A_{cc} > A_{w1} \tag{13}$$

The ultimate strain is taken as ε_{cu}=0.003 for the unconfined concrete, and ε_{cu} =0.006 in case of the confined end region for the specimen SWF5. The constant factor c is determined here empirically as c=6. The factor reflects theoretically the effects of additional deformation components, such as (1) the elastic deformation besides the hinge region, (2) the elastic and inelastic shear deformation, and (3) the very conservative assumption on the ultimate strain. The ultimate deformability in the test is defined as the deformation at 80% of the maximum strength in the skeleton of the hysteretic responses, and larger of positive or negative, and are compared with the calculated results are shown in Table 2 and Figure 8(b). The calculation gives a fair and conservative estimation of the observed deformability varying with the test parameters, such as the wall thickness, the length and the confinement detail. The assumptions above are further being verified through other past test data, especially on the ultimate compressive strains in the hinge regions of concrete. The measured strains in the test are shown in Figure 9, where the higher values are observed than 0.006 increasing with the thickness of the walls. These values should be investigated and discussed further in details, also in relations with the reference lengths for measurement as well as three-dimensional behaviour and confinement.



(a) Ultimate strength (kN) (b) Ultimate deformation capacity (rad)

Figure 8. Observed and calculated ultimate strengths and deformation capacity (SWF1 through SWF6)



(a) Location of displacement gauges (b) Compressive strains at base

Figure 9. Observed compressive strains at ultimate deformation capacity (SWF1 through SWF6)

6. CONCLUSIONS

The seismic behaviour of columns with wing walls are investigated by a series of tests conducted recent years. The new method of evaluating the ultimate shear strength is verified through the test data, which gives better correlations than the current method. The results of the flexural tests in 2010 are described varying shear span ratio, wall thickness and length and confinement details. The ultimate strength could be approximated well by a simple design equation based on flexural theory. The observed ultimate deformations at 20% strength decay from the maximum were compared with the calculation, constantly factored from the simple theoretical flexural deformations assuming the ultimate strain of concrete and the length of compressive hinge region. The calculation gives a fair correlation with the observed variation with the test parameters, though the constant factor needs be verified further.

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