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Experimental and analytical study on load paths of RC squat walls with openings

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Two series of experimental tests on reinforced concrete (RC) squat walls with irregular openings were analysed. The experimental evidence indicated that flanges significantly affect the failure modes, initial stiffness, ultimate strength and deformation capacity. Given the complexity of the stress distribution in walls with irregular openings, strut-and-tie models are generally used in design and analysis to explicitly demonstrate the shear force transfer and load distribution paths. A comparison of experimental data with analytical results from the existing strut-and-tie model indicated that the latter may underestimate the ultimate strength by 51%. Although the strut-and-tie model is a lower-bound solution, the underestimation of 51% is still very large and may lead to unfavourable shear failure. An improved strut-and-tie model was thus developed, based on test observations. The improved strut-and-tie model can provide a more accurate prediction for the ultimate strength of walls with irregular openings. Based on the model, design recommendations and suggestions for reinforcement details in RC walls with irregular openings are also presented.

Notation

f'_c	concrete cylinder compressive strength
f_u	ultimate strength of reinforcement
f_y	yield strain of reinforcement
h_w	height of wall
K_i	initial stiffness of wall
l_{C2}	distance between compression forces C_1 and C_2
$M_{B(+)}$	moment capacity of the section
$M_{Bp(+)}$	moment calculated according to the plane section hypothesis without consideration of the influence of compression force C_2
P	design strength
P_f	ultimate lateral loading
P_i	predicted ultimate strength of wall
P_n	nominal strength
Δ	top displacement of wall
Δ_y	yield displacement
δ	top drift ratio of wall
ε_y	yield strength of reinforcement
θ	angle of resultant compressive strut in panel
λ_f	flexural reduction factor due to shear transfer
λ_o	overstrength factor for steel
λ_P	shear proportion carried by bottom panel
μ_Δ	displacement ductility factor
$\phi_{o,w}$	flexural overstrength factor

Introduction

The function of reinforced concrete (RC) walls in buildings is primarily to resist lateral loads imposed by wind or earthquakes. Over recent decades, extensive studies (Kim and Han, 2013; Li *et al.*, 2016; Mun and Yang, 2015; Peng and Wong, 2011; Salonikios *et al.*, 1999; Sanjayan, 2000; Wallace, 1995; Wibowo *et al.*, 2013; Wood, 1990; Zygouris *et al.*, 2013) have been carried out to evaluate the reliability of the current design provisions of ACI 318-08 (ACI, 2008) and NZS 3101 (SNZ, 1995) for the seismic resistance of RC walls. However, studies regarding RC walls with irregular openings are relatively few, even though they are included in the design provisions of ACI 318-08 and NZS 3101. In addition, no specific design procedures for RC walls with irregular openings are provided in ACI 318-08 (ACI, 2008) or NZS 3101 (SNZ, 1995). As suggested in the design codes, the strut-and-tie method has been used as one of the main study approaches due to the complex stress distribution in these walls. Ritter (1899) and Morsch (1902) first proposed the strut-and-tie method as a truss analogy, a century ago, and it has been refined and improved significantly since the 1960s (ASCE-ACI Committee 445, 1998; Schlaich *et al.*, 1987). Strut-and-tie models were developed to fulfil the need for improving the detailing of reinforcement, particularly in regions of structures where plane sections do not remain plane after bending. It has been alleged that inappropriate

design of such discontinuous regions of RC elements has been one of the main causes of the poor performance or failure of some structures.

Paulay and Priestley (1992) presented an analytical model to determine the strength capacity of a truss-type structure, which can be applied to structural walls with irregular openings. The model enables selected ties to attain structural yielding while keeping the remaining elements in an elastic mode. This approach presumes walls with ductile (flexure) failure modes while avoiding brittle (shear) failure modes. However, experimental results reported by Li and Wu (2005) and Yanez *et al.* (1992) indicated that the existing strut-and-tie method proposed by Paulay and Priestley (1992) would underestimate the ultimate strength of walls with irregular openings by 51%, which is undeniably quite large. Thus, if existing strut-and-tie models are utilised for flexural design, the flexural reinforcement may over-reinforce and thus result in walls failing in brittle shear rather than flexural failure. Therefore, to improve the prediction of the ultimate strength of walls with irregular openings, existing strut-and-tie models should be refined.

In this study, finite-element analysis was conducted to better understand the complex stress distribution in walls with irregular openings and therefore determine the direction of the principal load path of the walls in conjunction with experimental observations. For this purpose, an improved strut-and-tie model including the shear-resistant contribution of local struts was developed. For easier application by design engineers, design recommendations are proposed and a series of explicit reinforcement details are also provided in this paper.

Experimental programme

Designed specimens

Two series (S-F and S) of six one-third scaled RC walls were studied to examine the seismic behaviour of RC walls with irregular openings. The S-F series was tested by Li *et al.* (2016) at Nanyang Technological University, Singapore, while the S series tests were conducted by Yanez (1993) at the University of Canterbury, New Zealand. As the main experimental results have already been described by Li *et al.* (2016), this paper will briefly describe the specimen design and test results and focus on the discussion of the load path in RC walls with irregular openings and design recommendations to improve the behaviour of RC walls with irregular openings. The specimens were tested under quasi-static reversed cyclic loads to simulate earthquake action. The principal parameters of the specimens are flanges, opening ratio (size of the opening) and location of the openings. Figure 1 shows the dimensions and reinforcement details of the specimens. For the S-F series, the vertical and horizontal reinforcement consisted of two types of reinforcing bars: T10 (deformed rebar of diameter 10 mm) and R10 (plain rebar of diameter 10 mm). For the S series, deformed steel

bars, identified as HD, were utilised as both vertical and horizontal reinforcements. As shown in Figure 1, the S-F specimens had similar dimensions and reinforcement details as the corresponding S specimens, save for the absence of flanges at the wall boundaries.

Material properties

The average cylinder compressive strengths measured on the days of testing of S-F2, S-F3, S-F4, S2, S3 and S4 were 36.9, 35.0, 36.3, 23.0, 26.0 and 23.0 MPa, respectively. The properties of the reinforcing bars are given in Table 1.

Loading sequence and instrumentation

Details of the test setup are shown in Figure 2. As shown in Figure 3, reversed cyclic lateral loading was applied on the top of the wall according to the loading sequence recommended by Park (1989). Initially, force control was applied before the wall reached the design strength P , in increments of $\pm 0.25P$. After reaching the design strength, the displacement-controlled method was evoked. The increase in displacement was based on the ductility factor $\mu_{\Delta} = \Delta/\Delta_y$, where Δ is the imposed top drift while Δ_y is the structural yield displacement. This method has commonly been utilised in previous studies (Fenwick and Bull, 2000; Priestley, 2003). As shown in Figure 4, two methods were used to determine the yield displacement. In the first method, the yield displacement Δ_{y1} is defined as the intersection of the secant stiffness through the first yield strength with nominal strength P_n in the backbone curve. For prediction purposes, the yield displacement Δ_{y1} corresponds to the assumed yield strength of $0.75P_n$. In the second method, the measured yield displacement Δ_{y2} is defined as the intersection of the secant stiffness through the measured yield strength with the nominal strength P_n in the lateral load-displacement hysteretic curve. The average value of Δ_{y1} and Δ_{y2} was utilised in this work.

The specimens were extensively instrumented to monitor global responses as well as local behaviour. As shown in Figure 5, a series of linear variable displacement transducers (LVDTs) was installed to monitor rotational, shear and translational displacement. The shear deformation was measured by diagonally installed LVDTs. Two LVDTs were horizontally installed at positions slightly above the wall base to measure sliding of the wall base. Another three LVDTs were installed to monitor slip of the foundation. The strains in the steel bars at critical regions were monitored using pre-installed strain gauges.

Experimental results

Figure 6 compares the hysteretic response of the S-F specimens with that of the corresponding S series specimens. As shown in the figure, flanges increased the ultimate strength and the initial stiffness by 93% and 202%, respectively. However, the maximum top drift decreased if flanges were incorporated. For the S series, all specimens exceeded the drift ratio (DR)

Figure 1. Dimensions (in mm) and reinforcement details of test walls with irregular openings: (a) S-F2 and S2; (b) S-F3 and S3; (c) S-F4 and S4

	f_y : MPa	ε_y	f_u : MPa
R10	382	0.001913	481
T10	467	0.002612	541
HD8	475	0.002260	690

Table 1. Measured steel bar properties

of 2%, which was not achieved by any of the specimens in the S-F series. Comparing the results of S4 and S2, it was found that walls with small openings could achieve larger strength and deformation capacity than walls with larger openings. Comparison of the results of S3 and S2 shows that walls with openings further away from the boundaries could achieve larger strength and deformation capacity than walls with openings close to the boundaries. However, for the S-F series of specimens, walls with openings further away from the boundaries (S-F3) achieved a lower maximum strength than that of the wall (S-F2) with an opening close to the boundaries. This was an unexpected finding.

Figure 7 shows a comparison of the crack patterns and failure modes of the S-F specimens. As illustrated in the figure, sliding shear failure controlled the failure modes of S-F2 and S-F4, while flexural failure with concrete crushing and rebar fracture in the flanges controlled the failure modes of S-F3. The crack patterns in S-F2, S-F3 and S-F4 are asymmetric in the negative and positive load directions due to irregular openings. It should be noted that the dashed lines in the figures represent cracks in the negative load direction, while continuous lines represent cracks in the positive load direction. The S series of specimens showed crack patterns similar to the corresponding S-F series of specimens, as detailed elsewhere (Yanez, 1993; Yanez *et al.*, 1992). Thus, flanges in the wall

boundaries do not affect the crack pattern of walls even with irregularly distributed openings, as the direction of the cracks normally coincides with the direction of compressive struts in the strut-and-tie models. Hence, similar strut-and-tie models should be utilised to design both series of specimens, but the models in positive and negative load directions should be different for tested specimens.

The displacement ductility is defined as

$$1. \quad \mu_{\Delta} = \Delta / \Delta_y$$

As the yield displacement Δ_y and ultimate displacement Δ will affect the displacement ductility capacity of tested specimens significantly, Δ_y and Δ need to be defined properly. The ultimate displacement (at failure stage) is here defined as the displacement corresponding to 20% degradation from ultimate strength. The yield displacement Δ_y is defined as shown in Figure 4. In addition, the table shows the initial stiffness (K_i) values, defined as the ratio of yield strength to yield displacement. The yield displacements obtained from the first method are generally larger than those obtained from the second method. This can be explained by the fact that the measured yield strength is less than $0.75P_n$.

Table 2 lists the strength and displacement ductility capacity of the S-F and S series specimens based on the yield displacement determined by the hysteretic curve method. As shown in the table, the ultimate strengths of the S-F series specimens were normally reached at $\mu_{\Delta} = 2.0-2.5$. However, the ultimate strength of the S series specimens was reached at $\mu_{\Delta} = 3.4-4.8$ in the positive direction and at $\mu_{\Delta} = 3.6-6.7$ in the negative direction. Generally, the S series specimens showed a much higher displacement ductility capacity than the corresponding S-F series specimens.

Figure 2. Details of the test setup

Figure 3. Loading history

Figure 4. Methods to determine the yield displacement of tested specimens

Figure 5. LVDT positions of S-F2

(a)

(b)

(c)

Figure 6. Comparison of load–displacement responses of flanged walls and corresponding rectangular walls

Figure 7. Crack pattern and failure mode of S-F series of specimens with irregular openings: (a) S-F2; (b) S-F3; (c) S-F4

Existing strut-and-tie models for the design of walls with openings

As developed by Yanez *et al.* (1992) and Paulay and Priestley (1992), the possible strut-and-tie models for the S-F and S series were similar; they are illustrated in Figures 8 and 9, respectively. In these models, the flanges were assumed as axially tensile or compressive components, while the horizontal ties were located in the middle beam zone.

According to the strut-and-tie model shown in Figure 8(a), the negative ultimate strength of S-F2 was determined as -214.2 kN, which was controlled by the tensile capacity of vertical tie AG, where 6T10 bars were placed. The ultimate strength (220.0 kN) of the wall in the positive direction was controlled by the tensile capacity of horizontal tie LK. The position of node L in the model for positive loading was determined by the resultant forces provided by the steel bars of 8T10. The

		S-F2	S-F3	S-F4	S2	S3	S4
Δ_y : mm		6.8	8.2	5.3	5.6	5.2	4.1
K_i : kN/mm		55.4	41.0	80.7	21.3	27.9	26.7
$P_{\max}^a$: kN	(-)	395.3	357.8	435.5	228	239	236
	(+)	381.2	358.5	453.6	208	217	235
$\mu_{\Delta P_{\max}}^b$	(-)	2.0	2.4	2.4	6.7	5.3	3.6
	(+)	2.0	2.4	2.5	4.8	3.4	4.3
$\mu_{\Delta \max}^c$		3.8	5.5	6.1	10.4	10.8	14.6

^aUltimate strength

^bDisplacement ductility level at ultimate strength

^cMaximum displacement ductility

Table 2. Strengths and ductility of tested wall specimens

minimum width of strut LQ was determined by assuming the effective compressive strength of the concrete with skew cracks to be $0.68f_c$, as suggested by Schlaich *et al.* (1987). The position of node Q can be determined by a trial-and-error approach when the compressive force of strut LQ is determined. Assuming that the compressive force of strut LQ splays out by 25° from its ends (which is the minimum angle between a strut and a tie as stated in ACI 318-08 (ACI, 2008)), the secondary reinforcement in the panel zone was designed according to the tensile force T . The strut-and-tie model of S-F3 for negative loading was identical to that of S-F2, where the same negative ideal strength of $P_i = -214.2$ kN was achieved. However, significant differences were observed between the models of these two specimens under positive loads. The positive ultimate strength was predicted to be $P_i = 203.3$ kN. The strut-and-tie models of S-F4 were developed in an approach similar to that of S-F3, as shown in Figure 8(b). The negative and positive ultimate strengths were controlled by the tensile capacities of ties BD and LK, respectively. The ultimate strengths of S-F4 in the negative and positive load directions were $P_i = \pm 220.0$ kN. Considering that tie SM helps to transfer the shear force, tie LK became more critical than the vertical ties in the bottom of the model under positive loads.

For S2, S3 and S4, the predicted ultimate strengths in the positive and negative load directions are $+139/-145$ kN, $+139/-140$ kN and $+139/-142$ kN, respectively. However, as mentioned above, the measured ultimate strengths of S2, S3 and S4 in the positive and negative load directions were $+208/-227$ kN, $+217/-239$ kN and $+235/-236$ kN.

The existing strut-and-tie model thus underestimates the ultimate strength of the S series specimens by 40.9%. For the S-F specimens, the existing strut-and-tie model underestimates the ultimate strength by 51.0%. The existing strut-and-tie model suggested by Paulay and Priestley (1992), which is commonly utilised in design, is therefore too conservative for the design of RC walls with irregular openings. Modifications to improve the accuracy are thus necessary.

Improved strut-and-tie models

Load paths of tested specimens

The load paths of a wall with irregular openings subjected to cyclic lateral loading at the top beam can be illustrated by the principal stress flow with the help of finite-element analysis. In this study, the validated commercial software WCOMD-2003 was utilised to visualise the principal stress flow in the walls. The concrete was modelled using a smeared crack model and a joint element model. The smeared crack model is suitable for members with distributed two-dimensional reinforcement. For local discontinuous regions, the joint element model was applied. This program, which combines the advantages of these two types of models, can predict the seismic behaviour of RC walls well (Maekawa *et al.*, 2003; Okamura and Maekawa, 1991). In the fixed crack model, when the principal stress reaches the concrete tensile strength, a crack develops perpendicular to the principal tensile stress direction. However, the crack orientation does not change during successive loading. For the smeared crack model, the generation and propagation of an individual crack is considered an average within a finite region. The constitutive law of cracked concrete in WCOMD-2003 is developed based on the fixed smeared crack approach. It consists of a compression model, a tension stiffening model and a shear transfer model (Maekawa *et al.*, 2003; Okamura and Maekawa, 1991). Modelling of the reinforcing bar in this program takes the effects of bonding between the bar and concrete into account. For modelling local discontinuities, the joint element, which is a one-dimensional interface element with zero thickness, is used. Eight-node quadrilateral isoparametric elements are applied to model RC and plain concrete plates.

Figures 10–12 illustrate the principal stress flows of the S-F series of specimens at two critical loading stages: yield displacement and ultimate displacement. As shown in the figures, the principal stress flows of the S-F specimens in these two stages were similar. Identical strut-and-tie models can thus simulate the load path of the wall at different loading stages. Moreover, as shown in the figures, the column zones could

(b)

Developments and details of improved strut-and-tie models

one tie. Its position is located in the centroidal axis of the combined reinforcements. The strut position is determined by ensuring that the concrete compressive stress is lower than the strength limitations, as suggested by Schlaich *et al.* (1987): $0.68f'_c$ for a strut with parallel cracks and $0.51f'_c$ for a strut with skew cracks.

Figures 13 and 14 show the improved models for S-F2 and S2, respectively. According to the above assumptions, the tie area and capacity can be determined. Table 3 lists the main properties of the struts in the improved strut-and-tie models for S-F2 to S-F4. Although the geometry of the struts is

Figure 9. Strut-and-tie model of S series specimens as suggested by Paulay and Priestley (1992): (a) S-F2; (b) S-F4

Figure 10. Principal stress flow of S-F2

Figure 11. Principal stress flow of S-F3

Figure 12. Principal stress flow of S-F4

difficult to determine as they may have bottle-shaped stress fields, the struts can be idealised into a prismatic or uniformly tapered shape according to the geometry of the nodal zone. For a concentrated node, its geometry can be clearly quantified by the boundary of the bearing plate or tie. In the case of walls with irregular openings, where generally no bearing plate is present, the geometries of the nodes and struts are determined based on the divisions of the beam and column zones and the positions of the openings. Normally, the strut widths are uniformly tapered and the nodes are not hydrostatic, which is commonly applied in corbels or deep beams. Figure 15 presents the strut details of S-F2; the extended nodal zones are also depicted. Table 4 lists the main properties of the ties of the strut-and-tie models for S-F2 to S-F4. The strut width applied to determine the strut strength was the smaller dimension perpendicular to the axis of the strut.

In order to achieve a ductile failure mode, the critical component in a proper strut-and-tie model should be the tie, rather than the strut or a node. Tables 3 and 4 indicate that the most critical component in the improved strut-and-tie models of S-F2 is tie BD when the wall is subjected to negative loads, and ties MR and NS when the wall is subjected to positive loads. The strut stresses are normally lower than the stress limitation suggested by Schlaich *et al.* (1987). However, struts QW and BE achieved relatively higher compressive strengths than the others. The strength of strut QW is even higher than the limitation ($0.68f'_c$). This is because (a) the angles of struts QW and BE are determined by the shear proportions carried

by the columns, which are represented by these struts, and (b) the strut widths are measured from the opening edges to their central lines, which results in underestimation of the strut widths. The influences of flanges at the wall boundaries were not incorporated. Experimental observations indicated that the upper end of strut QW was crushed at a load stage that was much later than the stage corresponding to the ultimate strength. The mechanisms of columns to resist shear will be discussed in more detail later in the paper.

For the improved strut-and-tie model of S-F3, the strength of strut AB exceeded the strength limits. The higher compressive strength achieved in strut AB is because of the strut located at the upper panel, where the rigid top beam will restrict its deformation. The strengths of the struts in the columns are lower, mainly due to the wider columns in S-F3. In the model of S-F4, the strengths of most of the struts were lower than the limit of $0.68f'_c$. Similar to S-F3, this is due to the rigidity of the top beam and the fact that strut AB was located at the upper panel. Moreover, S-F4 had the widest columns of the three walls. The strength of a non-hydrostatic node can be determined by the method proposed by Marti (1985), through which a CCT node can be transferred into a CCC node (CCT represents a nodal zone bounded by two or more struts and a tie, and CCC a nodal zone bounded by three or more struts). The stress states of nodes Q from all of the walls with irregular openings, which are the most critical nodes in the improved strut-and-tie models, are shown in Figure 16. As shown in the figure, node Q connected to struts LQ and QW and tie NQ.

Figure 13. Improved strut-and-tie model for design of S-F2

Figure 14. Improved strut-and-tie model for design of S2

Tie NQ is transferred into a strut pressing the node behind, and the edges of the struts and tie intersect at points A, B and C, which define the nodal region. The forces of struts LQ and QW and tie NQ are represented by F_C , F_B and F_A , respectively. The strength in the nodal zone is evaluated based on the steps suggested by Marti (1985).

- Step 1: Find out the poles Q_A , Q_B and Q_C from Mohr's circle of an individual strut.
- Step 2: Draw parallel lines to the sides AB, BC and AC.

- Step 3: Determine the biaxial stress within the triangle ABC.

For S-F2, a high principal compressive stress ($1.03f'_c$) is obtained because of the high compressive strength of strut QW. Nodes Q in the strut-and-tie models of S-F3 and S-F4 were evaluated in a similar way and resulted in principal compressive stresses of $0.59f'_c$ and $0.47f'_c$, respectively (see Figures 16(b) and 16(c)). As shown in Figure 14, S2 had a similarly improved strut-and-tie model to the corresponding S-F2.

Wall and direction	Strut	Angle: degrees	Area: mm ²	Loading factor	$\sigma_{\text{cmax}}^{\text{a}}$: N/mm ²	σ_{cmax}/f_c
S-F2: Negative	AB	41.5	31 800	1.335	11.2	0.30
	BE ^b	78.7	10 560	0.903	22.9	0.62
	DG	39.8	37 200	1.071	7.70	0.21
	FG	58.3	20 400	0.337	4.42	0.12
S-F2: Positive	KL	38.2	29 640	1.272	13.2	0.36
	LQ	53.1	21 600	1.666	23.8	0.64
	QW ^b	77.6	11 040	1.364	38.1	1.03
	NU	50.5	19 200	1.110 ^c	17.8	0.48
	MU	35.7	13 920	0.871 ^d	19.3	0.52
S-F3: Negative	AB	44.4	16 800	0.740	26.0	0.74
	BE ^b	73.2	17 760	1.024	17.98	0.51
	DG	39.8	20 400	0.919	14.05	0.40
	FG	58.3	20 400	0.563	8.61	0.25
S-F3: Positive	KL	43.3	36 960	1.374	10.08	0.29
	LQ	52.8	18 720	1.654	24.0	0.68
	QW ^b	71.8	18 720	1.386	20.1	0.57
	NU	60.6	19 200	1.155 ^c	16.3	0.46
	MU	38.9	13 920	0.729 ^d	14.2	0.40
S-F4: Negative	AB	42.2	16 080	1.350	27.8	0.77
	BE ^b	68.5	45 360	0.974	7.11	0.20
	DG	43.8	40 800	0.891	7.24	0.20
	FG	60.2	20 832	0.718	11.4	0.31
S-F4: Positive	KL	40.8	36 960	1.321	11.9	0.33
	LQ	55.5	28 800	1.766	20.5	0.56
	QW ^b	68.5	35 760	1.563	14.6	0.40
	NU	58.4	9600	—	7.64 ^e	0.21
	MU	41.2	40 800	0.568 ^d	4.65	0.13

^aStrength developed in strut when maximum strength of model is reached

^bStrut angle determined by the shear taken by the column it represents

^cAssuming strut NU carrying all force of tie NQ

^dAssuming strut MU carrying all force of tie NQ

^eStress calculated by assuming tie NS yields

Table 3. Main properties of struts in improved strut-and-tie models for S-F2 to S-F4

Verification against experimental and analytical results

Comparison of Figures 8 and 13 shows that the improved strut-and-tie models and the strut-and-tie models suggested by Paulay and Priestley (1992) have the following differences.

- The compressive struts in column zones are considered to carry part of the shear force.
- The column zone in the centre of the wall is subjected to tension when under either positive or negative loads.

- When negative loads are applied, tie OD appears in the upper section of the right column zone while ties EF and FH appear in the lower panel.

The improved strut-and-tie models were verified against the crack patterns and strain gauge readings.

Verification against crack patterns

As shown in Figure 17, the directions of compressive struts coincide well with the directions of the diagonal cracks. When

Figure 15. Strut details of S-F2

the wall was subjected to negative loading, short diagonal cracks were observed in the region where tie EF is located. However, few horizontal cracks were observed in the upper column at the location of tie OD. This indicates that the tensile force in tie OD is low. When the specimen was subjected to positive loading, diagonal cracks in the upper panel zone were flatter than those in the middle panel zone. Horizontal cracks were also observed in the middle column zone and some diagonal cracks in the bottom corner panel zone.

As shown in Figure 17, the shear force can only be transferred through beam and column zones due to the irregularly distributed openings. Thus, the diagonal sections connecting the corners of openings and passing through the nodal zones are critical for shear transfer. Force equilibrium could be established according to Figure 17. It should be noted that the reinforcement in the beam and column zones played very important roles in transferring the lateral force.

Verification through strain gauge results

The improved strut-and-tie models were verified against strain gauge readings obtained from the tests. The strain gauge readings from S-F2 are presented and discussed as an example. As illustrated in the improved strut-and-tie model of S-F2 subjected to positive loading, the shear force is transferred to the first storey via struts KL and LQ by behaving as a bilinear arch. The arch action requires a constant tensile force in tie KM along its entire length. However, tie MR will develop a larger tensile force than KM due to the diagonal strut developed in the first storey, as shown in Figure 17.

Figure 18 illustrates the results of strain gauges attached to one of the vertical bars located in the left flange, which were

lumped into ties KM and MR. As shown in the figure, all three strain gauges recorded considerable tensile strain when positive load was applied. Strain gauge #15, corresponding to tie MR, recorded a much higher tensile strain than #40 and #60, which corresponded to tie KM. However, no significant tensile strain was recorded in these strain gauges when negative load was applied, which agrees well with the improved strut-and-tie model.

Figure 19 presents the results of strain gauges attached to one of the vertical reinforcements located in the central column zone, which corresponded to ties LT and AI for positive and negative loads, respectively. As shown in the figure, tensile strain was observed in all gauges for both positive and negative loading. However, #57 recorded limited tensile strains under positive load while it recorded considerable tensile strain under negative loading. This was mainly due to the fact that the vertical rebar located in the upper part of the centre column zone only provides tensile force in negative loading. Similarly, the results from strain gauges of #67, #68 and #69 provided evidence for tie NQ, as shown in Figure 20. For ties BD, NS, OD and DJ, the strain gauge results also proved the accuracy of the improved strut-and-tie models. The strain gauge results from S-F3 and S-F4 are shown in Figures 21–24 and, similarly, the strain gauge results agree well with the strut-and-tie models.

Proportions of shear force carried by columns

To determine the percentage of the shear force transferred by beam zones and column zones, analytical analysis based on test observations and finite-element analysis (WCOMD-2003) was conducted. The shear force taken by a column zone can be estimated by subtracting the shear proportion taken by the beam zones. The shear transferred by beam zones can be evaluated

Wall and direction	Tie	Lumped reinforcements	Capacity: kN	Loading factor
S-F2: Negative	OD	8R10 + 2T10	313.2	0.262
	DJ	8R10 + 2T10	313.2	0.948
	AI	6T10	220.0	0.622
	FH	4T10	146.6	0.268
	BD	6T10	220.0	0.823
	EF	4T10	146.6	0.177
S-F2: Positive	KM	8R10 + 2T10	313.2	0.787
	MR	8R10 + 2T10	313.2	1.295 ^a
	NS	4T10	146.6	0.858 ^b
	LT	6T10	220.0	0.545
	NQ	6T10	220.0	0.707
S-F3: Negative	OD	8R10 + 2T10	313.2	0.274
	DJ	8R10 + 2T10	313.2	0.865
	AI	6T10	220.0	0.706
	FH	6T10	220.0	0.479
	BD	6T10	220.0	0.704
	EF	4T10	146.6	0.296
S-F3: Positive	KM	8R10 + 2T10	313.2	0.942
	MR	8R10 + 2T10	313.2	1.40 ^a
	NS	4T10	146.6	1.01 ^b
	LT	6T10	220.0	0.375
	NQ	6T10	220.0	0.567
S-F4: Negative	OD	10R10 + 2T10	373.2	0.292
	DJ	10R10 + 2T10	313.2	0.909
	AI	6T10	220.0	0.614
	FH	2R10 + 4T10	206.6	0.624
	BD	6T10	220.0	0.643
	EF	2R10 + 4T10	206.6	0.357
S-F4: Positive	KM	10R10 + 2T10	373.2	0.863
	MR	10R10 + 2T10	373.2	1.237 ^a
	NS	2T10	73.3	0.694 ^b
	LT	6T10	220.0	0.592
	NQ	6T10	220.0	0.427

^aAssuming strut MU carrying all force of tie NQ

^bAssuming strut NU carrying all force of tie NQ

Table 4. Main properties of ties in improved strut-and-tie models for S-F2 to S-F4

based on measured strain gauge readings. For example, the readings from gauge #69 of S-F2 were utilised to represent the strains of the beam zone located at the lower part of the wall, as shown in Figure 20. It should be noted that concrete in the beam zones was also taken into account, based on the tension constitutive law proposed by Belarbi and Hsu (1994). The

concrete area was calculated based on the area of the RC zone, as proposed by An and Maekawa (1998). Figure 25 illustrates the proportions of shear force carried by the compressive struts located in the column zones of S-F2. As shown in the figure, both the experimental and numerical results indicate that the proportion of shear force carried by section 1–1 is significant

Figure 16. Strength of node Q in S-F series of specimens

Figure 17. Crack patterns and schematic illustration of the improved strut-and-tie models for S-F2

Figure 18. Readings of strain gauges in vertical rebar of left flange of S-F2

Figure 19. Readings of strain gauges in vertical rebar of middle column of S-F2

when the specimen is subjected to positive loads, while that of section 2–2 becomes significant when the specimen is subjected to negative loads. However, the shear force carried by section 1–1 began to drop when a large DR was achieved. This was primarily due to high axial load, and concrete crushing began to occur in the struts when DR became large. With an increase in the number of load cycles, no obvious decrease was observed in the proportion of the shear force contribution in section 2–2 (middle column). This agrees well with both the test observation (no concrete crushing occurred there) and the strut-and-tie model (relatively lower axial force was determined). The

experimental results were generally higher than the numerical results. This may be because the tensile strains in the reinforcement of the beam zone were underestimated, as only some key points were measured by the gauges. Moreover, the shear force transferred by additional mechanisms such as aggregate interlocking was not explicitly quantified and was subsumed to the contribution of the compressive struts in the column zones.

The numerical results indicated that considerable shear force was transferred from the third storey to the second storey by the upper column zone corresponding to section 3–3.

Figure 20. Readings of strain gauges in horizontal rebar of lower beam zone of S-F2

Figure 21. Readings of strain gauges in vertical rebar of left flange of S-F3

The contribution of the column zone in accordance with section 3–3 is about 13% and 10% for S-F2 subjected to negative and positive loading, respectively. This finding is utilised to illustrate the underestimation of the improved strut-and-tie model in the following section.

Ultimate strengths estimated by improved strut-and-tie models

The finite-element results (WCOMD-2003) and crack patterns indicated that the local struts in the bottom panel zone would help to resist the shear force and should be included in the model. The improved model thus generated local struts at the

bottom panel zone, as shown in Figures 13 and 14. Analysis of the improved strut-and-tie models indicated that the ultimate strengths of S-F2 are controlled by tie BD under negative loading and ties MR and NS under positive loading. The ultimate strengths of S-F2 under negative and positive loading were determined to be -267.3 kN and 308.5 kN, respectively; these values are about 70% and 80% of the measured negative and positive ultimate strengths. For S-F4, ties BD and FH controlled the negative strength while ties MR and NS controlled the positive strength. This indicates that the ultimate strength of this specimen was related to the amount of reinforcement in the bottom panel. The ultimate strengths of

Figure 22. Readings of strain gauges in vertical rebar of middle column of S-F3

Figure 23. Readings of strain gauges in vertical rebar of left flange of S-F4

S-F4 predicted by the improved strut-and-tie models were -331.3 kN and 333.7 kN, which are 76% and 74% of the experimental ultimate strengths, respectively. Compared with the existing strut-and-tie models, the improved models thus predicted the ultimate strengths of the test specimens much better, but they still cannot yet predict the experimental ultimate strengths of the walls satisfactorily.

Flexural behaviour of wall with irregular openings

The improved strut-and-tie models developed in the previous section reflect the main force transfer mechanisms of walls with irregular openings. However, the ultimate strengths of the

walls are still not predicted with sufficient accuracy. The results indicated that the walls achieved their ultimate strength at early loading stages, when most vertical reinforcing bars had yielded while the horizontal reinforcing bars were still elastic. It is thus possible that the ultimate strength of walls with irregular openings is still controlled by flexural behaviour and can be estimated by flexural design.

The main characteristic of flexural design is the linear distribution of strain along a plane (plane section hypothesis). Figure 26 plots the average strains of S-F2 at the first storey, which were detected by the LVDTs installed at the boundaries and the middle sections of the walls. As shown in the figure, the

Figure 24. Readings of strain gauges in vertical rebar of middle column of S-F4

Figure 25. Proportions of shear force carried by columns of S-F2

strains are close to linear distributions. Similar behaviour was also observed in the other walls. Table 5 lists the ultimate lateral loading, P_f , calculated by assuming the plane section hypothesis applied in the bottom sections for either negative or positive loading. As shown in the table, a flexural design method could predict the ultimate strength in the negative direction well. However, this method may slightly overestimate the positive ultimate strength. This reflects the different characteristics of force equilibrium on the sections under positive and negative loading.

Figure 27 illustrates the force equilibrium on the bottom section of walls with irregular openings. Under negative loading, only one compression force occurs at one side of the

plane. It is thus similar to the force distribution of a solid wall. This type of flexural equilibrium can be regarded as a flexural type. Compared with the forces under negative loading, one more compression force, C_2 , is observed in the bottom section under positive loading, which reduces the flexural capacity of this bottom section. This type of flexure can be regarded as a flexural-shear type, and the flexural capacity of the section is reduced due to the transfer of shear force.

The flexural capacity of a flexural-shear section can be conceptually estimated for a cantilever wall with irregular openings based on the force equilibrium of the section plotted in Figure 27. Assume that $M_{Bp(+)}$ is the moment calculated

Figure 26. Average strain of first storey of S-F2. (1) represents the first cycle in a drift ratio stage

according to the plane section hypothesis without consideration of the influence of C_2 . Given the moment capacity of the section is $M_{B(+)}$, the following equation between the two moments can be drawn according to the improved strut-and-tie models

$$2. \quad M_{B(+)} = M_{Bp(+)} - (\lambda_P V \cdot \tan \theta) l_{c2}$$

where λ_P is the shear proportion carried by the bottom panel, l_{c2} is the distance between compression forces C_1 and C_2 , θ is the angle of the resultant compressive strut in the panel and $\lambda_P V$ is the horizontal component of C_2 . Let $V = M_{B(+)} / h_w$, where h_w is the wall height. Modifying Equation 2 yields

$$3. \quad M_{B(+)} = \frac{M_{Bp(+)}}{1 + \lambda_P \cdot l_{c2} / h_w \cdot \tan \theta}$$

The flexural reduction factor due to the shear transfer, λ_f , can also be obtained based on Equation 4.

$$4. \quad \lambda_f = \frac{1}{1 + \lambda_P \cdot l_{c2} / h_w \cdot \tan \theta}$$

The reduction factor is determined based on the shear distribution between the column zone and the panel zone, the opening width and the angle of the strut to carry the shear force in the bottom panel. As λ_P and l_{c2} can be estimated according to the improved strut-and-tie models, and the strut angle θ can be calculated as the average strut angles in the models, the ultimate lateral loads were computed and are listed in Table 5. As shown in the table, the calculations and experimental results match well.

Design recommendations

Ultimate strength

The strut-and-tie model and capacity design procedure are the basis of the design of a wall with irregular openings. Generally, a design using a strut-and-tie model is conservative due to the fact that the strut-and-tie model is inherently a lower-bound solution. However, if the capacity design procedure is based on an improper strut-and-tie model, the walls may be over-reinforced in flexure and this may result in a shear failure mode. The following two measures, based on the previous analysis, can be applied to avoid this brittle failure mode. Firstly, a strut-and-tie model that can predict the ultimate strength reasonably should be chosen. Secondly, the over-strength factors should be determined based on ultimate flexural and shear strengths. The ultimate flexural strength of a flexure type section of a wall with irregular openings can be determined by the plane section hypothesis, as discussed in the previous section. However, the flexural strength of a flexure-shear type section needs to include the strength reduction factor, because shear force transferring at the bottom column may decrease its flexural strength. As indicated in this study, a wall with an aspect ratio close to 1.0 is still able to attain flexural controlled failure modes. Therefore, a strut-and-tie model that can predict the ultimate strength of walls similar to their flexural strength obtained by cantilever design could be utilised as a proper model for the capacity design of walls with irregular openings. In order to avoid brittle shear, the over-strength factor can be determined by the ultimate flexural strength obtained by cantilever design and the maximum shear strength as predicted by the strut-and-tie model. The following steps can be used to determine the over-strength factor for the loading case shown in Figure 27.

- The flexural moments of the wall in two loading directions $M_{Bo(+)}$ and $M_{Bo(-)}$ are determined by considering the steel over-strength $\lambda_o f_y$, where λ_o is the over-strength factor for steel.

	S-F2		S-F3		S-F4	
	Negative	Positive	Negative	Positive	Negative	Positive
P_f^a : kN	-352.1	409.8	-357.4	403.5	418.4	421.6
P_e^b : kN	-395.3	381.2	-357.8	358.5	-435.5	453.6
P_f/P_e	0.89	1.08	1.0	1.25	0.96	0.93
$\lambda_f P_e$	—	333.2	—	323.8	—	382.2

^aAssuming plane section hypothesis can be applied to the bottom section under positive and negative loadings

^bExperimental maximum strengths

Table 5. Maximum strengths estimated by flexural capacities

Figure 27. Strut-and-tie models and force equilibria of bottom section of walls with irregular openings in two loading directions

- The flexural over-strength factor is computed by $\phi_{o,w} = M_{Bo}/(Ph_w)$, where M_{Bo} is the higher value out of $M_{Bo(+)}$ and $M_{Bo(-)}$, P is the lower value of the strengths predicted by the strut-and-tie models in different loading directions and h_w is the wall height.

Deformation capacity

In order to improve the deformation capacity of the walls, based on test observations, the following series of detailed requirements is suggested.

- Enough confinement must be provided to the column zones to increase their capacities in sustaining compression and shear.
- Enough reinforcement should be provided to horizontal beam zones that are critical to sustain lateral loading in

order to ensure that these zones can sustain the additional shear force transferred from column zones after the concrete crushes.

- The bottom panel zones should be strengthened, as that is where high compression and shear forces may concentrate.
- Boundary elements should be added to the walls to help carry the combined action of the loads, as shown in the previous analysis.
- The openings should be kept at a certain distance from the boundary to avoid compressive failures occurring in the small column zones.

Figure 28 shows a schematic illustration of the improved reinforcement details for a wall with irregular openings, including confinements on the column zones and nodes and diagonal reinforcement in the bottom panel. The confinement in the

Figure 28. Reinforcement requirements for full ductile walls with irregular openings

column zone should be sufficient to ensure that the longitudinal bar will not buckle under compression. To prevent sliding shear failure, diagonal reinforcements should be placed in the bottom panel of the walls. The diagonal reinforcements of the ties, as presented in Figure 28, need to be fully anchored in the nodes with effective hooks. However, the amount of diagonal reinforcement needed requires more investigation. By using the proposed design recommendation and suggested reinforcement details, the behaviour of walls with irregular openings can be significantly improved.

Conclusions

Based on the test results and analytical analysis, the following conclusions were drawn.

- A comparison of the results of two series of tests indicated that flanges could increase the ultimate strength and initial stiffness of a wall with irregular openings by 93.2% and 202.2%, respectively.
- The frequently utilised existing strut-and-tie models proposed by Yanez *et al.* (1992) and Paulay and Priestley (1992) may underestimate the ultimate strength of the S-F and S series of specimens by 51% and 41%, respectively. This is mainly due to the fact the shear resisting capacity from local struts at panel zones is not included in these models.
- Based on test observations and finite-element results, improved strut-and-tie models were developed. The accuracy of the models was verified by comparing crack patterns with the direction of the struts in the models and strain gauge readings. Comparing the ultimate strengths from the improved models with those obtained from tests showed that the improved

models could limit the difference to within 30%.

Although not satisfactory, as the strut-and-tie models are lower-bound solutions and the models applied here predominantly represent only the primary load path inside a wall, they do offer an improvement on the existing models.

- Design recommendations were proposed to help engineers design walls with openings properly and prevent undesirable brittle shear failure. In a flexure type section, where compression and tension zones can be distinguished clearly, the flexural strength can be estimated by the plane section hypothesis. However, for a flexural-shear type section, a flexural reduction factor λ_f should be applied to consider the effects of shear transfer by compressive struts at the columns.
- To improve deformation capacity or ductility capacity, several special reinforcement details were suggested. Transverse reinforcement should also be provided to confine the columns and prevent premature rebar buckling and concrete crushing. Diagonal reinforcement should be included to prevent sliding shear failure at the bottom panel zone, especially for walls with flanges.

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