Seismic Behavior and Analysis of Dual Function Slitted Shear Wall

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Abstract
A new type of passive controlled RC shear wall, called dual function slitted shear wall (DFW wall), was proposed by the first author. DFW wall is formed by setting connectors in the vertical slits of RC slitted wall. DFW wall can provide greater stiffness and strength than RC slitted wall during small and moderate earthquakes, and performs a ductile flexure failure as RC slitted wall with the failure of connectors during severe earthquakes. Experimental studies to investigate the seismic behavior of DFW wall, including seven wall specimens and thirteen connector specimens, were introduced first in this paper. An analytical model was then proposed to calculate the full skeleton load-displacement relation for DFW wall. The analysis results were found to agree well with test ones. A dual seismic structural model was used to study the dynamic response of DFW wall using the time history analysis method. Experimental research and analysis demonstrated the dual control functions, and showed that DFW wall has better seismic performance than the corresponding slitted shear wall.

Keywords: dual seismic structure; seismic control; slitted shear wall; connector; seismic response

Introduction
To improve the brittle shear failure of solid reinforced concrete shear wall (SW), Muto (1974) suggested a slitted wall (SLW) which can develop a ductile flexure failure. However the initial lateral stiffness and strength capacity are much decreased due to the slits introduced, which may result in displacement and strength control problems during small and moderate earthquakes. By setting connectors in the slits, a new type of RC shear wall, called dual function slitted shear wall (DFW wall), is suggested by the first author (1999, shown in Fig. 1). With the connector setting, DFW wall performs somewhat like a solid wall with greater stiffness and strength during small earthquakes (or service load) and moderate earthquakes (Function 1). The connectors fail or yield under severe earthquakes so that the wall is converted to a slitted wall automatically with a ductile flexure failure mode (Function 2). Damage can be mainly restricted to the connectors, which are easy to repair, under moderate earthquakes, while damage degree to the main part of the wall is mitigated under severe earthquakes.

Seismic behaviors of DFW wall were first studied by experiment with seven specimens in this paper. The shear stress-deformation relations were also obtained by thirteen connector tests. Based on the experimental results, an analytical model was proposed to calculate the full skeleton lateral load-displacement relation for DFW wall.

Experimental Research of DFW wall
The specimen is a single story frame edged wall, as shown in Fig. 1. A solid wall, a slitted wall and five DFW walls were tested. The five DFW walls and the slitted wall had same geometrical dimensions for frame beam, frame column and wall column. The frame and wall panel dimensions of the solid wall are also the same. The specimen dimensions are shown in Table 1.
The specimens were divided into two groups. There were one DFW wall (DFW1), one slitted wall (SLW) and one solid wall (SW) in Group 1, which was used to compare their behavior. The specimens in Group 2 were all DFW walls. The main research parameters are connector settings, stiffness ratio and strength ratio of connector to wall-column. The connector was made of concrete with reinforcement bars passed through. The specimens’ parameters and material strength are given in Table 2. The specimens were tested under lateral cyclic load at the frame beam axis.

Table 1. Specimen Parameters

| Group | f_c (MPa) | f_y (MPa) | f_r (MPa) | A_{cc} (mm^2) | A_{ch} (mm^2) | A_{sh} (mm^2) | A_{sv} (mm^2) | A_{sc} (mm^2) |
|-------|-----------|-----------|-----------|---------------|---------------|---------------|---------------|---------------|
| SW    | 22.7      | 336       | 255       | 200           | 100           | 0             | 0             | 0             |
| 1     | 20.5      | 27.5      | 100       | 200           | 150           | 150           | 100           | 100           |
| 2     | 31.3      | 26.3      | 260       | 150           | 120           | 120           | 120           | 120           |

Table 2. Specimen Dimensions (mm)

| H_w  | B_w  | t_w  | D   | a  | t_c | b_c×h_c | b_h×h_h |
|------|------|------|-----|----|-----|---------|---------|
| 800  | 1000 | 60   | 225 | 20 | 40  | 150×150 | 150×250 |

Note, H_w: wall height; B_w: wall width; t_w: wall thickness; D: wall-column width; a: slit width; t_c: connector thickness; b_c×h_c: frame column section width and height; b_h×h_h: frame beam section width and height.

Fig. 4 shows the τ-R relation skeleton curve of DFW4 under lateral load, where τ=P/A is the average shear stress across the wall horizontal section and R=ΔH_w/2 is the drift angle. The load process can be divided into five stages:

1. **Elastic stage (OA):** Crack (① in Fig. 2) was first observed in the connectors due to shear. Before cracking, DFW wall behavior somewhat like a solid wall with large lateral stiffness, about 2~3 times larger than that of the corresponding slitted wall.

2. **Elasto-plastic stage after cracking of connector (AB):** The flexure moment in wall-columns then caused horizontal cracks (② in Fig. 2) at the top and bottom sections. With increasing load, more shear cracks were observed in the connectors. At about 0.5P_m (P_m is the maximum lateral force), the cracks width in the connectors was about 0.1~0.2mm. The flexure stress in the wall-column cross sections near the connectors then caused horizontal cracks (③ in Fig. 2). The shear force in the wall-column segments induced some diagonal shear cracks (④ in Fig. 2). A larger deformation was observed when longitudinal reinforcement in the wall-column at the top and bottom sections yielded (Point B in Fig. 4).

3. **Connector began to fail (BC):** After point B, the connector shear cracks developed rapidly and the connector concrete began to crack as load approached the maximum (Point C in Fig. 4).

4. **Connector out of work (CD):** The connector shear stiffness dramatically decreased after the maximum load, which resulted in less connection between the wall-column. The loading capacity of DFW wall decreased quickly. Cracks ③ and ④ did not develope at this stage.

5. **Ductile loading stage (DE):** After the connector concrete had totally crashed and been out of work (Point D in Fig. 4), the wall, except the bars passing through the connectors, converted to a slitted wall. The load capacity became stable with large deformation at this stage. Due to the remaining steel bars in the connectors, the load capacity at this stage was about 10~20 percent higher than that of the corresponding slitted wall.

Fig. 5 compares the τ-R curves of DFW1, SLW and SW.
and SW. It can be seen that the solid wall (SW) had the largest initial lateral stiffness and strength but failed in a brittle shear mode, while the slitted wall (SLW) had the lowest initial lateral stiffness and strength but failed in ductile flexure mode.

Fig. 3(a). Lateral load-displacement hysteresis relations of group 1

Fig. 3(b). Lateral load-displacement hysteresis relations of group 2
As for DFW wall (DFW1), the initial lateral stiffness was a little smaller than SW but larger than SLW, about three times that of SLW in the elastic stage (shown in Table 3). The lateral stiffness decreased as the deformation increased. After the connectors totally failed, the lateral stiffness was almost the same as that of SLW, about 1/50 of its initial value. Thus the connectors can provide proper stiffness control for different earthquake intensities. The structure deformation can be controlled with large lateral stiffness at the initial stage under small earthquakes (or serviceability load), while a less earthquake force response may be resulted in due to a less lateral stiffness under severe earthquakes.

The strength of DFW1 was found between SW and SLW. The maximum strength of DFW1 was about 40% higher than SLW. Thus, larger strength can be provided by the DFW wall under moderate earthquakes compared to the slitted wall. The $\tau$-$R$ curve of DFW1 tended to that of SLW with a slightly higher load capacity after failure of the connectors. A ductile flexure failure mode of DFW1 was finally formed as that of SLW.

The stiffness ratio of connector to wall-column is the main parameter affecting the initial stiffness and maximum strength for DFW wall (Ye and Kang, 1999). Fig. 6 shows the $\tau$-$R$ skeleton curves of Group 2. The connectors in DFW3 were made of steel bars only, while the connectors in DFW4 were made with concrete and the same steel bars as that in the DFW3 connectors. The comparison of DFW3 and DFW4 showed that the connector shear stiffness greatly affects the lateral stiffness of DFW wall. DFW3 performed more like a slitted wall with a slightly increased strength due to the steel connectors. DFW4 had a larger stiffness and strength than DFW3 due to larger shear stiffness of the connectors and the same deformation capacity as DFW3 in the ductile loading stage.

Increasing reinforcement in the connector caused DFW5 to have a higher maximum strength and load capacity in the ductile loading stage than DFW4. The connector of specimen DFW2 was made of concrete only without reinforcement. Although its initial stiffness and maximum strength was larger due to almost the same connector shear stiffness as DFW4 and DFW5, its loading capacity decreased quickly after the connector crashed and resulted in a lower loading capacity in the ductile loading stage compared to the other specimens in Group 2.

The main test results of all specimens are listed in Table 3.

Table 3. Experimental results

| Specimen | SW | SLW | DFW1 | DFW2 | DFW3 | DFW4 | DFW5 |
|----------|----|-----|------|------|------|------|------|
| Cracking load $P_{cr}$(kN) | 34.5 | 14.6 | 20 | 25.1 | 10.3 | 25.34 | 25.73 |
| Cracking displacement $\Delta_{cr}$(mm) | 0.081 | 0.122 | 0.065 | 0.084 | 0.073 | 0.079 | 0.078 |
| Initial stiffness $k_{cr}$(kN/mm) | 427.8 | 119.7 | 306.2 | 299.2 | 141.9 | 317.9 | 329.9 |
| Yielding load $P_{y}$(kN) | -- | 75 | 167.4 | 144.7 | 73 | 130.3 | 145 |
| Yielding displacement $\Delta_{y}$(mm) | -- | 2.0 | 4.85 | 2.15 | 2.36 | 2.85 | 2.12 |
| Maximum load $P_{m}$(kN) | 339 | 157 | 220.4 | 170 | 116.4 | 161.6 | 194 |
| Displacement at maximum load, $\Delta_{m}$(mm) | 7.92 | 13.94 | 9.93 | 3.88 | 11.2 | 9.79 | 7.93 |
| Load at beginning of ductile range, $P_{d}$(kN) | -- | -- | 156.3 | 106 | 116.4 | 118 | 131 |
| Displacement at beginning of ductile range, $\Delta_{d}$(mm) | -- | -- | 20 | 11.9 | 11.9 | 15.5 | 20 |
| Load at End of ductile range, $P_{e}$(kN) | -- | -- | 140.7 | 95.4 | 104.8 | 106.2 | 117.9 |
| Displacement at End of ductile range, $\Delta_{e}$(mm) | -- | -- | 26.3 | 26 | 37.8 | 51 | 68.6 |
Experimental Research of Connector

The connectors are key control components in DFW wall. Finite element analysis showed that the connectors in DFW wall are under vertical shear action (Ye and Kang 1999). To analyse DFW wall, the shear-deformation relation of the connector should be obtained firstly. Thus, experimental research of the connector were conducted. The test setting is shown in Fig. 7. Two vertical opposing forces were applied to induce almost pure shear stress at the center connector component in the specimens. The dash line forces and supports in Fig. 7 were used to apply reversal shear stress in the connector.

The parameters considered in the test are, (1) reinforcement ratio in the connector; (2) compressive stress $\sigma_x$ perpendicular to the slit; (3) connector size; and (4) loading manner (monotonic and cyclic). Thirteen specimens were tested.

The specimen’s dimensions are: wall plate thickness $b=100$mm, connector thickness $t=90$mm and connector width $a=20$mm. The parameters of all specimens are shown in table 4. A typical specimen reinforcement is shown in Fig. 8. The relative vertical deformation $\Delta$ at the two connector sides was measured in the test.

Fig. 7 is a typical $\tau-\gamma$ relation result obtained in the test, where $\tau=P/\theta h$ is the average shear stress in the connector and $\gamma=\Delta a$ is the shear deformation. Due to large shear stiffness, the shear deformation was very small at the initial loading stage and has a linear relation with shear stress. First, shear crack (point A in Fig. 9) was observed at the top or bottom edges of the connector at about a 45-degree angle. More shear cracks were developed following an increase in shear force, while shear stiffness decreased. If reversal shear force was applied, about the same amount of shear cracks were observed in a reversed 45-degree angle. The cracks were found to be concentrated in the part of the connector before the maximum shear force was attained. The shear cracks pattern is shown in Fig.9. The reinforcement yielded a little before the

| No. | $A_s$ | $f_y$ (MPa) | $f_c$ (MPa) | $\sigma_x$ | $h$(mm) |
|-----|-------|-------------|------------|-----------|---------|
| 2-0-0d0 | —     | —           | 26.14      | 0         | 200     |
| 2-3-6f0 | 3φ6  | 310.9       | 26.19      | 0         | 200     |
| 2-3-8d0 | 3φ8  | 316.3       | 26.89      | 0         | 200     |
| 2-3-8f0 | 3φ10 | 254.4       | 22.38      | 0         | 200     |
| 3-5-6d0 | 5φ6  | 310.9       | 26.54      | 0         | 300     |
| 3-5-10d0 | 5φ10 | 254.4       | 24.23      | 0         | 300     |
| 2-10-6d0 | 10φ6 | 228.3       | 34.22      | 0         | 200     |
| 2-10-6f0 | 10φ6 | 228.3       | 38.05      | 0         | 200     |
| 2-10-6d1 | 10φ6 | 228.3       | 29.10      | 0.1$f_c$ | 200     |
| 2-10-6f1 | 10φ6 | 228.3       | 32.81      | 0.1$f_c$ | 200     |
| 2-10-8f0 | 10φ8 | 215.2       | 36.78      | 0         | 200     |
| 2-10-8f2 | 10φ8 | 215.2       | 36.66      | 0.2$f_c$ | 200     |

Note: $A_s$-reinforcement passing through connector; $f_y$-yielding strength of reinforcement; $f_c$-compressive strength of concrete; $\sigma_x$-compressive stress perpendicular to the slit; $h$-height of connector; symbol “d” in test No. means monotonic loading; symbol “f” in test No. means cyclic loading.
maximum load (at point B' in Fig. 9). After the maximum shear force (point B in Fig. 9), the shear strength decreased rapidly due to crash of concrete and the shear deformation increased quickly. As the shear deformation became large enough, the shear force slowly increased again. This is due to the fact that some flexure mechanism was formed in the test as very large relative vertical deformation resulted between the two connector sides. The shear deformation in DFW wall connectors was found to be less than such shear deformation value. Thus, the \( \tau - \gamma \) relation of the connector was idealized as Fig. 10.

The skeleton curve of \( \tau - \gamma \) relation for cyclic loading test was found to be almost the same as that of the corresponding monotonic loading specimen. A pinch hysteresis pattern was observed in \( \tau - \gamma \) relation for the cyclic loading specimen.

(2) Yielding Point B': according the test results, the yielding shear stress \( \tau_y \) is about 90 percent of the maximum shear stress at point B, i.e., \( \tau_y = 0.9 \tau_c \). The yielding shear deformation \( \gamma_y \) was found to have a relationship with the yielding strain of reinforcement, ratio of bar diameter to width of connector and compressive stress \( \sigma_y \). The following formula was suggested based on analysis and test results

\[
\gamma_y = \eta \psi \frac{d}{a} \left( \frac{\rho f'_y + \sigma_y}{\rho f'_x} \right) \frac{f'_x}{E_s} \tag{2}
\]

where, \( \eta \) is an influencing factor of the compressive stress \( \sigma_y \), \( \eta = 1.0 \) for \( \sigma_y = 0 \) and \( \eta = 1.1 \) for \( \sigma_y = (0.1-0.2) f_c \); \( \psi \) is an influencing factor of loading manner, \( \psi = 1.095 \) for monotonic loading and \( \psi = 1.607 \) for cyclic loading based on the test results; \( d \) is the diameter of the reinforcement bar; \( a \) is the width of connector; \( \rho = A_i / th \); \( f'_x \) and \( E_s \) are the yielding strength and modulus of reinforcement respectively. The comparison of calculated shear deformation with the test ones is shown in Fig. 12.

(3) Maximum Point B: the shear strength of connector \( \tau_u \) related to concrete strength, reinforcement ratio and strength. Based on the analysis ultimate stress state of connector, strength criterion of concrete under combined shear and compressive stress and test results, the following formula was suggested (Zhao, Wang and Ye 2001),

\[
\frac{\tau_u}{f_c} = 0.139 + 0.897 \frac{\rho f'_y + \sigma_y}{f_c} - 0.753 \left( \frac{\rho f'_y + \sigma_y}{f_c} \right)^2 \tag{3}
\]

A coefficient 0.9 should be considered for cyclic loading. Fig. 13 shows the comparison of Eq. (3) with the test results.

As for shear deformation at shear strength \( \tau_u \), the following formula was suggested (Zhao, Wang and Ye 2001),

\[
\gamma_u = 0.01 (\rho f'_y + \sigma_y) \tag{4}
\]
in which, the unit of \( f_y \) and \( \sigma_x \) is MPa. Fig. 14 shows the comparison of Eq. (4) with the test results.

(4) Point C: the remnant shear strength and corresponding shear deformation at point C is approximately adopted as \( 0.65 \tau_u \) and \( 6\gamma_u \), respectively, based on the test results.

Analysis of DFW Wall

The analysis model of DFW wall is shown in Fig. 15 based on the following assumptions:

(1) the bottom ends of the wall-column and frame column were fixed, while the top ends were free horizontally but rigid in resisting flexure moments.

(2) the connector was subjected to shear force only and a spring with the shear stress-deformation relation in previous part was used.

(3) the parts in the wall-column at the connector level were regarded as rigid.

(4) with restriction of the outside frame, there exists axial compressive stress in the wall-columns besides the moment and shear forces induced by lateral force. According to the suggestion of Muto (1974), the axial compressive stress in wall-columns was suggested as,

\[
\sigma_a = k \left( \frac{H}{D} \right) \bar{f}
\]

where, \( H_w \) and \( D \) are height and width of the wall-column respectively; \( \bar{f} = V_w/A_w \) is the average shear stress in wall-column; \( V_w \) is shear force in wall-column; \( A_w \) is the cross section area of wall-column and \( k \) is the coefficient, Muto (1974) suggested \( k=0.7 \) before yielding of the wall-column and \( k=0.9 \) at the maximum lateral load.

With the suggested model, full relation of the lateral force and displacement for DFW wall could be obtained by the following nonlinear analysis steps:

(1) With proper axial stress-strain relations of concrete and reinforcement bar and plane remains plane assumption, the moment-curvature relation of the wall-column and the frame-column sections can be obtained first;

(2) To increase curvature at the top end of the wall-column and the frame-column step by step. The corresponding moments were then determined based on the moment-curvature relation obtained previously;

(3) Assuming a shear deformation in the connectors and calculating the corresponding shear force, the lateral load at the top of the wall-column and frame-column can be determined from equilibrium equation;

(4) The wall-column and frame-column were divided into many finite segments, the moments and the corresponding curvature at every section of the segments with the known top end moments and shear force in the connectors were then determined;

(5) Calculating the shear deformation from the rotation of segment at the connector level by accumulating curvature of every segment along the height of wall-column and frame-column;

(6) If the difference between the calculated shear deformation and assumed one was tolerant, then continue to the next step; if not, then repeat steps (3)–(5);

(7) Calculating lateral displacement at the top of the wall-column and the frame-column by accumulating curvature of every segment along the column height;

(8) Repeat steps (2)–(7) to obtain the full lateral load-displacement relation of wall-column and frame-column; the full lateral load-displacement relation of DFW wall can then be obtained by the superposition method.

The lateral load-displacement curves comparison between test and those analyzed are shown in Fig. 16, and a reasonable agreement can be found. The lateral
load-displacement curves of the corresponding slitted wall were also obtained by the above analysis procedure and are shown in the same figure. The difference between the DFW wall and its correspond-
ing slitted wall were used for dynamic analysis in the next part.

Seismic Response Analysis of DFW Wall

To further investigate the seismic performance of DFW wall under earthquake motion, dynamic response analysis was conducted by the time history analysis method. A SDOF dual seismic structure system model, suggested by the first author (Ye and Oyang, 2000), was used for the analysis.

The so-called dual resistant structure system is the structure that has two structure systems (or elements), one is called the main structure system; another is called the secondary structure system (or secondary element).

The main structure has enough strength to sustain the service load but less lateral stiffness so that small seismic force response results when the single main structure is considered. The main structure can also form a proper yielding mechanism with enough ductility so that it can survive a severe earthquake even when the secondary structure is out of function.

The secondary structure (or element) is a supplement to the main structure to control the structure performance, energy dissipation and damage mitigation. The secondary structure (or element) can also be easily repaired or replaced.

Fig. 16. Comparison of calculated lateral load-displacement relations and test ones

Fig. 17. SDOF Dual seismic structure system

Fig. 17(a) shows the dynamic analysis model of the SDOF dual seismic structure system for DFW wall. The main and secondary structures are parallel in order to resist lateral seismic force, i.e., the lateral displacements are the same for the main and the secondary structures, the sum of shear forces of the main and the secondary structures equals to the lateral force response of the system mass. The secondary structure has a less yielding displacement than the main structure, so it yields before the main structure and dissipates energy by its hysteresis loops.
For DFW wall, the corresponding slitted wall is taken as the main structure and the connectors are taken as the secondary elements. The contribution of secondary elements in DFW wall to the lateral load-displacement relation was obtained by extracting that of the corresponding slitted wall from that of DFW wall, as shown in Fig. 17(b).

Based on the test results, a pinch effect was found in the hysteresis relation for DFW wall (Fig. 3). Thus the tri-linear Takeda slip hysteresis model was used in the nonlinear dynamic analysis (Eto and Takeda, 1973). The comparison of the lateral load-displacement hysteresis relation of the test and the Takeda slip hysteresis model of a typical specimen is shown in Fig. 18.

The Newmark-β method was used in the nonlinear time history analysis. El Centro 1940 (NS) earthquake record was used as exciting input. The acceleration amplitude was modified to 100gal, 200gal and 400gal to represent small, moderate and severe earthquake levels.

DFW4 and its corresponding slitted wall were used for analysis. The initial period of corresponding slitted wall was adopted as 0.6sec. The yielding strength of corresponding slitted wall is one-fifth of the elastic force response under severe earthquake input.

| Earthquake level | Max. Disp. Resp.(mm) | Ductility of main structure |
|------------------|----------------------|----------------------------|
|                  | DFW4  | SLW4 | DFW4  | SLW4 |
| Small            | 11.49 | 16.16| 0.67  | 0.53  |
| Moderate         | 28.07 | 36.00| 0.78  | 1.29  |
| Severe           | 64.96 | 72.06| 0.90  | 2.98  |

Table 5 shows the response comparison of DFW4 and its corresponding slitted wall (SLW4 in Table 5). It can be seen that the maximum displacement responses of DFW4 are about 33%, 22% and 10% less than the corresponding slitted wall under small, moderate and severe earthquake intensities, respectively. From comparison of the ductility of the main structure and the corresponding slitted wall, it could be seen that the damage degree of the main structure in DFW wall is also less than the corresponding slitted wall under moderate and severe earthquakes.

Conclusion

Experimental and analytical research in this paper supported the invention of the control function of DFW wall. The seismic performance can be controlled by setting connectors in the slits to satisfy different design criteria under different earthquake levels.

Compared with the corresponding slitted wall, the initial lateral stiffness of DFW wall is about 2–3 times higher, and the maximum strength of DFW wall is about 40–60% higher, which provides proper displacement and loading capability control under small and moderate earthquake motion. After failure of the connectors, the DFW wall is converted to the ductile slitted wall automatically. The stiffness at the ductile stage is about 1/50 of the initial stiffness, which result in less earthquake force response under severe earthquake motion.

The seismic behavior of DFW wall mainly depends on the connector setting and its parameters. The connector shear stress-deformation can be determined by the suggestion in this paper.

The suggested analysis model in Fig. 15 can predict the lateral load-displacement of DFW wall well. With the SDOF dual seismic system model in Fig. 17, the seismic response can be obtained for DFW wall.

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