

ARTICLE

Comprehensive approach for examining shear-critical RC walls using detailed experimental measurements

Eissa Fathalla^{1,2}  | Boyan Mihaylov¹

¹Urban and Environmental Research Unit (UEE), Univ. of Liège, Liège, Belgium

²Structural Engineering Department, Cairo University, Giza, Egypt

Correspondence

Eissa Fathalla, Structural Engineering Department, Faculty of Engineering, Cairo University, Giza 12613, Egypt; Department of ArGenCo, Urban and Environmental Research Unit (UEE), University of Liège, Building B52, Quartier Polytech 1, Allée de la Découverte 9, B-4000 Liège, Belgium.
Email: mmeissa@uliege.be

Abstract

Current design codes lack robust assessment methods for shear-critical non-slender members based on on-site crack measurements. Therefore, in this paper, a comprehensive framework is proposed for quantifying the shear resisting mechanisms of short shear-critical walls on the basis of detailed test results. This framework is applied to two full-scale shear walls loaded to shear failure. The tests were performed with detailed measurements including digital image correlation, providing displacement and strain fields across the entire face of the specimens. The results demonstrate the substantial impact of crack geometry on the relative contribution of the shear resisting mechanisms. Furthermore, it is confirmed that the shear carried by the compression zone, referred to as the critical loading zone (CLZ), has a governing contribution to the total shear. Finally, a set of hypotheses towards a crack-based assessment method are proposed on the basis of the three-parameter kinematic theory for shear walls. The method quantifies the damage state of the CLZ from geometrical and crack measurements, enabling rational on-site assessments.

KEYWORDS

crack-based assessment, reinforced concrete, shear mechanisms, shear walls, shear-critical

1 | INTRODUCTION

Reinforced concrete (RC) shear walls are common elements in the structures of buildings, towers, bridges (piers), and other concrete infrastructure. They typically resist the gravity loads in combination with lateral loads induced by wind or seismic events. Shear-critical RC walls have been of main concern in the structural field due to their complex behavior and their brittle nature, as they can fail suddenly without significant warnings. Following lateral load events, these members can exhibit diagonal shear cracks^{1,2} as illustrated in Figure 1, raising questions about their future structural safety.

Current design codes lack robust methods for assessment of cracked shear-critical RC walls. Existing methods and guidelines are typically qualitative and cannot

predict the remaining capacity of walls based on crack measurements. Even advanced numerical approaches, such as nonlinear finite element simulations, exhibit limitations in accurately accounting for existing cracks, as it is challenging to introduce discrete cracks in current smeared-crack formulations.

To understand how existing cracks transfer forces, Palipana et al.³ have introduced a comprehensive framework for deep beams by utilizing crack measurements derived from digital image correlation. Similarly, other studies have been conducted for slender beams confirming the reliability of such an approach in exploring shear critical members.^{4,5} A study by Calvi et al.⁶ has proposed methodologies for assessing panels subjected to pure shear and axial stresses. Lantsoght et al.⁷ have incorporated aggregate interlock relationships into their



FIGURE 1 Diagonal shear cracks of RC walls in actual buildings after earthquake events.¹

assessment of the shear behavior of cracked bridges. Most recently, Trandafir et al. have proposed a crack-based assessment approach for deep beams, where the geometry of existing cracks is used as an input in kinematics-based analysis for evaluating shear residual capacity.^{8,9} However, there remains a lack of studies and assessment approaches for cracked shear walls.

In this paper, a comprehensive framework is proposed for quantifying shear resisting mechanisms of short shear-critical walls from experimental measurements. The framework is informed by the three-parameter kinematic theory (3PKT).^{10,11} Two full-scale wall tests were adopted from a recent experimental program¹² to evaluate their shear resisting mechanisms. In these tests, three-dimensional full-view digital image correlation (DIC) was conducted to obtain detailed displacement and strain fields. By utilizing these deformations in combination with relevant constitutive relationships, the shear resisting mechanisms are quantified, and the results are compared to the predictions of the 3PKT approach.

In addition, based on the experimental results and analysis, hypotheses for the crack-based assessment of shear-critical walls are presented, using the 3PKT.^{10,11} Short shear-critical shear walls generally fail by crushing of the critical loading zone (CLZ) at the compressive toe of the wall. This zone is shown to contribute significantly to the total shear resistance of the wall. Therefore, at the end of this paper, a set of hypotheses for crack-based assessment (CBA) of the state of damage of the CLZ is introduced.

2 | SUMMARY OF PAST EXPERIMENTS

The analysis and the proposed framework will be applied to two full-scale shear wall tests—see Figure 2.¹² Test

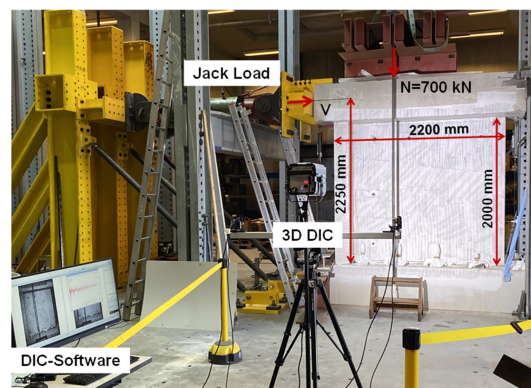


FIGURE 2 Dimensions and test setup of investigated shear walls.

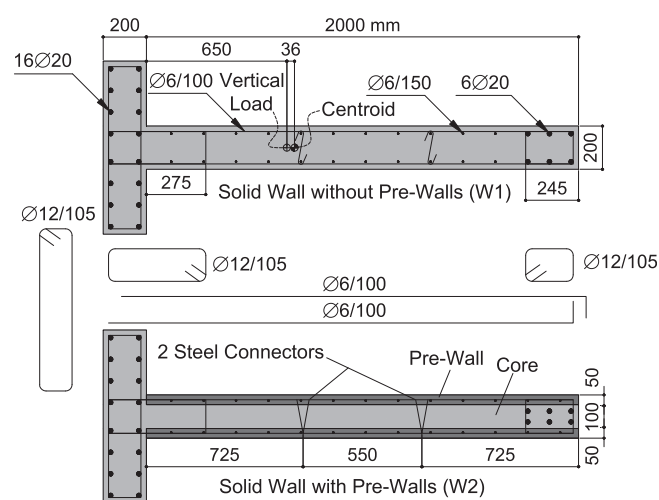


FIGURE 3 Cross-sectional dimensions and reinforcement of shear walls W1 and W2.

specimen W1 was a traditional cast-in-place wall, and specimen W2 was a wall with precast pre-walls (external pre-walls and cast-in-place core). The walls had a T-section with a tension flange as shown in Figure 3. The total length of the walls h was 2200 mm, the width b was 200 mm, the flange width b_f was 800 mm, the flange thickness t_f was 200 mm, the clear wall height a_{cl} was 2000 mm, and the height subjected to shear a was 2250 mm. The concrete properties of the two walls were almost identical, with compressive strengths f_c of 51.7 and 51.8 MPa (core: $f_c = 52.1$ MPa and average of pre-walls: $f_c = 51.35$ MPa), and modulus of elasticity E_c of 29,250 and 30,720 MPa, for W1 and W2, respectively. The maximum size of coarse aggregates a_g was 16 mm.

The details of the reinforcement of the two walls are shown in Figure 3. The web of the walls was reinforced with a steel mesh of $\text{Ø}6@150$ mm in the vertical direction, and $\text{Ø}6@100$ mm in the horizontal direction. The flange was reinforced with vertical reinforcement of $16\text{Ø}20$ and transverse reinforcement (hoops) of $\text{Ø}12@105$ mm. On

TABLE 1 Properties of steel reinforcement.

Bar diameter (mm)	20	6
Yield strength, f_y (MPa)	563	514
Ultimate strength, f_u (MPa)	645	626
Modulus of elasticity, E_s (GPa)	207.4	192.1
Yield strain, ε_y (‰)	2.71	2.67
Ultimate rupture strain, ε_u (‰)	60	93

the flexural-compression side of the section, the walls were reinforced with vertical reinforcement of 6 $\emptyset$ 20 concentrated along a horizontal length of 245 mm, and transverse reinforcement (hoops) of $\emptyset$ 12@105 mm. Table 1 summarizes the measured properties of the reinforcement used in the walls.

The two walls were tested monotonically under increasing lateral load V and a constant vertical load N of 700 kN as shown in Figure 2. The location of the vertical load almost coincided with the centroidal axis of the gross cross-section of the walls—see Figure 3. The lateral load was applied in 10 steps up to failure. After each loading step, the load was reduced by 10% and kept constant to conduct crack width measurements and take photos for reporting. The measuring instrumentation included 14 displacement transducers on one face of the walls, as well as a three-dimensional DIC system on the opposite face (Figure 2). The DIC system had two 24-megapixel cameras, which allowed for detailed measurements across the entire face.

The walls failed with the opening of a critical diagonal (shear) crack and crushing of the CLZ. The measured shear strength was 2599 and 2363 kN for wall W1 and W2, respectively. The difference in the strength of $\sim 9\%$ between the specimens was attributed to other possible factors such as the size of the compression zone and/or limited debonding between the cast-in-place core and the pre-walls in the CLZ of W2.¹²

3 | OVERVIEW OF THREE-PARAMETER KINEMATIC THEORY

As mentioned earlier, the analysis of the experimental data from tests W1 and W2 will be informed by the 3PKT for short shear-critical walls.^{10,11} Figure 4a shows an overview of the kinematic theory of short shear walls failing in shear: opening of a critical diagonal shear crack and crushing of the concrete at the wall's compression toe. The detailed kinematic modeling of the presented walls was provided elsewhere.¹²

The 3PKT includes information about global and local deformations of such walls, as well as about the shear-resisting mechanisms across the critical cracks.

The 3PKT is built on a kinematic model with three degrees of freedom (DOFs) as depicted in Figure 4a.^{10,11} The 3 DOFs define the position of the concrete block above the critical diagonal crack, while the region below the crack is described as a fan of cracks and struts. The first DOF $\varepsilon_{t,avg}$ corresponds to the elongation of the flexural-tension reinforcement of the wall (vertical tie). Associated with this DOF, the critical diagonal crack widens due to the rotation of the concrete block around the toe of the wall (point A). The second DOF Δ_c is the lateral displacement of the rigid block with respect to the fan due to the shear deformations within the CLZ. This DOF results in widening and slip displacements in the critical crack. The third DOF is the downward displacement Δ_{cx} occurring in the CLZ, which results in rotation of the rigid block about pivot B at the bottom of the vertical tie. This DOF is associated with widening and slip displacements in the critical crack.

The complete 3PKT method is based on three fundamentals: equilibrium conditions, constitutive relationships, and compatibility of deformations (kinematics).¹¹ According to the 3PKT, the total shear carried by the shear wall is the summation of four contributions: the shear carried by the CLZ, V_{CLZ} (function of DOFs Δ_c and Δ_{cx}), shear carried by the aggregate interlock, V_{ci} (function of Δ_c , Δ_{cx} , and $\varepsilon_{t,avg}$), shear carried by the web reinforcement, V_s (function of Δ_c , Δ_{cx} , and $\varepsilon_{t,avg}$), and shear carried by the dowel action of the flexural-tension reinforcement, V_d (function of Δ_c and $\varepsilon_{t,avg}$).

In the solution of the 3PKT, DOFs Δ_c and Δ_{cx} , as well as the corresponding strength of the CLZ, are computed in a first step, assuming that the concrete in the CLZ crushes at failure. The size (radius) of the CLZ, l_{CLZ} , is assumed to be $3l_{ble}\cos\alpha_1$, where l_{ble} is the characteristic length of the CLZ and α_1 is the angle of the critical diagonal crack (Figure 4b). The characteristic length l_{ble} defines the location of the critical section of the CLZ, which is perpendicular to the critical crack. By integration of compressive stresses along this section, the contribution of the CLZ, V_{CLZ} , is calculated. In a second step, DOF $\varepsilon_{t,avg}$ is determined by solving an equilibrium condition: the sum of the shear contributions ($V_{CLZ} + V_{ci} + V_s + V_d$) must equal the shear derived from the moment equilibrium of the wall. The moment equilibrium is taken about the compression toe in the base section and includes the tension force in the vertical tie (function of $\varepsilon_{t,avg}$).

4 | MEASURED DEFORMATIONS FROM DIGITAL IMAGE CORRELATION

The DIC system and software VIC-3D¹³ were used to obtain comprehensive in-plane and out-of-plane

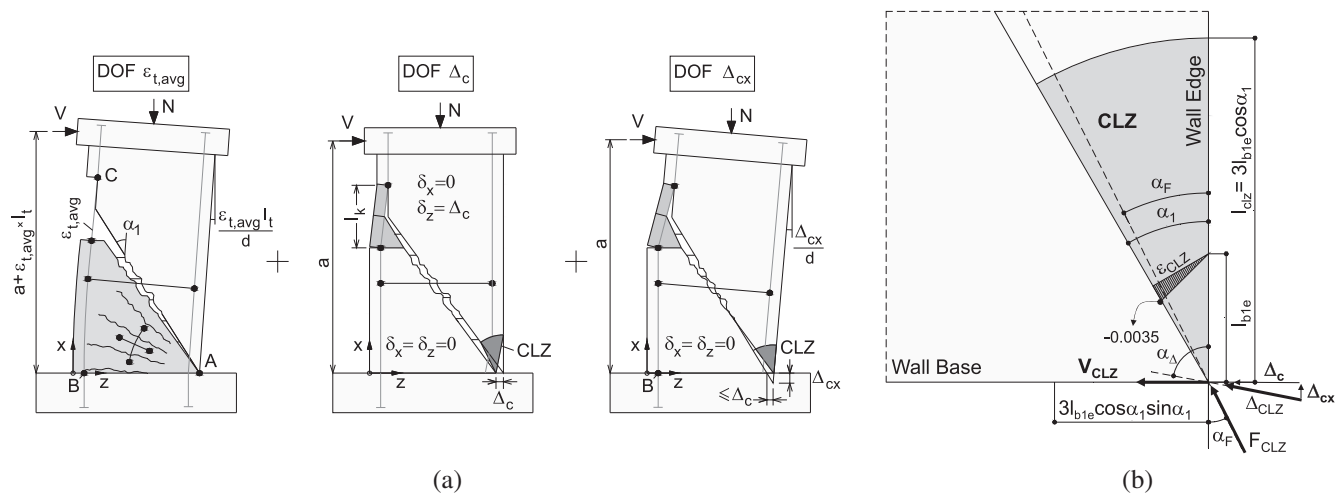


FIGURE 4 Basis of three-parameter kinematic theory (3PKT) for short shear walls: (a) three degrees of freedoms^{10,11} and (b) CLZ model.¹¹

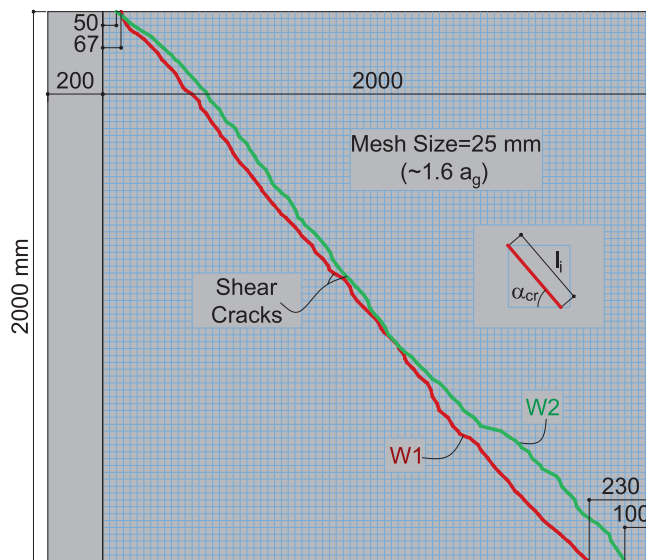


FIGURE 5 Discretization of critical shear cracks in walls W1 and W2.

displacement fields of specimens W1 and W2, as well as in-plane strain fields. The strain fields included the principal strains ε_1 (maximum) and strain ε_2 (minimum), and the principal strain angle θ_p . The post-processed DIC measurements were validated elsewhere by comparing the DIC results with conventional transducers readings.¹² Finally, these extensive measurements will be used to gain valuable insights into the deformations associated with each shear-resisting mechanism in short walls.

At first, the shape of the critical diagonal shear crack is discretized from photos on a square mesh of size 25 mm ($\sim 1.6a_g$), as shown in Figure 5. This discretization results in 128 and 140 segments for W1 and W2, respectively. Subsequently, based on this discretization, the horizontal w_h and vertical w_v crack displacements along the

critical diagonal crack at peak lateral load V_u are obtained, as illustrated in Figure 6. They were obtained via virtual extensometers (horizontal and vertical) drawn in VIC-3D. In this figure at the top, the crack displacements are represented as graphical bars perpendicular to the local crack angles along the critical shear cracks. Ultimately, these results of crack displacement are invaluable in assessing the shear-resisting mechanisms of aggregate interlock and web reinforcement.

As evident from the results, the maximum horizontal crack displacement $w_{h,max}$ was measured at approximately mid-depth of the walls, reaching values at failure of 1.7 and 1.5 mm for walls W1 and W2, respectively. On the other hand, the maximum vertical crack displacements $w_{v,max}$ were near the top end of the critical crack and reached values of 1.2 and 0.6 mm for walls W1 and W2, respectively. It is evident that the horizontal crack displacements are larger than the vertical ones, attributed to the low aspect ratio of the walls ($a/h \sim 1.0$) and the low axial load ($n = N/f_c b h \sim 3\%$).

In addition, the crack measurements reveal a noticeable reduction in displacements near both ends of the critical crack. The top end experiences reduced crack displacements, mainly due to the crack control provided by the longitudinal flexural reinforcement. At the bottom end of the crack, reductions in crack displacements start at approximately ~ 650 and ~ 370 mm from the wall edge for walls W1 and W2, respectively. These locations identify the size of the CLZ, which correspond to the points where the crack displacements started to be disturbed with a reducing trend associated with the compressive deformations within the CLZ. Also, it can be seen that the crack displacements diminish completely near the crack tip due to the boundary conditions imposed by the foundation.

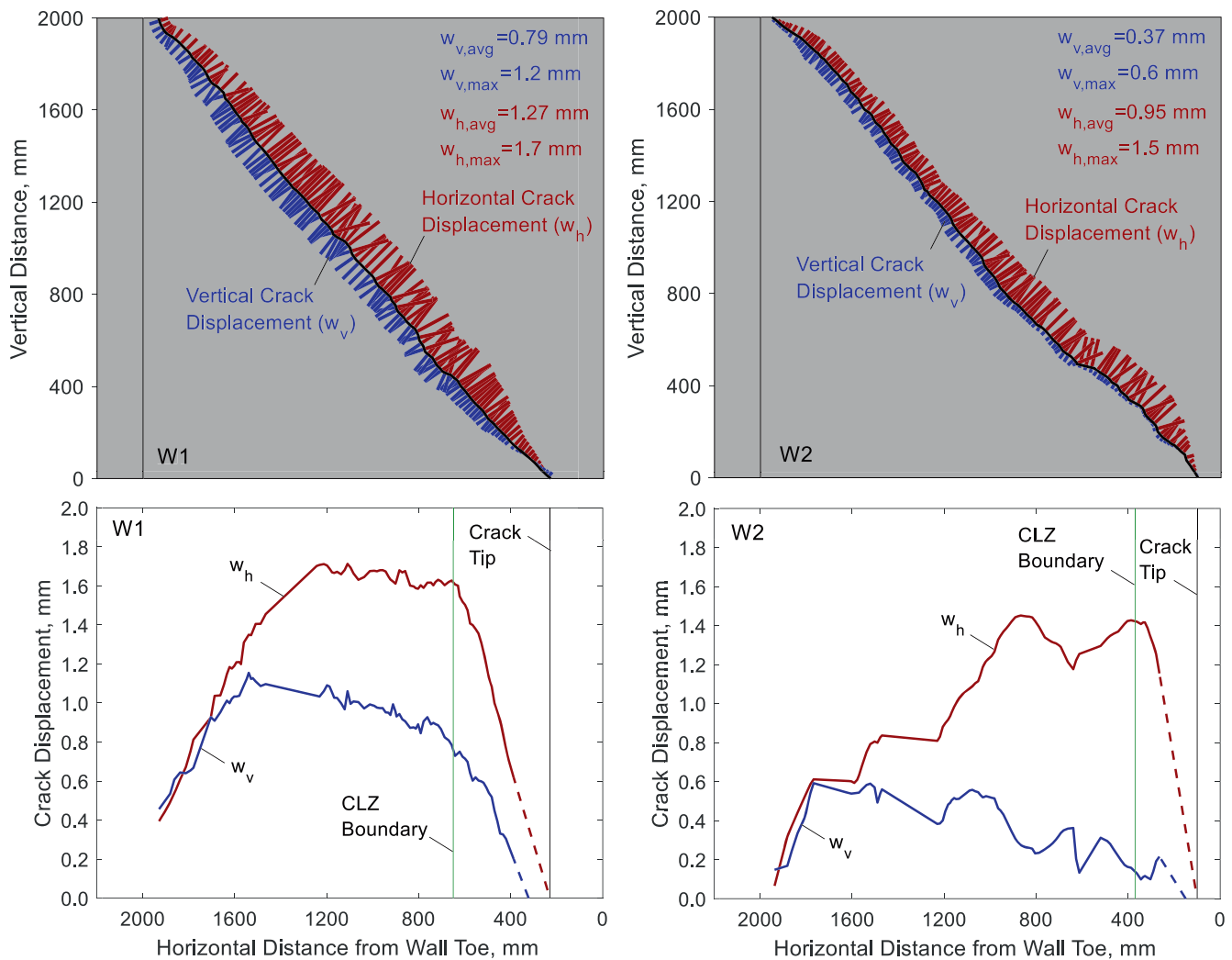


FIGURE 6 Measured crack displacements along the critical shear cracks of walls W1 and W2 at failure.

By comparing the horizontal w_h and vertical w_v crack displacements of both walls, W1 exhibited larger average values ($w_{h,avg} = 1.27$ mm vs. 0.95 mm, and $w_{v,avg} = 0.79$ mm vs. 0.37 mm), which is attributed to the random variation of the critical crack that can affect the size of the compression zone at the toe of the wall. This will be discussed later in some detail.

Secondly, to evaluate the shear-resisting contribution of the CLZ, it is necessary to define the size of this zone. The CLZ is conceptually enclosed by a circular sector with a center at the wall's toe, and a radius extending to the boundary point of the CLZ along the critical crack. In Figure 6, the boundary is marked by the vertical green lines. The estimated radius of the CLZ l_{clz} is 784 mm for wall W1 and 491 mm for wall W2. Subsequently, the characteristic length of the CLZ l_{ble} is calculated following the 3PKT¹¹ for direct comparisons as $l_{ble} = l_{clz} / (3\cos\alpha_{CLZ})$, where α_{CLZ} is taken as the average local crack angles of the critical shear crack within the CLZ. Based on this procedure, the critical sections of the CLZ of walls

W1 and W2 are drawn on the crack diagrams in Figure 7 (see green lines). Finally, the distributions of principal tensile strain ε_1 and compressive strain ε_2 in the CLZ are plotted in Figure 7 along these inclined sections.

Regarding the principal compressive strains ε_2 , the distribution exhibits nonlinear variations with maximum and minimum values of -0.76% and -3.1% for wall W1, and -0.57% and -3.3% for wall W2, near the crack and at the wall edge, respectively. The maximum compressive strains within the CLZs are consistent with the observed crushing of these zones at failure. On the other hand, the principal tensile strains ε_1 are primarily attributed to secondary shear cracks induced by transverse tension in the block above the critical crack. The transverse tension can reduce the compressive strength of concrete by means of compression softening; therefore, it was important to estimate the transverse tensile strains along the studied sections.

Finally, to assess the shear resistance provided by dowel action of the reinforcement in the tension flange,

FIGURE 7 Principal strains distribution along the studied sections within the CLZ of walls W1 and W2 at failure.

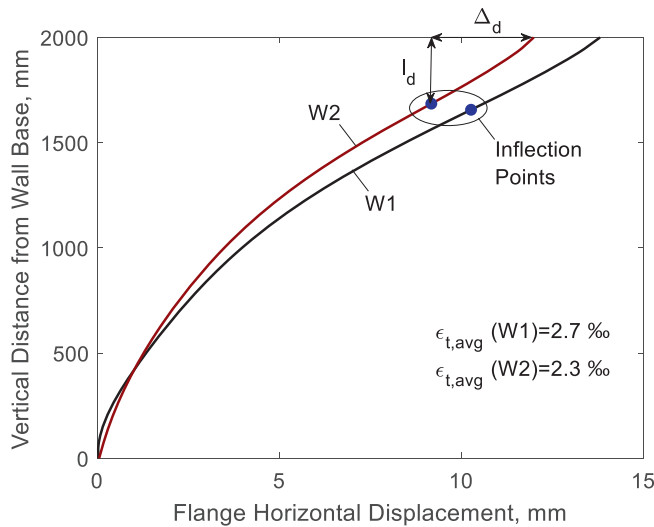
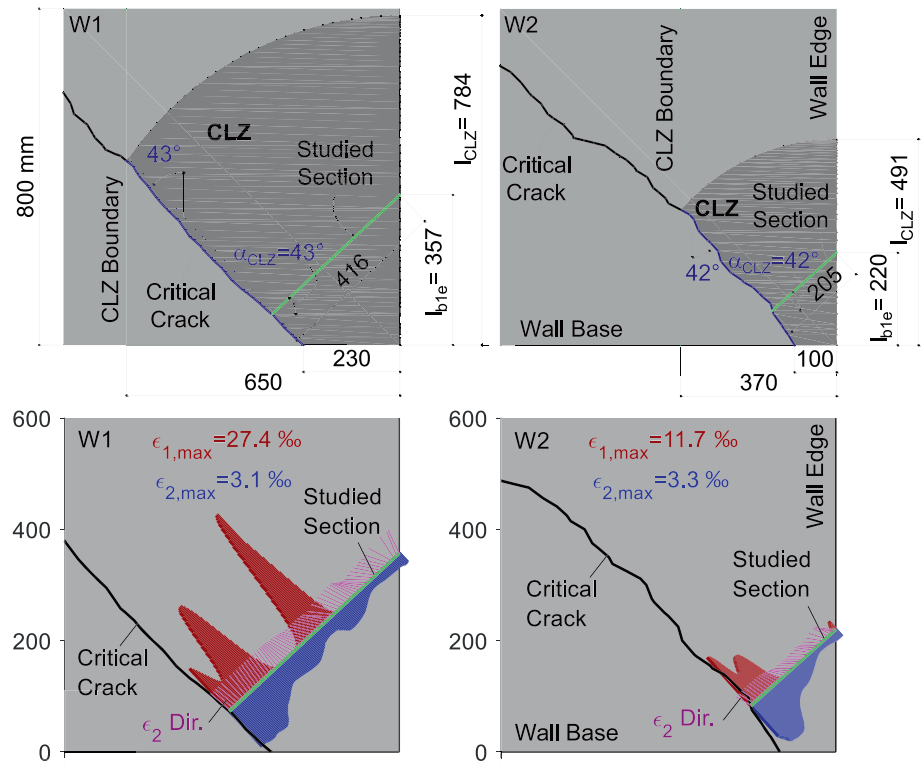


FIGURE 8 Horizontal displacements along the center line of the tension flange of walls W1 and W2 at failure.

the horizontal displacements along the flange centerline of the walls are obtained, as presented in Figure 8. An inflection point is noticeable along both deformed shapes. This inflection point can be mathematically calculated using the second derivative of the deformations. The calculated heights of the inflection point from the base of the walls are approximately 1656 and 1686 mm for walls W1 and W2, respectively. Next, the deformation of the bar-dowels Δ_d is estimated as the relative horizontal displacement between the calculated inflection

point and the top edge of the wall ($\Delta_d = 3.5$ mm for wall W1 and 2.8 mm for wall W2), while the dowel length l_d is the distance between the inflection point and the top edge of the wall ($l_d = 344$ mm for wall W1 and 314 mm for wall W2). Furthermore, the average tensile strains along the flange $\epsilon_{t,avg}$ were evaluated at 2.7‰ for W1 and 2.3‰ for W2.¹² These strains will be taken into account as the tension on in the bar-dowels reduces the dowel resistance.

5 | QUANTIFYING SHEAR RESISTING MECHANISMS

5.1 | Constitutive relationships

5.1.1 | Critical loading zone

The CLZ resists shear mainly by means of inclined compressive stresses. Therefore, the shear force V_{CLZ} can be evaluated by integrating the stresses along the defined critical sections within the CLZ. In the previous section, the principal strains ϵ_1 and ϵ_2 along these sections were obtained from DIC measurements. These strains can be transferred to stresses by using appropriate constitutive relationships for concrete with input values of f_c and E_c .

The tensile stresses σ_1 in the concrete can be obtained by a linear-elastic relationship ($\sigma_1 = E_c \epsilon_1$) up to the cracking stress ($f_{ct} = 0.33 f_c^{0.5}$), and zero for larger strains. On the other hand, for the compressive stresses σ_2 , the

behavior is more complex due to the effect of transverse tension on the compressive strength; thus, an effective concrete strength $f_{c,eff}$ (MPa) needs to be calculated.

In the case of uncracked concrete ($\varepsilon_1 \leq f_{ct}/E_c$), a simplified Kupfer failure criterion¹⁴ is adopted in Equation (1), as proposed for plain concrete to account for the interaction between the tensile and compressive stresses in the concrete. In the case of cracked concrete ($\varepsilon_1 > f_{ct}/E_c$), the compression-softening model by Vecchio and Collins¹⁵ is adopted in Equation (1), as developed based on shear tests of RC panels with multiple transverse cracks.

Subsequently, the principal compressive stresses σ_2 can be calculated as in Equation (2) from stress-strain a relationship of concrete in compression⁵ by using the effective concrete strength. Finally, in order to obtain the normal σ_n and shear stresses τ with respect to the studied section inclined at angle α_{CLZ} , Mohr's circle is applied (see Equation (3)) assuming that the angle of the principal strains θ_p obtained from the DIC coincides with the angle of the principal stresses.

$$f_{c,eff} = \begin{cases} f_c - 4\sigma_1 & \text{if } \varepsilon_1 \leq f_{ct}/E_c \text{ [14]} \\ \frac{f_c}{(0.8 + 170\varepsilon_1)} & \text{if } \varepsilon_1 > f_{ct}/E_c \text{ [15]} \end{cases} \quad (1)$$

$$\sigma_2 = \frac{E_c \varepsilon_2}{1 + \left(\frac{\varepsilon_2}{\varepsilon_0}\right)^\alpha}, \quad \alpha = 0.5 + \frac{f_{c,eff}}{20} + \frac{f_{c,eff}^2}{1500} \quad (2)$$

$$\varepsilon_0 = \frac{\alpha f_{c,eff}}{E_c (\alpha - 1)^{(1 - (1/\alpha))}}$$

$$\begin{aligned} \sigma_n &= \sigma_2 + 0.5(\sigma_1 - \sigma_2) \\ &\quad - 0.5[(\sigma_1 - \sigma_2) \cos(2|\alpha_{CLZ} - \theta_p|)] \\ \tau &= 0.5[(\sigma_1 - \sigma_2) \sin(2|\alpha_{CLZ} - \theta_p|)] \end{aligned} \quad (3)$$

5.1.2 | Aggregate interlock

The resistance provided by aggregate interlock V_{ci} can be estimated by adopting the contact density model (CDM) proposed by Li et al.¹⁶ The CDM allows to calculate the tangential stresses ν_{ci} and normal compressive stresses n_{ci} on each crack segment as a function of the crack width w and crack slip s . A modified version of the CDM is used in this study as proposed elsewhere⁸ based on shear panels tests conducted by Calvi et al.¹⁷—Equation (4). The modification consists of a reduction factor of 0.35 applied on the yield strength of the contact springs to

account for large crack displacements as those exhibited by walls W1 and W2.

$$\begin{aligned} \nu_{ci} &= \int_{-\pi/2}^{\pi/2} A_t \sigma_{con}(\theta) K(w_\theta) \Omega(\theta) \sin(\theta) d\theta \\ n_{ci} &= \int_{-\pi/2}^{\pi/2} A_t \sigma_{con}(\theta) K(w_\theta) \Omega(\theta) \cos(\theta) d\theta \\ \Omega(\theta) &= 0.5 \cos(\theta), \quad A_t = 1.27 \\ w_\theta &= s \sin(\theta) - w \cos(\theta) \\ \sigma_{con}(\theta) &= 0.35 \times \min \left[13.7 f_c^{1/3} \frac{w_\theta}{0.04}, 13.7 f_c^{1/3} \right] \\ K(w_\theta) &= 1 - \exp \left(1 - \frac{0.5 a_g}{w} \right) \geq 0 \end{aligned} \quad (4)$$

5.1.3 | Web reinforcement

The shear resistance provided by the web reinforcement V_s is due to the tensile forces in the reinforcement induced by the opening of the critical diagonal crack. In the previous section, the horizontal crack displacements w_h along the critical shear crack were obtained by the DIC. Next, by utilizing a bond-slip relationships proposed by Sigrist,¹⁸ the stresses in the web reinforcement can be obtained as a function of w_h . In this bond model, the bar is considered anchored on each side of the crack and the anchorage length is divided into two segments: l_1 near the crack surface where the reinforcement has yielded, and l_2 right after where the reinforcement is elastic. The bond stress is assumed constant in each segment along the bar with values f_{ct} within l_1 and $2f_{ct}$ within l_2 ($f_{ct} = 0.33 f_c^{0.5}$, where f_{ct} is the tensile strength of the concrete). The stress-strain response of the reinforcement is idealized as bi-linear (elastic-plastic with strain hardening). Through bar equilibrium, the relation between bar stress f_v and crack displacement is derived.

5.1.4 | Dowel action

As discussed earlier, the shear resistance V_d provided by dowel action is due to the transverse displacement Δ_d of the bars induced by the shear crack. This contribution can be calculated from Equation (5) based on a simple model of a fixed-fixed beam that behaves in a symmetrical double curvature. The strength of the dowels is limited by the formation of plastic hinges at their ends. Here, an effective strength $f_{y,eff}$ is calculated, which depends on the yield strength of the bars f_y and the ratio of the actual tensile strain $\varepsilon_{t,avg}$ to the yield strain ε_y of the bars:

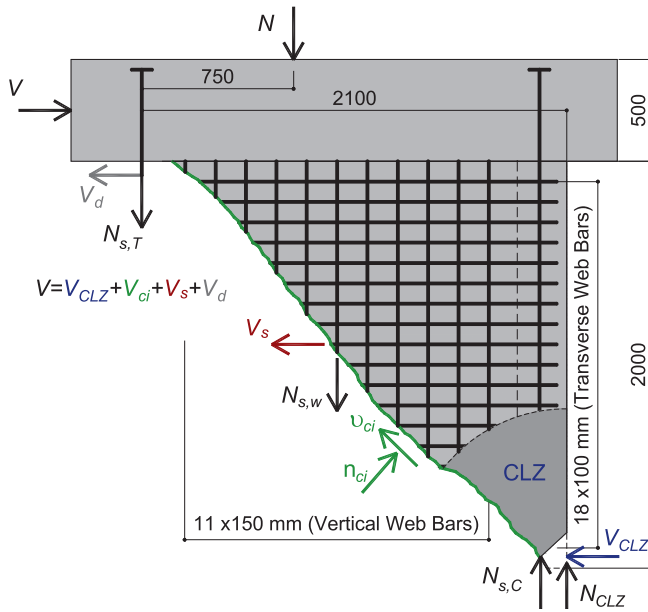


FIGURE 9 Free body diagram to evaluate the shear resisting components.

$$V_d = n_b \frac{12E_s \pi d_b^4}{64l_k^3} \Delta_d \leq n_b f_{y,eff} \frac{d_b^3}{3l_k}, \quad (5)$$

$$f_{y,eff} = f_y \left[1 - \left(\frac{\epsilon_{t,avg}}{\epsilon_y} \right)^2 \right] \geq 0$$

where n_b is the number of bars of flexural longitudinal reinforcement (18 bars in W1 and W2), E_s is the modulus of elasticity of the bars, d_b is the diameter of the bars, f_y is the yield strength of the bars, $f_{y,eff}$ is the effective yield strength of the bars, and l_d is the dowel's length.

5.2 | Evaluated shear resisting mechanisms

Figure 9 shows the free body diagram of the walls used to evaluate the shear resisting mechanisms. The free body is bound by the critical shear crack and the critical section within the CLZ. The total shear carried by the walls is the summation of the four shear-resisting components: compression in the CLZ V_{CLZ} , tension in the web reinforcement V_s , shear provided by aggregate interlock across the shear crack V_{ci} , and dowel action provided by the longitudinal flexural reinforcement V_d .

5.2.1 | Critical loading zone

From the obtained principal strains in Figure 7, and by implementing the constitutive relationships of concrete in compression discussed earlier, the normal stresses σ_n

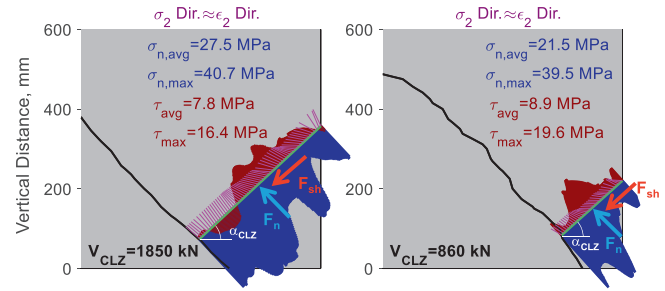


FIGURE 10 Evaluated normal stresses σ_n and shear stresses τ along the studied section within the CLZ of walls W1 and W2 at failure.

and shear stresses τ along the studied section within the CLZ are calculated as shown in Figure 10 (Equations (1)–(3)).

The normal stresses σ_n are compressive, with and average value of 27.5 MPa for W1 and 21.5 MPa for wall W2. The maximum compressive stress for both walls is ~ 40 MPa, which is 77% of the uniaxial compressive strength of the concrete. The concrete crushes at lower stresses due the compression softening induced by the secondary shear cracks within the CLZ. Note also that σ_n are not the principal compressive stresses (the principal directions are marked in Figure 10).

Stresses σ_n and τ are integrated along the studied sections to obtain the normal force F_n and tangential force F_{sh} . Finally, from the horizontal projection of these forces, the shear carried by the CLZ V_{CLZ} is evaluated as follows:

$$V_{CLZ} = F_n \sin \alpha_{CLZ} + F_{sh} \cos \alpha_{CLZ} \quad (6)$$

The values of V_{CLZ} are 1850 and 860 kN for walls W1 and W2, respectively. Therefore, V_{CLZ} of wall W1 is ~ 2.2 times larger than that of W2, which is mainly due to the size of the CLZ: l_{clz} of wall W1 is $\sim 60\%$ larger. Moreover, from the magnitude of forces along the studied sections, the orientation of the resultant force of the CLZ with respect to the vertical axis is obtained: $\alpha_F \sim 54^\circ$ for W1 and $\sim 65^\circ$ for W2.

5.2.2 | Aggregate interlock

At first, based on the discretization conducted in Figure 5, the local angles α_{cr} with respect to the horizontal axis and the length l_i of the crack segments are obtained. In a second step, the measured local angles and crack displacements in Figure 6 are used to calculate the crack width w_i and slip s_i on each crack segment in Equation (7). They are calculated by projecting the corresponding vertical w_{vi} and horizontal w_{hi} crack

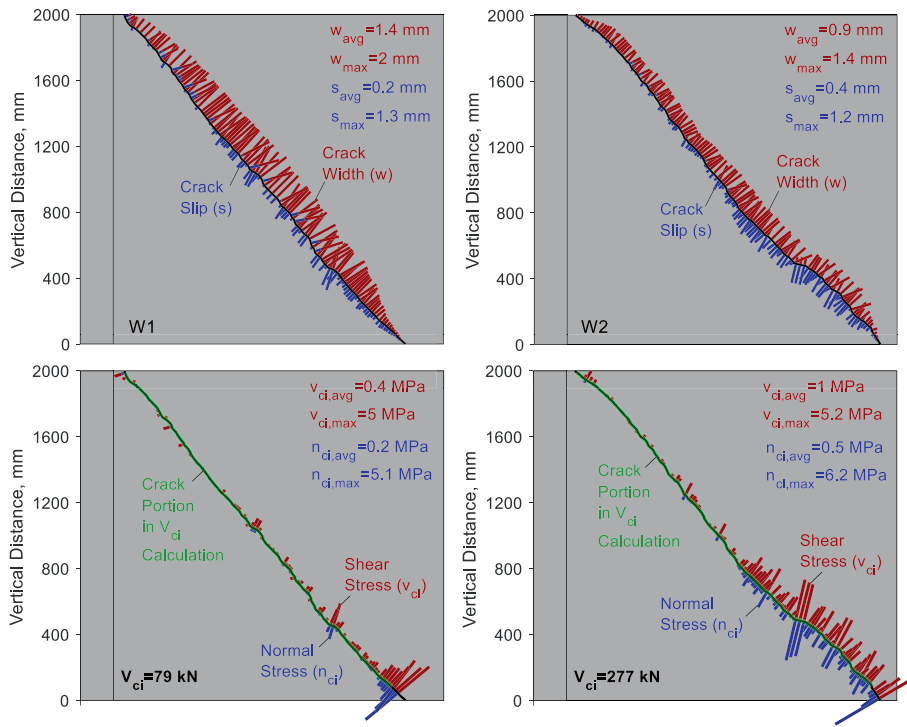


FIGURE 11 Measured crack displacements (width, slip) and corresponding shear and normal stresses along the critical shear cracks of walls W1 and W2 at failure.

displacements in the perpendicular and parallel directions to each crack segment.

$$\begin{aligned} w_i &= w_{hi} \sin \alpha_{cri} + w_{vi} \cos \alpha_{cri} \\ s_i &= w_{hi} \cos \alpha_{cri} - w_{vi} \sin \alpha_{cri} \end{aligned} \quad (7)$$

The variation of crack widths w_i and slips s_i along the critical shear crack is shown in Figure 11. As evident from the figure, wall W1 shows larger crack widths compared to wall W2 ($w_{avg} = 1.4$ mm vs. 0.9 mm, $w_{max} = 2.0$ mm vs. 1.4 mm). On the other hand, wall W2 had larger crack slips than W1 ($s_{avg} = 0.4$ mm vs. 0.2 mm), mostly concentrated along the bottom part of the critical crack within a vertical distance of ~ 800 mm from the base.

The distribution of the local crack angles of the two walls is illustrated in the histogram in Figure 12. It is seen that the distributions exhibit similar trends with the majority of the crack segments lying within the range of $45^\circ < \alpha_{cr} < 55^\circ$. Also, wall W2 has a larger number of segments with smaller crack angles (flatter cracks), which explains the larger slips in W2. The average crack angles are 49° and 47° for W1 and W2, respectively.

Using the crack widths and slips, the aggregate interlock stresses are evaluated via the CDM. Figure 11 shows the distribution of these stresses along the critical diagonal cracks of the two tested walls. It is clear that the magnitude of the shear stresses v_{ci} for W2 are larger than

those for W1 ($v_{ci,avg} = 1.0$ MPa vs. 0.40 MPa, $v_{ci,max} = 5.2$ MPa vs. 5.0 MPa). Similarly for the normal stresses, W2 has larger values compared to W1 ($n_{ci,avg} = 0.50$ MPa vs. 0.20 MPa, $n_{ci,max} = 6.2$ MPa vs. 5.1 MPa). These larger stresses can be explained by the combination of smaller crack widths and larger crack slips in W2 compared to W1.

Finally, the shear carried by aggregate interlock is calculated from Equation (8) by integrating the shear and normal stresses along the shear crack.

$$V_{ci} = \sum_{i=1}^{i=n} (v_{ci,i} \cos \alpha_{cr,i} - n_{ci,i} \sin \alpha_{cr,i}) l_i b \quad (8)$$

where n is the number of crack segments, l_i is the crack segment length, and b is the wall's width.

The values of V_{ci} are 79 and 277 kN for W1 and W2, respectively. Even though the shape of the critical cracks of the walls is similar (Figure 5), the shear carried by the aggregate interlock in W2 is ~ 3.5 times larger than that of W1. This is due to the higher shear stresses in combination with a larger number of segments with flatter angles along the shear crack of W2.

5.2.3 | Web reinforcement

As discussed earlier, the shear carried by the web reinforcement V_s is calculated based on the measured

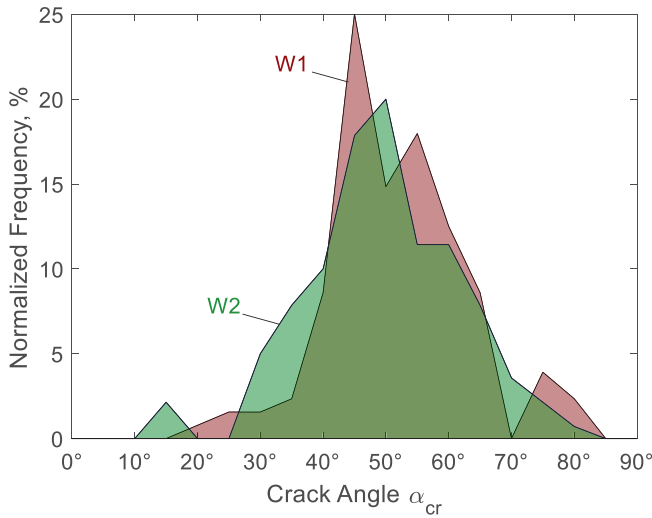


FIGURE 12 Histogram of local crack angles along the critical shear cracks of walls W1 and W2.

crack displacements in combination with a bond model.¹⁸ More specifically, at each level of the web reinforcement along the critical shear crack, the steel stresses f_{vi} are evaluated as a function of the horizontal crack opening w_{hi} .

Figure 13 shows the distribution of steel stresses along the height of the wall. The stresses are nearly uniform in both walls, where almost all the web reinforcement yielded in the critical crack and entered the strain-hardening regime of the steel ($f_{vi} > f_{yv}$). The shear resistance V_s is calculated as the summation of the contributions of each level of the web reinforcement V_{si} (dots in the figure) along the height of the wall:

$$V_s = \sum_{i=1}^{i=n_s} A_{vi} f_{vi}(w_{hi}) \quad (9)$$

where n_s is the number of levels of web reinforcement along the wall's height (19 levels), and A_{vi} is the area of the web reinforcement at each level.

From these calculations, the shear resistance provided by the web reinforcement in walls W1 and W2 is approximately the same: $V_s \sim 588$ kN.

5.2.4 | Dowel action

Finally, the shear carried by dowel action is calculated from Equation (5) by substituting the values of dowel lengths l_d , dowel transverse deformations Δ_d , and average strain of the flexural reinforcement $\epsilon_{t,avg}$ estimated earlier in Section 4. The resulting values are negligible: $V_d \sim 1$ and 24 kN for W1 and W2, respectively. This is due to the

large tensile strains in the bars at peak load, which limit the moment capacity of the plastic hinges in the bar-dowels.

5.2.5 | Summary of shear components; horizontal equilibrium

Table 2 shows a summary of the estimated shear-resisting components and their sum (total shear) V_{est} for the tested shear walls. The estimated shear is compared to the externally applied shear at failure V_{exp} . It can be seen that the agreement for wall W1 is excellent with $V_{exp}/V_{est} = 1.03$, while the shear strength of wall W2 is underestimated with $V_{exp}/V_{est} = 1.35$. Wall W2 is a hybrid wall, as it consists of three concrete components within its cross-section. Consequently, the shear crack may not have been uniform across the width of the cross-section, potentially due to limited debonding between the core and the pre-walls. Since the estimation of total shear for W2 was based solely on deformations measured on the surface of one of the pre-walls, this can explain the relatively larger discrepancy in the strength estimation.

As evident from Table 2, the predominant portion of shear in W1 is carried by the CLZ with $V_{CLZ}/V_{est} = 73\%$, followed by the web reinforcement, which contributes 24%. Conversely, the contributions of aggregate interlock and dowel action are negligible. In the case of W2, similar to W1, the primary contributor is the CLZ, accounting for 49% of the shear resistance, followed by the web reinforcement with 34%. In this wall, the aggregate interlock has a non-negligible contribution of 16%. Similarly to W1, the contribution of dowel action in W2 is negligible.

The key distinction between these two walls lies in the size of the CLZ, which was measured from the crack diagrams on the side of the DIC system. This difference may stem from random variations in the path of the critical shear crack, or be caused in part by local debonding between pre-walls and core concrete. Based on the measurements, it was determined that the size of the CLZ of W1 was approximately $\sim 60\%$ larger than that of W2 ($l_{clz} = 784$ mm vs. 491 mm). When the CLZ is larger, the associated shear deformations at failure increase, consequently leading to larger crack widths. Therefore, as V_{CLZ} increases, the contribution of aggregate interlock decreases as observed in W1.

5.3 | Vertical equilibrium

In addition to horizontal equilibrium, it is also important to confirm the vertical equilibrium of internal and external forces, taking into account the applied axial load of 700 kN in compression. The equilibrium is again studied

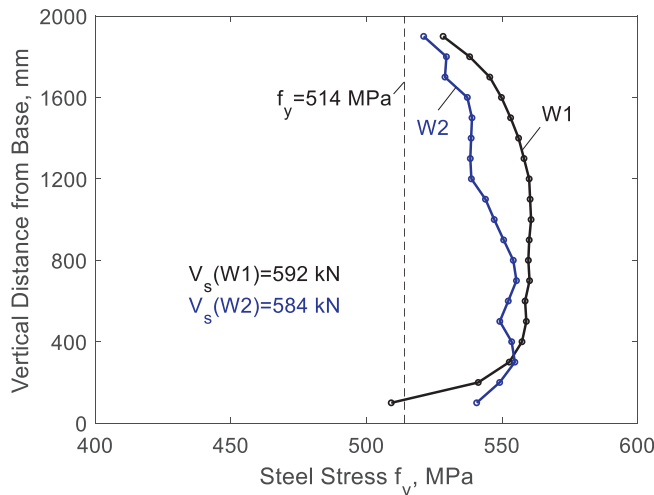


FIGURE 13 Estimated steel stresses of web reinforcement along the height of walls W1 and W2 at failure.

for the free body in Figure 9. Internal vertical forces develop in the CLZ, N_{CLZ} , in the vertical web reinforcement, $N_{s,w}$, in the tensile flexural reinforcement in the flange, $N_{s,T}$, in the compressive flexural reinforcement, $N_{s,C}$, and via aggregate interlock, N_{ci} .

Forces N_{CLZ} and N_{ci} are directly calculated as per Sections 5.2.1 and 5.2.2 by projecting the evaluated internal forces on the vertical axis. The contributions of the vertical reinforcement $N_{s,w}$ and $N_{s,T}$ are calculated from the measured vertical crack displacements w_v in combination with the bond-slip relationship.¹⁸ The vertical crack displacements associated with the vertical web reinforcement were obtained from Figure 6. On the other hand, the vertical crack displacement associated with the flexural-tension reinforcement are measured in the outermost top flexural crack in the flange at the peak load ($w = 0.18$ mm for W1 and $w = 0.10$ mm for W2). Finally, the contribution of the compressive flexural reinforcement $N_{s,C}$ is calculated from the measured vertical strains ϵ_{yy} at the centroid of the bars ($\epsilon_{yy} = -1.1\%$ for W1, $\epsilon_{yy} = -1.5\%$ for W2).

The results of these calculations are summarized in Table 3. The compressive internal forces are shown with negative signs, while the sum of all internal forces is denoted as N_{est} . It can be seen that the estimated axial forces are in a good agreement with the applied axial compression N_{exp} : experimental-to-estimated ratios of 1.08 and 1.28 for W1 and W2, respectively. According to the results of the horizontal and vertical equilibrium, the resisting and external forces acting on the free body in Figure 9 are unbalanced; therefore, it is not informative to check the moment equilibrium since the moments will vary based on the point of rotation under consideration.

TABLE 2 Summary of the evaluated shear components from DIC measurements performed on wall W1 and W2 at failure.

Wall	Shear force, V (kN)						V_{exp}/V_{est}
	V_{CLZ}	V_{ci}	V_s	V_d	V_{est}	V_{exp}	
W1	1850	79	592	1	2522	2599	1.03
W2	860	277	584	24	1745	2363	1.35

6 | COMPARISON WITH 3PKT PREDICTIONS

The 3PKT model for ultimate response¹¹ is used to predict the shear resisting components of the tested walls. As the compressive strength of the concrete varied only slightly, an average value $f_c = 51.75$ MPa is used for the simulations of the two walls. The predicted shear contributions are compared to the contributions evaluated in the previous section. The complete 3PKT calculations for the presented walls are available elsewhere.¹²

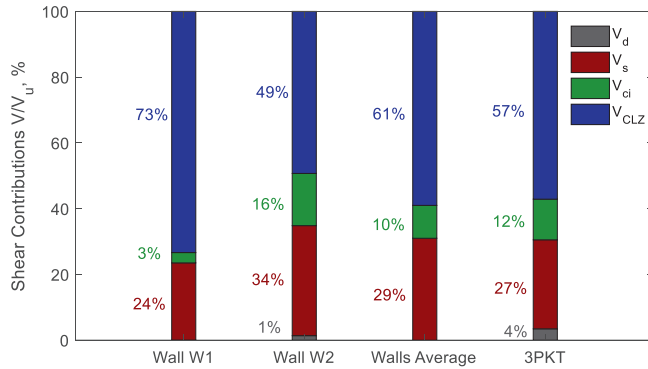
According to the 3PKT,¹¹ the predicted shear strength of the walls is 2181 kN, which is in a good agreement with the experimental values. The average experimental-to-predicted ratio of the two walls is 1.13. The agreement is also adequate in terms of shear-resisting contributions as presented in Figure 14. On average, the predicted relative contributions match well with those estimated from the DIC measurements: 57% versus 61% for V_{CLZ} , 12% versus 10%, 27% versus 29% for V_s , and 4% versus 0 for V_d . These results show that the 3PKT approach has a solid physical basis, and add to its extensive validation reported elsewhere.¹¹

7 | HYPOTHESES TOWARDS A CRACK-BASED ASSESSMENT METHOD

As illustrated above, the CLZ has a significant contribution, accounting for $\sim 60\%$ of the total shear strength of the studied short walls. The failure of such walls is primarily triggered by the crushing of the CLZ. Furthermore, it was shown that variations in the shape of the critical crack can lead to significant differences in the resistance of the CLZ, with the resistance varying by a factor of ~ 2 among the studied walls. Therefore, this section presents a set of hypotheses towards a CBA framework for short shear-critical walls based on the CLZ. The CBA uses solely on-site measurements of geometry and crack displacements performed in the vicinity of the CLZ. It should be highlighted that the CBA applies for squat shear walls that exhibited fully propagated diagonal shear crack and it is limited to walls with an aspect ratio a/h less than 3.0.

TABLE 3 Estimated vertical resisting forces of walls W1 and W2 at failure.

Wall	Vertical force, N (kN)							N_{exp}/N_{est}
	N_{CLZ}	N_{ci}	$N_{s,w}$	$N_{s,T}$	$N_{s,c}$	N_{est}	N_{exp}	
W1	−1362	−127	324	945	−430	−650	−700	1.08
W2	−408	−542	285	706	−586	−546	−700	1.28

**FIGURE 14** Estimated versus predicted (3PKT) contributions of the shear resisting mechanisms of walls W1 and W2 at failure.

The main hypothesis of the CBA is rooted in evaluating the extent of damage in the CLZ. This evaluation involves comparing the resultant displacement in the CLZ (demand) to its ultimate displacement (capacity). Ultimately, these required displacements will be linked to the displacements in the shear cracks within the CLZ, on the basis of the kinematics of diagonally cracked shear walls.

Initially, it is imperative to establish the geometry of the CLZ. As detailed DIC measurements are not available in site inspections, the size of the CLZ, l_{CLZ} , cannot be measured directly as in Section 4. Instead, it will be derived from two measurable parameters: the depth of the CLZ d_{CLZ} and the angle of the CLZ α_{CLZ} with respect to the vertical axis—see Figure 15. These parameters have been introduced in a CBA method for deep beams.⁸ Parameter d_{CLZ} represents the perpendicular distance to the critical shear crack, measured between the crack tip and the wall edge. On the other hand, the angle of the CLZ α_{CLZ} is measured on a line extending from the crack tip to a distance of $3d_{CLZ}$. The end of this line lies on the critical crack (see point C) and represents the boundary of the CLZ. With measured d_{CLZ} and α_{CLZ} , the size (radius) l_{CLZ} and characteristic length l_{b1e} of the CLZ is directly calculated with Equations (10)–(14) based on the 3PKT and the geometry shown in Figure 15:

$$x_0 \approx d_{CLZ} \cos \alpha_{CLZ} \text{ or directly measured} \quad (10)$$

$$x_c = x_0 + 3d_{CLZ} \sin \alpha_{CLZ} \quad (11)$$

$$y_c = 3d_{CLZ} \cos \alpha_{CLZ} \quad (12)$$

$$l_{CLZ} = \sqrt{x_c^2 + y_c^2} \quad (13)$$

$$l_{b1e} = \frac{l_{CLZ}}{3 \cos \alpha_{CLZ}} \quad (14)$$

The characteristic length l_{b1e} is used to calculate the displacement capacity of the CLZ $\Delta_{CLZ,u}$ according to the 3PKT—see Equation (15). The ultimate compressive strain $\epsilon_{CLZ,u}$ in Equation (15) is estimated at 3‰, which is consistent with the DIC results in Figure 7. The equation also contains the angles of the compression force α_F and resultant displacement α_Δ in the CLZ (see Figure 4). The angle of the compression force can be reasonably approximated as $\alpha_F \approx \alpha_{CLZ}$, assuming that the compression flows parallel to the critical crack. However, determining angle α_Δ is more challenging, as it requires to estimate the ratio between DOFs Δ_c and Δ_{cx} of the 3PKT. The values of Δ_c and Δ_{cx} are also needed to directly estimate the displacement demand in the CLZ given by Equation (16).

$$\Delta_{CLZ,u} = \epsilon_{CLZ,u} \frac{3l_{b1e} \cos \alpha_{CLZ}}{\cos(\alpha_\Delta - \alpha_F)} \text{ (capacity)} \quad (15)$$

$$\Delta_{CLZ} = \sqrt{\Delta_c^2 + \Delta_{cx}^2} \text{ (demand)} \quad (16)$$

To estimate Δ_c and Δ_{cx} , it is proposed to measure horizontal and vertical crack displacements in the vicinity of the CLZ. More specifically, as shown in Figure 15, the horizontal crack displacements $w_{h,i}$ are measured along a horizontal line passing through point C, and the vertical displacements $w_{v,i}$ are measured along a vertical line through the same point. The sums of these displacements are used to estimate DOFs Δ_c and Δ_{cx} as follows:

$$\Delta_c \approx \sum_i w_{h,i} \quad (17)$$

$$\Delta_{cx} \approx \frac{\sum_i w_{v,i} x_0}{x_c - x_0} \quad (18)$$

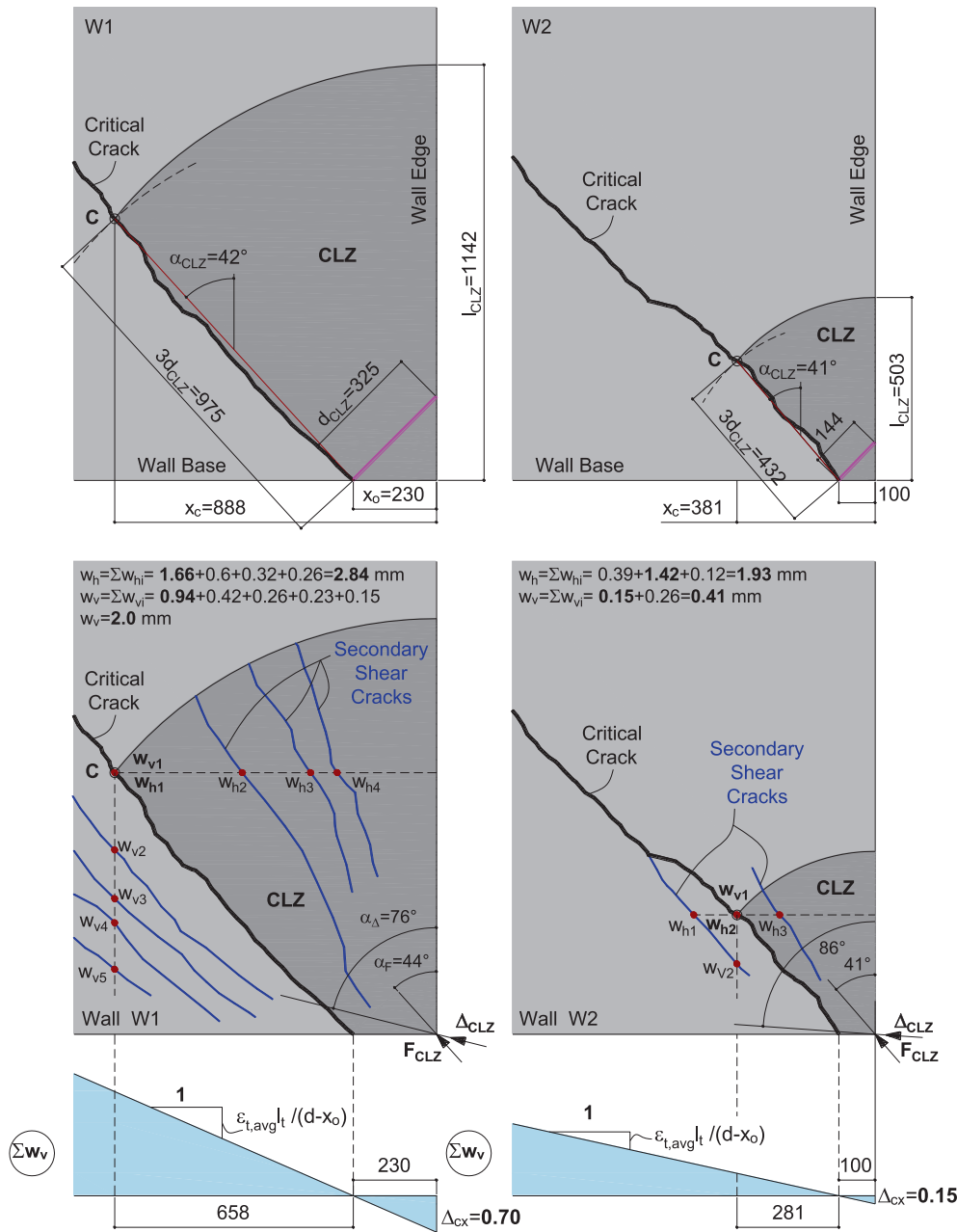


FIGURE 15 CBA of the critical loading zone of short shear-critical shear walls.

Equation (17) stems directly from the definition of DOF Δ_c illustrated in Figure 4. It assumes that the contribution of DOFs $\varepsilon_{t,avg}$ and Δ_{cx} to $w_{h,i}$ is negligible near the base of the wall. Equation (18), on the other hand, is derived by linking DOFs $\varepsilon_{t,avg}$ and Δ_{cx} , both of which contribute to the vertical crack displacements $w_{v,i}$ (Figure 4). It is assumed that as a result of $\varepsilon_{t,avg}$ and Δ_{cx} , the concrete block above the critical diagonal crack rotates about the tip of the crack in the base section. This rotation is expressed with the fraction in Equation (18). When the rotation is multiplied by x_o , it produces the vertical downwards displacement along the edge of the wall, which

corresponds to DOF Δ_{cx} . This is illustrated with the linear displacement variation in Figure 15.

With estimated DOFs Δ_c and Δ_{cx} , angle α_Δ in Equation (15) is obtained from:

$$\alpha_\Delta = \tan^{-1}(\Delta_c / \Delta_{cx}), \text{ where } \alpha_{CLZ} \leq \alpha_\Delta \leq 90^\circ \quad (19)$$

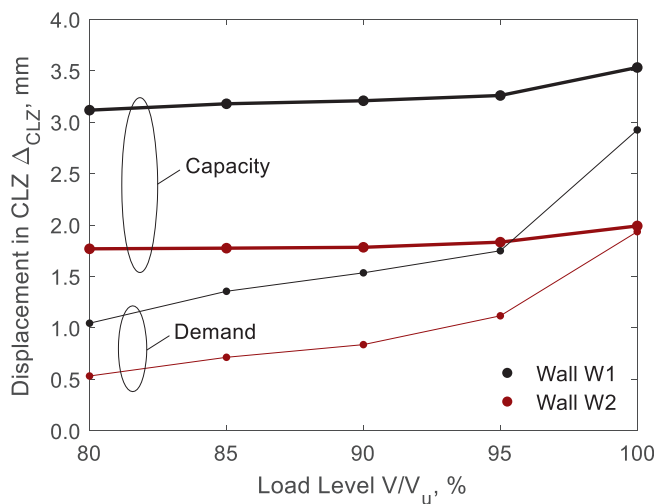
The described CBA was applied to walls W1 and W2 at failure as shown in Figure 15 and summarized in Table 4. The geometry of the CLZ was measured on both faces of the wall, and the average value of l_{CLZ} was used to calculate the characteristic length l_{b1e} (Figure 15 shows one face only). It can be seen that

TABLE 4 Results of CBA applied to W1 and W2 at failure.

	Wall W1	Wall W2
d_{CLZ} (mm)	325/228	144/125
α_{CLZ} and α_F (°)	42/47 ^a	41/43 ^a
l_{CLZ} (mm)	1142/805 ^a	503/441 ^a
l_{ble} (mm)	437	208
Δ_{cx} (mm)	0.70	0.15
Δ_c (mm)	2.84	1.93
α_Δ (mm)	76	86
$\Delta_{CLZ,u}$ (capacity) (mm)	3.53	1.99
Δ_{CLZ} (demand) (mm)	2.92	1.94
Demand/capacity	0.83	0.97

Note: Pairs of values correspond to measurements on each side of the wall.

^aThe values on the opposite side of the DIC.

**FIGURE 16** Demand versus capacity of the shear deformation of the CLZ according to CBA.

l_{ble} of W1 is 2.1 larger than that of W2, which is consistent with the earlier observations in the study. It can also be seen that the estimated demand Δ_{CLZ} and capacity $\Delta_{CLZ,u}$ agree reasonably well at failure.

Furthermore, Figure 16 shows the demand and capacity estimated at different load levels ranging from 80% to 100% of V_u . As evident from the results, the estimated capacity remains nearly constant for the load stages at which the crack measurements $w_{h,i}$ and $w_{v,i}$ were performed. The small variations in $\Delta_{CLZ,u}$ are due to respective small variations in the estimated angle α_Δ . The displacement capacity increases with the load, and approaches the capacity near failure. Therefore, the assessment method meets two necessary conditions: (1) the estimated capacity is not sensitive to the stage at which the crack measurements are performed, and (2) the estimated capacity and demand agree reasonably

well at failure. As shown in Table 4, the demand-to-capacity ratios at failure are 0.83 and 0.97 for W1 and W2, respectively.

These results indicate that the CBA is a promising approach that combines simplicity and accuracy for the assessment of cracked walls. Nevertheless, further experimental studies are needed to develop and validate the CBA, including cyclic and reverse cyclic tests with detailed experimental measurements.

8 | SUMMARY AND CONCLUSIONS

This paper presented a comprehensive framework for quantifying the shear resisting mechanisms of shear-critical short shear walls (shear-span-to-depth ratio $a/h < 3.0$). The proposed framework was applied to two full-scale shear walls tests with tension flanges. The experimental data included full displacement and strain fields across the walls obtained from DIC measurements. The obtained results allowed to validate the relative contribution of the shear resisting mechanisms of the 3PKT method for short shear walls at failure. Finally, a set of hypotheses towards crack-based assessment of the state of damage of the CLZ was introduced using the 3PKT.

The main conclusions of the study are:

- The proposed framework for quantifying shear resisting mechanisms of short shear-critical walls has proven its robustness for studying the shear behavior of such structural members. The framework was validated through horizontal and vertical equilibrium, where the evaluated total shear and axial vertical forces matched well with the applied external forces at failure.
- The geometric properties of the critical shear crack play an important role in influencing the shear behavior of short shear walls. In the case of W1, the CLZ was significantly larger than that of W2. This disparity resulted in a substantial difference in the CLZ contribution, accounting for 73% of V_u for W1 and 49% of V_u for W2. At the same time, larger CLZs exhibit larger deformations at failure, and thus result in wider critical cracks. This explains the negligible contribution of aggregate interlock in W1 compared to the substantial contribution in W2 (16% of V_u).
- The contribution of the transverse web reinforcement for both walls was almost identical since all reinforcement was yielding at failure.
- The contribution of dowel action was negligible for both walls due to the large tensile strains/stresses (close to yield) of the flexural reinforcement.

- The relative contribution of the shear resisting mechanisms predicted by the 3PKT is in a good agreement with that obtained based on the DIC measurements in tests W1 and W2.
- The presented hypotheses for crack-based assessment of short shear-critical walls combine simplicity and accuracy for the evaluation of the state of damage of the CLZ. This method relies solely on-site measurements of geometry and crack displacements without the need of other information such as reinforcement details and material properties. More experimental studies are needed to further develop and validate the CBA, including cyclic and reverse cyclic tests with detailed experimental measurements.

DATA AVAILABILITY STATEMENT

The data that support the findings of this study are available from the corresponding author upon reasonable request.

ORCID

Eissa Fathalla  <https://orcid.org/0000-0002-6284-2567>

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AUTHOR BIOGRAPHIES



Eissa Fathalla, Assistant Professor, Structural Engineering Department, Faculty of Engineering, Cairo University, Giza, Egypt; Postdoctoral Fellow, Department of ArGenCo, Urban and Environmental Research Unit (UEE), University of Liege, Liege, Belgium. Email: mmeissa@uliege.be



Boyan Mihaylov, Associate Professor, Department of ArGenCo, Urban and Environmental Research Unit (UEE), University of Liege, Liege, Belgium. Email: boyan.mihaylov@uliege.be

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