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Modeling of GFRP-Reinforced Squat Walls under Lateral Loading

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Abstract: This paper presents mechanics-based modeling approaches to understand the shear behavior of squat walls reinforced with glass fiber reinforced polymer (GFRP) bars when subjected to lateral loading. The applicability of design provisions in published specifications is examined using collated laboratory test data, resulting in the need for developing revised guidelines. Analytical studies are undertaken to evaluate the effects of reinforcement type on the response of load-bearing walls and to establish failure criteria as a function of various stress states in constituents. Obvious distinctions are noticed in the behavior of squat walls with steel and GFRP rebars owing to their different reinforcing schemes, tension-stiffening mechanisms, and material properties. Newly proposed equations outperform existing ones in terms of predicting the shear capacity of GFRP-reinforced squat walls. Furthermore, based on geometric and reinforcing attributes, a novel determinant index is derived for the classification of structural walls into squat and slender categories, which overcomes the limitations of prevalent methodologies based solely on aspect ratio. A practical method is suggested to adjust the failure mode of walls with GFRP rebars, incorporating a characteristic reinforcement ratio.

Keywords: glass fiber reinforced polymer (GFRP); failure mode, modeling; shear; squat wall
INTRODUCTION

Shear walls are indispensable for a building structure to accommodate lateral loads. Improper designs accelerate the deterioration of load-bearing members and bring about serviceability problems such as excessive sidesway. Placing shear walls in right locations ensures the stability of building frames and the large wall stiffness controls the horizontal displacement of constituents within an acceptable limit stipulated in specifications. When subjected to lateral loading, both ends of a wall (typically called boundary elements with concentrated reinforcing bars) carry tension and compression forces. These elements, which are essential if the maximum compression stress near the end of a wall exceeds a certain limit, are instrumental in resisting load reversals and inhibiting unanticipated buckling. Depending upon aspect ratio \(\frac{h_w}{l_w}\), where \(h_w\) and \(l_w\) are the height and length of a wall, respectively, shear walls are categorized as squat and slender; however, no absolute demarcation is available from a behavioral perspective: a ratio between \(\frac{h_w}{l_w} = 1.0\) and 2.0 often plays a role as a bifurcation point. Among other particulars, the shear strength coefficient \(\alpha_c\) of structural walls in ACI 318 may fulfill the demand for practical guidance (\(\alpha_c = 3.0\) for \(\frac{h_w}{l_w} \leq 1.5\), 2.0 for \(\frac{h_w}{l_w} \geq 2.0\), and a linearly interpolated value for \(1.5 < \frac{h_w}{l_w} < 2.0\) in the US customary units). These classifications can be interpreted in a way that an aspect ratio is reasonably taken to be below 1.5 for squat walls and the ratio between 1.5 and 2.0 indicates a transition from squat to slender walls.

The application of non-metallic reinforcement becomes commonplace around the globe; accordingly, a building code with glass fiber reinforced polymer (GFRP) rebars (ACI CODE-440.11-22) was recently published to direct practicing engineers. While high strength, light weight, nonmagnetic composition, and low maintenance are some of the many advantages that GFRP composites offer, corrosion resistance is the most notable benefit when incorporated
in concrete structures\textsuperscript{12}. On the use of GFRP reinforcement for shear walls, a consensus was not yet made. Some researchers argue that technical evidence is insufficient for field application\textsuperscript{13}; by contrast, others claim that the non-yielding nature of GFRP with a low elastic modulus improves the seismic performance of concrete members\textsuperscript{14,15}. As far as GFRP-reinforced squat walls are concerned, limited research has been reported and only a few experimental papers are available\textsuperscript{8,16}. Further studies are thus necessary to understand the behavior of squat walls with GFRP rebars and to expand the applicable boundary of these nontraditional construction materials.

This paper discusses an analytical model to examine the response of GFRP-reinforced squat walls under lateral loading. With the aim of overcoming the limitations of technical findings from test data\textsuperscript{8,16}, detailed mechanics is accounted for and design recommendations are elaborated. In addition, an alternative expression is suggested to identify a behavioral threshold between squat and slender walls, which is not simply reliant on an aspect ratio.

**RESEARCH SIGNIFICANCE**

The design of shear walls is empirical and heavily relies on practitioners’ experience without systematic derivations\textsuperscript{17,18}. Notwithstanding the broad adoption of GFRP reinforcement in concrete members, little is known about its usage in squat walls. Because the failure mechanism of squat walls differs from that of slender walls (that is, the former tends to fail in shear, accompanied by diagonal tension cracks, whereas the latter fails in flexure\textsuperscript{1}), archetypal methods that are predicated upon ductile responses cannot be applied. Furthermore, in view of deficient ductility in squat walls, attention should be paid to how premature shear failure can be precluded by employing adequate technical approaches. A refined mechanics-based model is developed to
elucidate the intrinsic behavior of squat walls with GFRP rebar, leading to the proposal of practical design equations.

**BACKGROUND**

Expository discussions are presented with regard to the shear behavior of GFRP-reinforced concrete walls. Codified design provisions are reviewed and evaluated using test data, including a comparative analysis that investigates functional differences between squat walls with steel and GFRP rebar.

**Potential Failure Modes**

As conceptually visualized in Fig. 1, a GFRP-reinforced concrete wall may fail in flexure, shear, or a combination thereof. For instructional purposes, the load path of the wall is approximated with idealized joints connecting compression and tension segments (dotted and solid lines in Fig. 1, respectively). When the wall’s aspect ratio is lower than a certain limit, its failure is governed by compression struts parallel to diagonal tension cracks in the web and by the crushing of the end zone (the squat wall domain in Fig. 1). If an aspect ratio is higher than the limit, the wall tends to bend like a cantilever fixed at the base and horizontal cracks formed within the tension zone; eventually, it fails by either the rupture of GFRP or the crushing of the concrete (the slender wall domain in Fig. 1). Contingent upon the properties of wall structures, a transition between these two scenarios can be seen.

**Design Method**

Since the design of squat walls with GFRP reinforcement has not been fully documented in published specifications, the coalescence of ACI 440.1R-15, ACI 318-19, and ACI 440.11-22 may be utilized. The nominal shear capacity of a wall ($V_n$) is expressed as

$$V_n = V_c + V_f \leq V_{n,\text{max}}$$

(1)
\[ V_{n,\text{max}} = k_1 f_c^{0.5} dt_w = k_2 f_c^{0.5} l_w t_w \]  

(2)

where \( V_c \) and \( V_f \) are the nominal shear resistance of the concrete and reinforcement, respectively;

\( k_1 \) and \( k_2 \) are empirical constants (\( k_1 = 10 \) and \( k_2 = 8 \) for US customary units and \( k_1 = 0.83 \) and \( k_2 = 0.66 \) for metric units); \( t_w \) and \( l_w \) are the thickness and length of the wall, respectively; and \( d \) is the effective depth (\( d = 0.8 l_w \)). The individual components of Eq. 1 is provided by

\[ V_c = k_3 f_c^{0.5} t_w kd = k_4 f_c^{0.5} k l_w t_w \]  

(3)

\[ k = \sqrt{2 \rho_f n_f + \left( \rho_f n_f \right)^2 - \rho_f n_f} \]  

(4)

\[ V_f = A_{f_v} f_{f_v} d / s = \rho_h f_{f_v} l_w t_w \]  

(5)

\[ f_{f_v} = \Omega E_f \leq f_{j_b} \]  

(6)

where \( k_3 \) and \( k_4 \) are empirical constants (\( k_3 = 5 \) and 0.4 and \( k_4 = 4 \) and 0.32 for the US customary and metric units, respectively); \( \rho_f \) is the reinforcement ratio (\( \rho_f = A_{f_v} / b d \), in which \( A_{f_v} \) is the cross-sectional area of the shear reinforcement and \( b \) is the width of the wall); \( n_f \) is the modular ratio (\( n_f = E_f / E_c \), in which \( E_f \) and \( E_c \) are the elastic moduli of the GFRP and concrete, respectively); \( s \) is the center-to-center spacing of the rebars; \( f_{j_b} \) is the design strength of the bent stirrup made of GFRP; and \( \Omega \) is the strain limit of the reinforcement (\( \Omega = 0.004 \)).

**Appraisal**

**Existing test data**—Figure 2(a) shows a ratio between the experimental and nominal shear capacities of GFRP-reinforced squat walls (\( V_{\text{test}} \) and \( V_n \), respectively). The properties of test specimens excerpted from Table 1 are as follows: aspect ratio (\( h_w / l_w \)) = 0.68 and 1.14, compressive strength of concrete (\( f_{c'} \)) = 33 MPa (4,790 psi) to 40 MPa (5,800 psi), tensile strength of GFRP (\( f_{j_h} \), \( f_{j_v} \) and \( f_{j_u} \) for horizontal and vertical rebars in Table 1, respectively) = 1,022 MPa (148 ksi) to 1,100 MPa (160 ksi), and horizontal and vertical reinforcement ratios (\( \rho_h \)
and \( \rho_v \), respectively) = 0.38% to 0.7%. The specimens with \( h_w/l_w = 1.33 \) in Table 1 were excluded due to a low reinforcement ratio in the boundary element (\( \rho_{be} = 1.43\% \)), which will be accounted for in a subsequent section. Although the number of test specimens in Fig. 2(a) is insufficient to render conclusive information, owing to a lack of available data, it is substantiated that Eq. 1 underestimated the capacity of the walls; especially, significant conservatism was noticed (\( V_{test}/V_n > 3.0 \)) when the aspect ratio was \( h_w/l_w = 0.68 \). These discrepancies are ascribed to the fact that the expression of \( V_c \) in Eq. 1 was empirically calibrated using flexure-shear-combined responses alongside large diagonal tension cracks\(^{12}\); on the contrary, the shear-dominated behavior of the squat walls with a low aspect ratio entailed narrow inclined cracks parallel to the compression struts (Fig. 1). Accordingly, an improvement is required to better predict the capacity of squat walls with GFRP rebars, which can avert the placement of unnecessary shear reinforcement.

The portion of the concrete and GFRP resistance (Eqs. 3 and 5, respectively) is allocated in Fig. 2(b). For consistency, the allowable strain limit of \( \Omega = 0.004 \) was employed to calculate \( V_f \) in all specimens. The gap between the test and prediction spanned from 0.48 to 0.76 and the degree of margin (\( V_{test} - V_n \)) was apparent when the aspect ratio dropped to \( h_w/l_w = 0.68 \) (the SSQ series). This tendency again confirms that the design approach of ACI 440.11-22\(^{11}\) does not cover GFRP-reinforced concrete squat walls.

**Comparison against steel reinforcement**—To figure out behavioral differences between GFRP and steel rebars in squat walls, a comparative assessment was made. For steel-reinforced walls, a total of 171 test data were collated from literature\(^{19-49}\) with the succeeding properties (those of GFRP-reinforced walls were delineated in the preceding section): \( h_w/l_w = 0.21 \) to 1.5, \( f'c = 20 \) MPa (2,900 psi) to 70 MPa (10,150 psi), \( \rho_h \) and \( \rho_v = 0.25\% \) to 2.8%, and \( f_y = 284 \) MPa (41 ksi) to
750 MPa (109 ksi), in which $f_y$ is the yield strength of the rebars. Figure 3 graphs the test capacities of the walls, which were normalized by the cross-sectional area and concrete strength \( (f'_{c,l_w}) \) to accommodate variable geometric and material properties, as a function of primary design parameters. While the normalized capacities of both steel and GFRP cases decreased with an increase in the aspect ratio (Fig. 3(a)), their response range differed in the ordinate: \( 0.03 \leq V_{\text{test}}(f'_{c,l_w}) \leq 0.33 \) for steel and \( 0.07 \leq V_{\text{test}}(f'_{c,l_w}) \leq 0.16 \) for GFRP. Analogous patterns were noted for the normalized horizontal reinforcement ratios \( (\rho_{h,f_y}f'_{c}) \) for steel and \( \rho_{h,f_y}f'_{c} \) for GFRP) and vertical reinforcement ratios \( (\rho_{v,f_y}f'_{c}) \) for steel and \( \rho_{v,f_y}f'_{c} \) for GFRP) given in Figs. 3(b) and (c), respectively. These distinct ranges of the wall capacities, depending upon the reinforcement type, can be explained by deriving the maximum horizontal reinforcement ratio \( (\rho_{h,\text{max}}) \) when the shear capacity of the walls \( (V_{n-wall}) \) is equivalent to their shear-strength limit \( (V_{n,\text{max}}, \text{Eq. 2}) \), which represents the most critical state in a squat wall system: diagonal tension failure equals web-crushing.

The \( V_{n-wall} \) expressions for the steel- and GFRP-reinforced concrete walls are attained from ACI 318 (Eq. 7)\(^5\) and ACI 440.11-22 (Eq. 1)\(^11\)

\[
V_{n-wall} = \left( \alpha_c \lambda f'_{c}^{0.5} + \rho_h f_{y} \right) l_w t_w \tag{7}
\]

where \( \alpha_c \) is the shear strength coefficient \( (\alpha_c = 3.0 \) and 0.25 for the US customary and metric units for an aspect ratio of \( h_w/l_w \leq 1.5 \), respectively) and \( \lambda \) is the concrete strength factor \( (\lambda = 1.0 \) for ordinary concrete). After setting Eq. 7 = Eq. 2 and Eq. 1 = Eq. 2 for the steel and GFRP-reinforced concrete walls, respectively, the horizontal reinforcement ratio \( (\rho_h) \) is solved, which is equivalent to the maximum reinforcement ratio of each instance \( (\rho_{h,\text{max}}) \)

\[
\rho_{h,\text{max}} = \psi_1 \frac{f'_{c}^{0.5}}{f_{y} h} \quad \text{for steel reinforcement} \tag{8}
\]
\[ \rho_{h,\text{max}} = \left( \psi_2 - \psi_3 k \right) \frac{f_c^{0.5}}{f_{vc}} \]  
for GFRP reinforcement \hspace{1cm} (9)

where \( \psi_1, \psi_2, \) and \( \psi_3 \) are constants (\( \psi_1 = 5, \psi_2 = 8, \) and \( \psi_3 = 4 \) for the US customary units and \( \psi_1 = 0.41, \psi_2 = 0.66, \) and \( \psi_3 = 0.34 \) for the metric units). As demonstrated in Fig. 3(d), the majority of reinforcement ratios in the steel-reinforced walls (136 specimens or 80\% of the entire samples) exceeded the maximum ratio (\( \rho_{h,\text{max}} \)); contrarily, most ratios of the walls with GFRP were close to or less than the maximum ratio. These observations clarify that the amount of rebars was generally greater in the steel-reinforced walls than their GFRP counterparts, which was related to the high strength of GFRP, and that the contribution of these rebars to the shear capacity of the walls was dissimilar, justifying the need for an independent design approach pertaining to GFRP-reinforced squat walls.

**MODELING**

To comprehend the ramifications of steel and GFRP rebars for the shear behavior of reinforced concrete squat walls, a twofold analytical model is formulated at element and structural levels. This section outlines an overview of modeling processes along with implementation steps and verification against test data.

**Element Level**

**Framework**—A unit square panel\(^5\) represented shear-loaded wall elements with steel and GFRP rebars. The panel concrete had a tensile strength of \( f_t = 1.8 \text{ MPa (260 psi)} \), resulting from \( f'_c = 30 \text{ MPa (4,350 psi)} \)\(^5\), and was orthogonally reinforced with the rebars at a reinforcement ratio of \( \rho = 0.25\% \) to 3.0\%. The lower bound of the ratio conformed to the requirement of ACI 318-19\(^5\), while the upper bound enveloped the ratios of the experimental specimens presented in Fig. 3. The yield and ultimate strengths of the steel and GFRP rebars were \( f_y = 420 \text{ MPa (60 ksi)} \) and \( f_{yu} = 1,100 \text{ MPa (160 ksi)} \) with elastic moduli of \( E_s = 200 \text{ GPa (29,000 ksi)} \) and \( E_f = 60 \text{ GPa (8,700 ksi)} \).
ksi), respectively. The stress-strain behavior of the panel was computed as per the procedure of the Modified Compression Theory\textsuperscript{51}, incorporating tension-stiffening that realistically considered interactions between the concrete and rebars.

**Tension stiffening**—A schematic representation of the tension-stiffening mechanism is shown in Fig. 4(a). The tensile stress of the reinforced concrete segment ($\sigma_t$) is calculated by the summation of rebar stresses inside the concrete ($f_j$) and the surrounding concrete ($((A_c/A_f - 1)f_t')$, in which $A_c$ and $A_f$ are the cross-sectional areas of the concrete and rebar, respectively, and $f_t'$ is the stress of the concrete with tensioning-stiffening)

$$\sigma_t = E_f \varepsilon_m + \frac{1-\rho}{\rho} f_t'$$

where $\varepsilon_m$ is the tensile strain of the reinforced concrete. For the representation of tension stiffening in GFRP-reinforced concrete, three candidate expressions were chosen\textsuperscript{52-54}

1. $f_t' = f_t \exp \left[-1100 (\varepsilon_m - \varepsilon_{cr}) \left(\frac{E_f}{200,000}\right)\right]$ \hspace{0.5cm} (11)
2. $f_t' = f_t \exp \left[-1500 (\varepsilon_m - \varepsilon_{cr}) \left(\frac{E_f}{200,000}\right)\right]$ \hspace{0.5cm} (12)
3. $f_t' = f_t \left[1 + \beta_1 (\varepsilon_m - \varepsilon_{cr}) \left(\frac{E_f}{200,000}\right)^\gamma\right]$ \hspace{0.5cm} (13)

where $\varepsilon_{cr}$ is the concrete strain at cracking; and $\beta_1$ and $\gamma$ are the tension-stiffening constants ($\beta_1 = 1,400$ and $\gamma = 0.8, 1.0, \text{and} 1.5$ for ribbed, sand coated, and helically wrapped GFRP bars, respectively\textsuperscript{54}). As plotted in Figs. 4(b) and (c), the downward propensity of Eqs. 11 and 12 was alike, whereas Eq. 13 overestimated the tension-stiffening effect. Given the marginal tension stiffening of GFRP-reinforced concrete members\textsuperscript{55}, Eq. 12 was used in this study. For the
occasion of steel-reinforced concrete, the tension stiffening model of Vecchio and Collins\textsuperscript{51} was adopted

\[ f'_t = f_t / \left(1 + (200\varepsilon_m)^{0.5}\right) \]  \hspace{1cm} (14)

**Constitutive relationship**—Figures 5(a) and (b) reveal the stress-strain relationship of the steel- and GFRP-reinforced concrete panels loaded in shear, respectively. To focus on the disparity of these rebar types, the average stress values (\(\nu\)) in the ordinate were normalized by the concrete strength (\(f'_c\)). The failure of the steel-reinforced panel was attributed to the yielding of the rebars combined with the crushing of the concrete (Fig. 5(a)), except for the heavily reinforced panel having \(\rho \geq 2.5\%\) that failed without yielding. On the GFRP-reinforced panel (Fig. 5(b)), concrete crushing was responsible for the failure with an exception of the lightly reinforced panel (\(\rho = 0.25\%\)). The low elastic modulus of GFRP caused much increase in strain under the same stress level, compared with the steel-reinforced case. Shown in Fig. 5(c) is a compilation of the maximum shear stresses with the reinforcement ratio of the panels. The rebar types obviously influenced the shear capacity of the panels, which reemphasizes the necessity of a customized model for GFRP-reinforced squat walls.

**Structural Level**

**Derivation**—A simplified free-body diagram of a failed squat wall (Fig. 6(a)) is illustrated in Fig. 6(b). In compliance with ACI 374.2R-13\textsuperscript{56}, the wall is loaded laterally and force equilibrium is achieved

\[ P = F_r + F_{hw} + F_c \]  \hspace{1cm} (15)

\[ C = T + F_{vw} \]  \hspace{1cm} (16)

\[ P = T \frac{l_w - b_{bc}}{h_w} + F_{hw} \frac{l_w - 2b_{bc}}{2h_w} \cot \theta + F_{vw} \frac{l_w - b_{bc}}{2h_w} \]  \hspace{1cm} (17)
where $P$ is the applied load; $F_t$ and $F_c$ are the resultant forces of the tension and compression boundary elements, respectively; $F_{hw}$ and $F_{vw}$ are the resultant forces of the web in the horizontal and vertical directions, respectively; $C$ and $T$ are the resistance of the boundary elements in compression and tension, respectively; $l_w$ and $h_w$ are the length and height of the wall, respectively; $b_{be}$ is the width of the boundary element; and $\theta$ is the crack angle in degrees. For the reason that the dowel action of GFRP rebars is negligible in a cracked plane, the $F_t$ term in Eq. 15 can be ignored. The horizontal force in the compression boundary element ($F_c$) is then obtained by combining Eqs. 15 and 17

$$F_c = P - F_{hw} = T \frac{l_w - b_{be}}{h_w} + F_{vw} \frac{l_w - b_{be}}{2h_w} + F_{hw} \left( \frac{l_w - 2b_{be}}{2h_w} \cot \theta - 1 \right) \quad (18)$$

The organizational format of Eq. 18 explains the load-bearing mechanism of the squat wall in Fig. 1, corroborated by the failure pattern of test specimens No. 6 to 11 in Table 1: the horizontal force ($F_c$) in Fig. 6(b) would increase with an increase in the aspect ratio of the web (related to $h_w/(l_w-2b_e)$ and $h_w/(l_w-b_e)$) and the vertical reinforcement in the web and the tension boundary element (concerned with $F_{vw}$ and $T$). Likewise, Eq. 18 can account for the failure mode of the slender wall in Fig. 1: the $F_c$ term decreases when the contribution of the vertical bars ($F_{vw}$) declines, which allows the progression of horizontal cracks along the web (that is, a precluded development of diagonal tension cracks). The linear elastic nature of GFRP rebars yields the succeeding expressions

$$F_{hw} = \frac{1}{2} E_f \varepsilon_h \left( l_w - 2b_{be} \right) t_w \cot \theta = \frac{1}{2} E_f \varepsilon_h A_{web} \cot \theta \quad (19)$$

$$F_{hv} = \frac{1}{2} E_v \varepsilon_v \left( l_w - 2b_{be} \right) t_w = \frac{1}{2} E_v \varepsilon_v A_{web} \quad (20)$$

$$T = \rho_{be} E_f \varepsilon_{be} b_{be} t_w = \rho_{be} E_f \varepsilon_{be} A_{be} = \rho_{be} E_f \varepsilon_{be} A_{be} \quad (21)$$
where $\rho_h$ and $\rho_v$ are the horizontal and vertical reinforcement ratios of the web, respectively; $\rho_{be}$ is the reinforcement ratio of the boundary element; and $\varepsilon_h$, $\varepsilon_v$, and $\varepsilon_{be}$ are the strains of the horizontal and vertical rebars and the boundary element, respectively. Because the web of a laterally loaded squat wall is subjected to uniform shear stress distributions\textsuperscript{57}, the strain of the vertical rebars along the cracked plane ($\varepsilon_v$) may be equated with that of the boundary element ($\varepsilon_{be}$) transmitting axial forces (Fig. 6(b)). This approximation ($\varepsilon_v = \varepsilon_{be}$) is supported by experimentally measured strains\textsuperscript{8}.

**Failure criteria**—Figure 7(a) depicts the possible failure modes of the squat wall model. Below is a succinct description on the individual cases

1) **Rupture of GFRP rebars in the web**: when the stress of the vertical and horizontal rebars ($f_v$ and $f_h$, respectively) is greater than the tensile strength of GFRP ($f_{tu}$), the rebars rupture. Contemplating that rebar strains at peak drift ratios in squat walls are generally smaller than the ultimate strain of commercially available GFRP rebars\textsuperscript{8,12}, the occurrence of this failure mode may be uncommon.

2) **Web-crushing**: the crushing failure of concrete in the web takes place if the principal compressive stress ($\sigma_{pc}$) reaches the softened concrete strength ($f_c$)

$$f_c = f'_c / (0.8 + 170\varepsilon_{pt})$$  \hspace{1cm} (22)

where $\varepsilon_{pt}$ is the principal tensile strain of the concrete. Equation 22\textsuperscript{58} denotes the degradation of concrete with an increase in the maximum normal strain when subjected to mechanical loading; in other words, the shear deformation of the web under the lateral load (Fig. 6) raises the principal strain, thereby weakening the concrete resistance without regard to the type of reinforcement. As such, Eq. 22 can be used for both steel- and FRP-reinforced concrete members\textsuperscript{59}.
3) **Rupture of GFRP rebars in the tension boundary element**: the rebars will rupture when their stress ($f_{te}$) equals the tensile strength ($f_{tu}$), which depends upon the amount of longitudinal rebars in the tension boundary element. Conventionally speaking, the tension boundary element of a shear wall transmits axial forces; thus, stress interactions between the normal and inclined components are negligible.

4) **Concrete crushing in the compression boundary element**: the combined shear and compression forces in the compression boundary element at the reference point associated with moment equilibrium (Fig. 6(b)) increase concrete stresses and prompt crushing failure ($\sigma_{pc} = f_c$). This failure type is frequently observed in squat walls tested in laboratories.$^{8,16}$

The notional explication of these failure modes is provided in Figs. 7(b) to (e). When the squat wall is loaded laterally, the stress and strain of the web increase in a steady manner (Fig. 7(b)). The stress states of the rebars and concrete in the web and the boundary elements are computed as detailed in the previous section, and those are compared against the early-mentioned failure criteria. The shear deformation of the web causes the elongation of the horizontal and vertical rebars (Fig. 7(c)) as well as the compression of the concrete (Fig. 7(d)). The lateral load also exerts axial tension and compression to the boundary elements (Figs. 7(c) and (e)). As drawn in Fig. 7(e), the shear-compression-combined action in the compression boundary element augments the concrete stress and can accelerate the development of the principal stress, resulting in the crushing of the concrete that is reported in laboratory research.$^{8,16}$
Implementation—The above-described model is solved with a procedure recapitulated in Fig. 8. Numerical iterations are necessary to determine the failure mode and load-bearing capacity of the squat wall:

**Step 1**: the geometric and material properties of the wall structure are collected as input parameters, including concrete and GFRP rebars.

**Step 2**: an initial shear strain in the web ($\gamma$) is assumed with a small fraction of the concrete cracking strain ($\gamma = 0.0005$ was chosen for the present study). Afterward, in accordance with the Modified Compression Field theory\textsuperscript{51}, the constituent strains of the concrete ($\varepsilon_{pt}$ and $\varepsilon_{pc}$, in which $\varepsilon_{pc}$ is the strain corresponding to the principal compressive stress $\sigma_{pc}$) and GFRP ($\varepsilon_{h}$ and $\varepsilon_{v}$) are calculated. Each of the four possible failure modes defined earlier is checked, belonging to the assumed shear strain.

**Step 3**: upon obtaining the strains in the web from Step 2, the forces in the boundary elements are computed ($C$, $F_{c}$, and $T$ in Eqs. 16, 18, and 21, respectively). For the failure of the tension boundary element, the tensile force $T$ is compared with the ultimate capacity of the rebar ($\rho_{refu}b_{refu}$). Regarding the compression boundary element, the maximum shear stress attained from the Modified Compression Field theory involving the compression force $C$ is multiplied by the cross-sectional area of the boundary element in order to ascertain the horizontal resistance $F_{c}$, which is evaluated against the shear strength of the element.

**Step 4**: the stresses and resultant forces from Step 3 are appraised per the criteria established in Fig. 7(a). If a failure condition is not satisfied, the shear strain $\gamma$ is increased ($\gamma_{i+1} = \gamma_{i} + \Delta\gamma$) and Steps 2 through 4 are repeated until a specific failure mode is found. Next, the nominal capacity of the squat wall ($V_n$) is quantified.
Verification—The proposed approach is validated employing the test data enumerated in Table 1. As witnessed in the laboratory, the predicted failure mode of the squat wall specimens was concrete-crushing in the compression boundary element (Fig. 7(a)). Figure 9(a) assesses the predictability of the nominal shear capacity ($V_n$). The capacity ratio of $V_{\text{test}}/V_n$ varied from 0.82 to 1.16 with the mean and standard deviation of 1.002 and 0.134, respectively. On the strain of the horizontal GFRP rebars in the web at the specimens’ peak loads, the theoretical values were comparable to the measured strain range. The strain limit of 0.004 in ACI.440.1R-15 served as the lower bound of the experimental strains (Fig. 9(b)), implying that this limit should be kept in the design of GFRP-reinforced squat walls.

DESIGN RECOMMENDATIONS

In an effort to improve the prediction of shear capacity in GFRP-reinforced walls, rational recommendations are made. Additionally, a new classification is proposed to definitize the taxonomy of squat and slender walls with an emphasis on not only wall geometries but also other attributes such as reinforcement ratios.

Proposed Revision

The shear capacity of the squat wall is composed of $F_c$ and $F_{hw}$ (Eq. 18). From a traditional design standpoint, the $F_c$ and $F_{hw}$ terms can be regarded as $V_c$ and $V_f$ in Eq. 1, respectively. Given that the shear-resisting mechanism of the compression boundary element (Fig. 6(b)) differs from the mechanism of conventional reinforced concrete beams accompanying dowel action and aggregate interlock, the existing expression of $V_c$ needs to be revised. Figure 10 instantiates a relationship between the capacity ratio of $V_{\text{test}}/V_n$ and the proportion of the concrete strength ($\alpha f'_c$, where $\alpha$ = fraction factor): conforming to the recommendation of prior research, the shear stress range of the walls at failure was represented by $\alpha f'_c$ with an upper limit of $0.3 f'_c$. 


For comparison, the $V_c$ term in Fig. 10(a) was set to be a product of the proportional stress and the cross-sectional area of the compression boundary element ($V_c = \alpha f'_c b_{betw}$). Within the scope of interest ($0.05 \leq \alpha \leq 0.3$), the capacity ratio gradually diminished with the fraction factor. The extent of discrepancy in the ratio was the least at $\alpha = 0.3$ and the corresponding average value of $V_{test}/V_n = 1.39$ was less than the value of 2.45 at $\alpha = 0.1$ ($0.1f'_c$ is equivalent to the current design expression of ACI 440.11-22\textsuperscript{(11)})). The enhanced capacity ratio with $\alpha = 0.3$ is attributed to the fact that the shear stress of $0.3f'_c$ generated higher resistance relative to the stress stemming from ACI 440.11-22\textsuperscript{(11)} and that the use of the compression boundary element ($b_{betw}$, Fig. 10(b)) in the cracked squat wall (Fig. 6) was more realistic than the use of the entire web in the existing design approaches\textsuperscript{5,11}. Consequently, Eq. 23 is suggested for Eq. 1

$$V_c = 0.3 f'_c b_{betw} = 0.3 f'_c \beta l_{tw}$$

(23)

where $\beta$ is the area ratio of the boundary element to the wall ($\beta = (b_{betw} / (l_{w}t_{w}))$. The nominal shear resistance of the squat wall is, therefore, written in conjunction with Eqs. 5 and 23

$$V_n = V_c + V_f = 0.3 f'_c \beta l_{tw} + \left(0.004E_f \right) A_{f} d / s \leq k_2f'_c^{0.5} l_{tw}$$

(24)

It should be noted that the allowable strain limit of $\Omega = 0.004$ in Eq. 6 was not modified as articulated in the Verification section.

**Determination of Failure Modes**

Unlike the traditional definition of squat walls based only on an aspect ratio, a new criterion may be established by manipulating the analytical model to encompass the unique features of GFRP-reinforced concrete walls. This attempt imparts technical merits since the reinforcing schemes of shear walls with steel and GFRP rebars are not the same. Rearranging Eqs. 18 to 21 yields Eq. 25 that manifests the strains of the horizontal and vertical GFRP rebars ($\varepsilon_h$, $\varepsilon_v$, and $\varepsilon_{be}$)
\[
F_c = \rho \varepsilon E_f e A_{be} \frac{l_w - b_{be}}{h_w} + \rho \varepsilon E_f e A_{web} \frac{l_w - b_{be}}{2h_w} + \rho f_{fu} A_{web} \cot \theta \left( \frac{l_w - 2b_{be}}{2h_w} \cot \theta - 1 \right)
\]  
(25)

where \(A_{be}\) and \(A_{web}\) are the cross-sectional areas of the boundary element and the web, respectively \((A_{be} = b_{be}t_w)\) and \((A_{web} = (l_w - 2b_{be})t_w)\). Aligning with the cracked web of the squat wall shown in Fig. 6, the angle \(\theta\) may be assumed to be 45° and the strain compatibility condition

\[
\cot^2 \theta = \left( \varepsilon_h + \varepsilon_{pc} \right) / \left( \varepsilon_v + \varepsilon_{pc} \right)
\]

in Vecchio and Collins\(^{51}\) enables

\[
\varepsilon_h = \left( \varepsilon_v + \varepsilon_{pc} \right) \cot^2 \theta - \varepsilon_{pc} = \varepsilon_v
\]  
(26)

Taking the previously-discussed uniform stress distribution of \(\varepsilon_v = \varepsilon_{be}\) and the strain limit of 0.004 stipulated in ACI 440.1R-15\(^{12}\),

\[
\varepsilon_h = \varepsilon_v = \varepsilon_{be} = 0.004
\]  
(27)

Then, Eq. 25 is restated as

\[
F_c = 0.25 \frac{f_{fu} A_{be} l_w - b_{be}}{h_w} + \rho f_{fu} A_{web} \frac{l_w - b_{be}}{2h_w} + \rho f_{fu} A_{web} \left( \frac{l_w - 2b_{be}}{2h_w} - 1 \right)
\]  
(28)

Dividing Eq. 28 by \(f_{fu} A_w\), in which \(A_w\) is the gross cross-sectional area of the wall \((A_w = l_w t_w = A_{web} + 2A_{be})\), provides a failure determinant index \((D)\)

\[
D = \frac{F_c}{f_{fu} A_w} = 0.25 \left( \frac{\rho F_{fu} A_{be} l_w - b_{be}}{A_w h_w} + \rho f_{fu} A_{web} \frac{l_w - b_{be}}{2h_w} + \rho f_{fu} A_{web} \left( \frac{l_w - 2b_{be}}{2h_w} - 1 \right) \right)
\]  
(29)

If this non-dimensional index is positive \((D > 0)\), the equilibrium condition depicted in Fig. 6(b) is satisfied; scilicet, the direction of the resultant force in the compression boundary element \((F_c)\) is opposite to the applied load \(P\). On the other hand, if the index is negative \((D < 0)\), the direction of these forces is the same; hence, the assumed crack angle of \(\theta = 45^\circ\) in Eqs. 26 and 29 becomes invalid and the angle has to be increased to comply with the equilibrium condition \((\theta > 45^\circ)\). In that circumstance, the crack pattern of the wall conforms to the archetypal pattern of a slender
wall (Fig. 11(a), inset). Equation 30 is thus adduced to discern the failure mode of structural walls with GFRP reinforcement

\[ D > 0 \rightarrow \text{squat walls with shear failure} \]
\[ D = 0 \rightarrow \text{transition with combined shear-flexural failure} \]
\[ D < 0 \rightarrow \text{slender walls with flexural failure} \] (30)

Allowing for the constituent terms in Eq. 29, GFRP-reinforced concrete walls with an aspect ratio of less than \( h_w/l_w = 1.5 \) can demonstrate flexural failure like the case of the slender category if their reinforcement ratios \((\rho_{be})\) are sufficiently low to precipitate horizontal tensile cracks. For instance, Fig. 11(b) displays the failure mode of the laboratory-tested squat walls listed in Table 1 as well as that of slender walls possessing aspect ratios greater than \( h_w/l_w = 2.0^{62,63} \). The specimens with an aspect ratio of \( h_w/l_w = 0.68 \) and \( 1.14 \) and a reinforcement ratio of \( \rho_{be} = 4.48 \) failed in shear \((D > 0)\), whereas the specimens with \( h_w/l_w = 1.33 \) were positioned in the \( D < 0 \) domain, which matches the flexural failure observed in the laboratory and proves that the aspect ratio of structural walls is not the only factor that divides the boundary between the squat and slender categories.

**Vertical Reinforcement in Boundary Elements**

A characteristic reinforcement ratio in the boundary elements \((\rho_{be,c})\) may be derived from the failure determinant function, which serves as a medium to adjust the failure mode of GFRP-reinforced concrete walls. At \( D = 0 \) in Eq. 29, the characteristic reinforcement ratio is specified to be

\[
\rho_{be,c} = \left( \rho_b \left( \frac{2h_w - l_w + 2b_{be}}{2(l_w - b_{be})} \right) - 0.5 \rho_r \right) \frac{(l_w - 2b_{be}) r_w}{b_{be} l_w} \] (31)

Equation 31 is a demarcation that apprehends whether a wall with GFRP rebars potentially fails in shear or flexure. If a reinforcement ratio in the boundary elements is greater than the
characteristic ratio \((\rho_{be,c} < \rho_{be})\), shear dominates as in the failure of a squat wall. For an engineering project, practitioners can tailor \(\rho_{be}\) to accomplish an intended failure of the subject wall. A concise version of Eq. 31 is offered by letting \(r_b = b_{be}/l_w\) and \(a_r = h_w/l_w\) under a usual reinforcing scheme of \(\rho_b = \rho_v\) in the web

\[
\rho_{be,c} = \rho_h \left( \frac{a_r - 1 + 1.5r_b}{1 - r_b} \right) (1/r_b - 2) \tag{32}
\]

**Parametric Studies**

The implications of geometric and reinforcing configurations for the failure of GFRP-reinforced concrete walls are visible in Figs. 12(a) through (c). A typical wall was selected (Specimen No. 8 in Table 1) for parametric investigations and its properties were used as the defaults, unless otherwise stated. Figure 12(a) exhibits the influence of a relative amount in placing vertical and horizontal rebars \((\rho_v/\rho_h)\). With the increased aspect ratio, the determinant index \((D)\) dwindled and the failure mode of the wall tended to shift from shear to flexure. The response curves were also affected by the vertical reinforcement ratio \(\rho_v\). Specifically, the placement of more vertical rebars retarded the transition of the failure mode because the shear friction of the wall ascended, so the load-bearing mechanism of the squat wall was preserved. The transformational threshold of \(D = 0\) that distinguishes the failure mode of the walls enveloped aspect ratios from \(h_w/l_w = 1.5\) to 2.0. This finding explicates the reason why a single aspect ratio was not suited for defining a limit between squat and slender walls, which was inconclusively argued in the structural concrete community\(^7\)\(^-\)\(^9\). The reinforcement ratio of the boundary elements \((\rho_{be})\) was influential in altering the failure mode of the walls (Fig. 12(b)). Even though the variation trend of \(D\) was similar to the case of Fig. 12(a), the impact of \(\rho_{be}\) was prominent in comparison with \(\rho_v\); namely, depending upon the value of \(\rho_{be}\), a GFRP-reinforced concrete wall with \(h_w/l_w > 2.0\) can still fail in shear like the occasion of a squat wall. The growth of the characteristic reinforcement ratio \((\rho_{be,c})\)
comprising a representative boundary element size of \( r_b = 0.1 \) is plotted in Fig. 12(c). The elevated slope of the characteristic ratio \( (\rho_{be,c}) \) with the reinforcement ratios of the web \( (\rho_v \text{ and } \rho_h) \) points out that the balanced failure condition of the wall \( (D = 0) \) necessitated more rebars as its aspect ratio rose, reaffirming the significance of GFRP amounts in classifying squat and slender walls. It is, however, worth noting that the reliance of the web reinforcement ratios disappeared when the aspect ratio was below \( h_w/l_w = 0.85 \): the structural member was sorted into a squat wall that failed in shear, regardless of the reinforcement ratios.

**SUMMARY AND CONCLUSIONS**

This paper has dealt with mechanics-based analytical modeling to construe the shear behavior of GFRP-reinforced squat walls when subjected to lateral loading. Through a rigorous review of existing design articles in tandem with experimental data, the limitations of current specifications were explored and the need for developing amended guidelines arose. Two-phase examinations, from local and global points of view, bring to light the influence of reinforcement type on the response of squat walls and their failure criteria as regards various stress states in structural components. A rational design proposal was made, coupled with a novel determinant index assorting load-bearing walls into squat and slender categories. Moreover, a characteristic reinforcement was rendered to assist engineering professionals in allocating architectural elements. The following are concluded.

- The provisions of ACI 440.11-22\textsuperscript{11} underestimated the shear capacity of GFRP-reinforced squat walls, particularly noticeable when an aspect ratio was as low as \( h_w/l_w = 0.68 \), owing to the empirical nature of the equations originating from flexure-shear-combined responses.
The behavioral differences of squat walls with steel and GFRP rebars were evident in terms of failure characteristics and shear stress developments. The source of these discrepancies was reinforcing amounts, tension-stiffening mechanisms, and material properties.

The mechanics-based model ameliorated the accuracy of predicting the shear capacity of GFRP-reinforced squat walls and led to the derivation of revised expressions, constituted with the cross-sectional area of the compression boundary element and the maximum allowable rebar strain of 0.004.

Contrary to the prevalent methodologies relying on ambiguous aspect ratios, the determinant index demystified the classification of squat walls by utilizing the geometric and reinforcing attributes of the walls.

The suggested characteristic reinforcement ratio would facilitate the adjustment of failure modes in GFRP-reinforced concrete walls involving an aspect ratio greater than $h_w/l_w = 0.85$, below which shear would be the dominant failure mode irrespective of reinforcing schemes in the boundary elements.

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Table 1. Summary of existing test programs on GFRP-reinforced squat walls [1 mm = 0.0394 in.; 1 MPa = 145 psi]

<table>
<thead>
<tr>
<th>No.</th>
<th>Reference</th>
<th>Specimen</th>
<th>$h_w$ (mm)</th>
<th>$l_w$ (mm)</th>
<th>$h_b/h_w$</th>
<th>$t_w$ (mm)</th>
<th>$t_b$ (mm)</th>
<th>$f'_c$ (MPa)</th>
<th>$f_{ub}$ (MPa)</th>
<th>$f_{ut}$ (MPa)</th>
<th>$f_{re}$ (MPa)</th>
<th>$p_{h}$ (%)</th>
<th>$p_{v}$ (%)</th>
<th>$p_{be}$ (%)</th>
<th>$p_{i}$ (%)</th>
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<th>$V_{test}$ (kN)</th>
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Notes: $h_w$ = wall height; $l_w$ = wall length; $t_w$ = wall thickness; $b_{be}$ = boundary element width; $f'_c$ = concrete compressive strength; $f_{ub}$ = tensile strength of web horizontal GFRP rebar; $f_{ut}$ = tensile strength of web vertical GFRP bar; $f_{u,be}$ = tensile strength of GFRP rebar in boundary elements; $p_{h}$ = web horizontal reinforcement ratio; $p_{v}$ = web vertical reinforcement ratio; $p_{be}$ = vertical reinforcement ratio in boundary elements; $N/(Af'_c)$ = axial load ratio applied to top of wall; $V_{test}$ = experimental capacity.

a) lateral drift at failure
Fig. 1. Conceptual failure modes of GFRP-reinforced concrete walls
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\[
\frac{T}{A_f} = \sigma_t = f_t + \left( \frac{A_r}{A_f} - 1 \right) f', \quad \rho = E_f \varepsilon_m + \frac{1}{\rho} f',
\]

**Fig. 4.** Tension stiffening of GFRP-reinforced concrete: (a) schematic representation; (b) progressive reduction of tensile stress in concrete; (c) strain-dependent response
Fig. 5. Element-level shear behavior: (a) steel-reinforced concrete panel; (b) GFRP-reinforced concrete panel; (c) comparison of maximum shear stresses.
Fig. 6. Analytical model: (a) test observation; (b) simplified free body diagram
Rebar rupture in boundary element
Rebar rupture in web
Web-crushing
Concrete crushing in compression zone

$$\sigma_{pc} = \frac{pc}{c} \tau = \frac{pc}{c}$$

Web shear stress

Shear capacity of web
Shear demand

Web shear strain

Web shear strain

Rebar tensile stress

$$f_u$$ (tensile strength)

Web shear strain

Vertical rebar in tension boundary element
Horizontal and vertical rebar in web

Fig. 7. Potential failure modes: (a) components; (b) stress-stain in web; (c) rebar stresses in web and tension boundary element; (d) concrete stress in web; (e) concrete stress in compression boundary element
Assume initial shear strain of web, $\gamma$

Calculate strain components of web ($\varepsilon_{pt}$, $\varepsilon_{pc}$, $\varepsilon_h$, and $\varepsilon_v$) for given shear strain ($\gamma$)

Calculate resultant forces ($C$, $F_c$, and $T$) in comp. and tension boundary elements

Compare capacity and demand of web, compression, and tension boundary element

Check failure conditions in constituent elements

Shear strength ($V_n = P$)

Fig. 8. Flowchart for implementation of proposed model
Fig. 9. Validation of proposed model: (a) shear capacity; (b) strain of horizontal rebars at peak load

[1 kN = 0.225 kips]
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