Ultimate Shear behavior and Modeling of Reinforced Concrete Members Jacketed by FRP and Steel

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Introduction

Existing reinforced concrete (RC) columns designed and constructed according to the old seismic specifications failed catastrophically under earthquake excitations owing to several major deficiencies: insufficient shear strength, insufficient flexural strength and low ductility. These column failures have led to numerous studies on a variety of strengthening techniques, especially concerning the seismic jacketing of existing deficient columns. This is because seismic jacketing greatly improves compressive strength, ultimate strength capacity, and ductility. Strengthening techniques based on jacketing usually involving various alternative jacketing materials—steel plate and Fiber Reinforced Polymer (FRP)—have been demonstrated to enhance the seismic performance and ductility. The stress-strain curves of various materials are shown in Fig. 1. Aboutaha et al. [1] tested several types of alternative RC columns with steel jackets, including rectangular solid steel jackets and partial steel jackets. They found that a rectangular steel jacket can improve the seismic performance of reinforced concrete columns with inadequate shear resistance. After that, Anggawidjaja et al. [2] proved that FRP jacketing such as Aramid Fiber Reinforced Polymer (AFRP), Polyethylene Naphthalate (PEN) and Polyethylene Terephthalate (PET) can be used to replace steel jacketing, because of their great promise in terms of corrosion resistance, long-term durability and fast installation.

The need for a precise analytical approach to assess the actual inelastic structural response of seismic-jacketed RC columns, especially at the large ductility level, has been increasingly a concern among design engineers. However, in spite of the interest in this field, there has been no attempt to develop analytical tools to describe the behavior of RC columns strengthened by alternative seismic jacketing materials. Modern design codes and standards [3] are to date only applicable for certain types of jacketing materials because of the complex interaction in the composite structures between the existing RC column and jacket.

In this paper, we propose an analytical method that considers the complex interaction between the flexure and shear strength behaviors, termed the Flexure-Shear Interaction (FSI) method, which predicts the behavior of RC columns jacketed by various types of alternative materials with varied jacketing shapes. In this analytical method, a traditional section analysis is applied together with a truss mechanism. As part of the overall design process, our FSI analytical method is expeditious and efficient for use in predicting structural response while maintaining a high degree of accuracy. This FSI analytical method is verified using existing experimental results, and we demonstrate shear strengthening and ductility enhancement of RC columns confined with various types of jacketing materials, including steel plate, CFRP, AFRP, PEN and PET FRP sheets.

Fig. 1 Stress-strain curves of various materials

Experimental Programs

Detail of Specimens

Ten simply-supported RC beams designed to fail in shear were subjected to four-point bending loads, as shown in Fig. 2a. The locations of strain gauges and Linear Variable Differential Transformers (LVDTs) are illustrated in Figs. 2b and 2c. Using RC beams rather than RC columns as the test members allows the elimination of the effects of pull-out from footings and lateral buckling of the longitudinal reinforcement, enabling more accurate shear deformation measurement. However, the current study of shear behavior is applicable only for the cases, in which axial loading is not a significant concern (e.g. bridge columns). Two groups of RC beams were prepared:

1. Group 1 included a reference RC beam (SP1) and five RC beams fully wrapped with different amounts of FRP composites (SP2–SP6), all with identical longitudinal and transverse steel reinforcement as the reference beam but with different strengthening ratios of FRP. Each specimen had a cross section of 250 mm (width) × 270 mm (height), whose corners
were chamfered with a radius of 11 mm, and the shear span was 600 mm. The longitudinal reinforcement and transverse steel reinforcement ratios were 2.53% and 0.17%, respectively, in all the six specimens, whereas the volumetric ratio (i.e., calculated based on the nominal thickness of the LRS FRP sheets) of the wrapped PET FRP sheets varied from 0.11% to 0.45%.

(2) Group 2 included four RC beams (SP7–SP10) that had different sectional dimensions and shear-span to effective-depth ratios to the reference beam. This group was designed to investigate the effects of the longitudinal reinforcement ratio and shear-span to effective-depth ratio. SP7, representing a deep beam, with dimensions 250 mm (width) × 500 mm (height) and a shear span of 1125 mm (see Table 1), whereas SP8 and SP9 had dimensions of 250 mm (width) × 270 mm (height) and a shear span length of 600 mm. SP10, representing a small section of beam, had dimensions 100 mm (width) × 150 mm (height) and a shear span of 300 mm. The specimen corners were chamfered with a radius of 11 mm. SP7 and SP9 were designed to have a similar shear strengthening ratio and shear-span to effective-depth ratio as SP5, whereas the longitudinal reinforcement ratio was made different. SP8 had a similar shear strengthening and longitudinal reinforcement ratio as SP3, whereas the shear-span to effective-depth ratio was made different. SP10 had a large spacing of transverse reinforcement significantly less than that required in the JSCE-2007 specification [3].

**Experimental Results**

The overall shear force vs. mid-span deflection responses of specimens SP1 to SP6 and SP7 to SP10 are presented in Figs. 3a and 3b, respectively; the shear force ($V_i$) is presented using a nominal shear stress ($\tau_i$) by dividing the shear force by the effective cross section (i.e., $\tau_i = V_i/\beta_{total}$). The mid-span deflection is presented by the drift ratio ($\delta$), which is defined as the ratio of the mid-span deflection ($\beta_{total}$) to the shear span ($a$).

The reference specimen (SP1) showed a linearly increasing portion until the peak load and a sudden drop of the load-carrying capacity afterwards, indicating a typical brittle shear failure of the member. During the tests of specimens SP2 to SP5 the evolution of the member’s mid-span deflection was terminated at the rupture of PET FRP sheets. The corners in SP5 were not well rounded, resulting in the premature rupture of FRP at a corner, and subsequently a lower ultimate ductility was achieved compared to SP4. For SP6, which failed in flexure, neither FRP rupture nor the decrease in shear capacity was observed even at the drift ratio of 12%, at which point the test was stopped owing to the extremely large deformation. It is interesting that specimens SP2 to SP5 also exhibited significant ductility although they failed in shear. The nominal shear stress achieved in the peak of the linear portion of the load-deflection response increased with the amount of PET fiber sheets, as did the drift ratio. This is because that, with increasing strengthening ratio, the confinement provided by LRS FRP not only prevented concrete from spalling off but also restrained the widening of shear cracks. The considerable ductility development before the member’s shear failure seems to be a unique characteristic of PET FRP-strengthened RC members. In other words, the shear failure is no longer brittle.
In the second group, SP10 was subjected to a brittle shear failure, and exhibited a load-deflection response similar to that of the reference SP1. The nominal shear strength of SP10 was the highest among all the specimens mainly because it had the smallest sectional dimensions (Fig. 3b). SP7 to SP9 exhibited ductile shear failure (Fig. 3b). SP7 and SP9 had similar shear-span to effective-depth ratio and strengthening ratio as SP5. The difference between these three specimens were their longitudinal reinforcement ratios; SP9, which had the lowest value, achieved the highest shear ductility, as shown in Fig. 3b, because of its highest shear to flexural strength ratio. SP7 exhibited the smallest ductility owing to its higher longitudinal reinforcement ratio, as shown in Fig. 3b. The largest sectional dimensions of SP7 may also be the reason for its lower shear ductility, because concrete degradation may be faster in the case of large-depth RC beams owing to the widening of concrete cracks in the web.

### Analytical Method

#### Flexural strength

To calculate the flexural strength, a section analysis is performed by dividing the section area into a number of discrete strips, and it is assumed that plane sections remain planes at any loading level. In this analysis, the increments in strain for the compression at the top fiber are fixed, and the strain across the depth of the cross-section is assumed to be proportional to the distance from the neutral axis, as shown in Fig. 4.

![Cross section and stress-strain compatibility](image)

**Fig. 3 Section analysis**

In the flexural strength model, the enhancement of the flexural strength is a consequence of the confined concrete stress-strain relation. For a given flexural cross section, the force and moment equilibrium conditions are given by Eq. (1) and Eq. (2), respectively. The corresponding shear force, $V_{mu}$, is obtained using Eq. (3).

$$P = \sum_{i=1}^{n} \sigma_{ci} A_{ci} + \sum_{j=1}^{m} \sigma_{cj} A_{cj}$$  \hspace{1cm} (1)

$$M = \sum_{i=1}^{n} \sigma_{ci} A_{ci} d_i + \sum_{j=1}^{m} \sigma_{cj} A_{cj} d_j$$  \hspace{1cm} (2)

$$V_{mu} = \frac{M}{a}$$  \hspace{1cm} (3)

where

- $\sigma_{ci}$ = stress in $i^{th}$ concrete layer
- $\sigma_{cj}$ = stress in $j^{th}$ longitudinal reinforcement
- $d_i$ = distance from top fiber to the centroid of $i^{th}$ concrete layer
- $d_j$ = distance from top fiber to the centroid of $j^{th}$ steel layer

$A_{ci}$ = area of $i^{th}$ concrete layer
$A_{cj}$ = area of $j^{th}$ longitudinal reinforcement
$P$ = axial force (N)
$M$ = moment at the considered cross section (N.mm)
$a$ = shear span (mm)
$i, j = 1, 2, 3…n$ or $m$

As expressed in Fig. 4(b), the strain compatibility equations of the $i^{th}$ concrete and the $j^{th}$ longitudinal reinforcement are given by the following equations:

$$\epsilon_{ci} = \epsilon_{ce} + \frac{\sigma_{ci} - \sigma_{ce}}{E_{ce}} d_i$$  \hspace{1cm} (4)

$$\epsilon_{cj} = \epsilon_{ce} + \frac{\sigma_{cj} - \sigma_{ce}}{E_{ce}} h - d_j$$  \hspace{1cm} (5)

From the section analysis, the secant modulus of material ($E_{ce}$, $E_{ja}$, $E_{sec}$, $E_{ec}$), the effective strength of concrete ($f'_{ce}$), strain at the extreme fiber ($\epsilon_{ce}$) and neutral axis depth ($x$) can be obtained. These parameters are applied in the shear strength model. However, the neutral axis depth used in the flexural strength model is not the same as that used in the shear strength model (Fig. 4) because of the shear crack opening.

#### Shear strength

The main limitation of the section analysis is that the effect of the shear strength behavior—shear crack opening and reduction of the neutral axis depth—on the flexural strength is not taken into account. The post-peak region of the load-deformation response is dominated by the shear strength behavior. To account for this, a truss mechanism approach proposed by Sato et al. [4] is combined with the section analysis to predict the shear strength more precisely.

![Shear strength model by Sato et al. [4]](image)

**Fig. 5 Shear strength model by Sato et al. [4]**

Fig. 5 illustrates the concept of the shear strength model based on the truss mechanism. Previous experimental observations by Sato et al. [4] showed that the shear strength of RC columns depends significantly on the secant stiffness of the flexural and shear reinforcements. As explained previously, the secant stiffness is obtained from the stress-strain relations of materials which satisfy the compatibility and equilibrium conditions in the flexural strength model. Moreover, the shear strength capacity continuously decreases after the yielding of the shear reinforcement, because the shear reinforcement contribution shows no further increase.

The total shear strength can be expressed as a sum of the contribution of the concrete ($V_c$) and the shear reinforcement ($V_{su}$), as shown in Eq. (6). This shear reinforcement consists of contribution from steel shear reinforcement and jacket.

$$V_{su} = V_c + V_{s+j}$$  \hspace{1cm} (6)
The parameters of the shear strength model are based on the experimental shear force component in the post-peak region, which is the potential region for dominant shear strength, because the observed shear force for a given deformation can be considered as the remaining shear strength for that deformation. The shear strength can be assumed to depend on four parameters: the secant stiffness of the flexural reinforcement ($\rho_sE_u$), the steel shear reinforcement and jacket ($\rho_sE_{ws}+\rho_fE_p$), the shear-span-to-depth ratio ($a/d$) and the effective concrete strength ($f_{cd}$). Using a non-linear regression analysis, the concrete contribution to shear strength can be written as:

$$V_c = \beta_d \cdot \beta_s \cdot f_{cd} \cdot b \cdot d$$

(7)

where $f_{cd} = 0.2 \sqrt{f_y} \cdot \beta_d = \sqrt{a/d} \cdot \beta_s = \sqrt{\frac{P}{2.5A_y f_{ws}}}$, $\beta_s = \sqrt{\rho_sE_{ws}+\rho_fE_p}$

**Verification**

Steel jacketing is a more common retrofit technique than concrete jacketing. Aboutaha et al. [1] proved that steel jacketing shows several advantages: a smaller increase in the cross-sectional dimensions, ease and speed of construction, lower cost of structural intervention and interruption of use, and a smaller increase in additional stiffness to the retrofitted column. Fig. 6 demonstrates the load-deformation responses from the experiment carried out by Aboutaha et al. and from the analytical method. With an rectangular jacket (Specimen SC10 in Fig. 6(a)), the column showed subsequent improved behavior compared to the control column (Specimen SC9 in Fig. 6(b)).

![Fig. 6 Load-deflection relations of database [1]](image)

The load-deformation responses of specimens tested by Anggawidjaja et al. are shown in Fig. 7. For specimens SP1-4, the effectiveness of Aramid, PEN, and PET fiber jacket in strength and ductility enhancement is examined, as shown in Figs. 7(a)-(d). The experimental results reveal that specimen SP4 with PET has higher ductility than PEN (SP3), Aramid (SP2) and the control (SP1) specimens. The analytical results for SP1-4 correlate well with the experimental results. The FSI analytical method can accurately predict the behavior of specimens SP2 and SP3 failing in flexure, and that of specimens SP1 and SP4 failing in low or high ductile shear. For flexure failure, the analytical results of specimens SP2 and SP3 show the reduction of load-carrying capacity due to the buckling of reinforcement. For ductile shear failure, specimens SP1 and SP4 clearly show the decrease in the load-carrying capacity at the intersection of flexural and shear strengths.

**Fig. 6 Load-deformation relations of database [2]**

**Conclusion**

An analytical method is presented taking into account the interaction of flexural and shear strength models. The conclusions of this research study can be summarized as follows:

1. The flexure-shear interaction analytical method can successfully predict the load-deformation responses of RC columns with various alternative jacketing materials: steel plate, concrete, and FRP materials. The analytical method is also applicable to both rectangular and elliptical cross-sections of steel jacket.

2. The flexural strength model connects with the shear strength model through the neutral axis depth of the shear crack region, whereas the shear strength model connects with the flexural strength model through the yielding of reinforcement.

3. The proposed shear strength model can predict the shear strength of reinforced concrete columns with various jacketing materials in the post-peak region. The experimental results show good correlation with the proposed predictive model.

**References**


