A NEW AND UNIFIED MODEL FOR PREDICTION OF ULTIMATE DEFORMATION OF LINEAR CONCRETE MEMBERS WITH AND WITHOUT JACKETING

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ABSTRACT

Existing prediction models for peak load of reinforced concrete columns with jacketing are mostly empirical and are different for different jacketing materials. For rational seismic design, ultimate deformation is a better limit state than peak load, however there are only few prediction models available. This study presents a new and unified analytical method with potential shear strength concept and consideration of flexure-shear interaction, which is applicable to both cases with and without jacketing. The method can be applied to various jacketing materials; reinforced concrete, steel plate and FRP sheet including conventional FRP and large rupturing strain (LRS) FRP. The model consists of flexure strength/stiffness model and shear strength/stiffness model. The flexure model interacts with the shear model through the reduced neutral axis depth and the increased tensile force in tension reinforcement by tension shift in shear cracking zone. On the other hand the shear model interacts with the flexure model through the yielding of tension reinforcement and the concrete strength degradation in compression softening. The method can predict the load-deformation relationship including post-peak region and better ultimate deformation with LRS FRP sheet in the experiments of this and previous studies.

KEYWORDS

Ultimate deformation, RC linear members, FRP jacketing, steel jacketing, shear strength, shear deformation, shear-flexure interaction, prediction model.

INTRODUCTION

For seismic design, ultimate deformation is usually a limit state, which is defined as a certain deformation in post-peak region of load-deformation relationship of members. Since the technological knowledge on seismic design has been still improved, many structures, which were designed and constructed under old seismic design codes, need seismic retrofit. The most typical method for seismic retrofit is jacketing. There are various materials applied for jacketing; reinforced concrete, fiber reinforced cementsitious material, steel plate and fiber reinforced polymer (FRP) sheet. The seismic design methods for jacketing are usually with some empirical nature, so that they are different for different materials. Furthermore, there are only few prediction methods available for ultimate deformation (JSCE 2010).

Considering the above fact as the background, this study was conducted to propose a new and unified model for predicting the ultimate deformation in post-peak region of linear concrete members (beams/columns) failing in either shear or flexure with and without jacketing of FRP and/or steel. The proposed model includes flexure and shear strength/deformation models in which flexure-shear interactions are considered. The proposed model can predict reinforcement strain, meaning that it can predict FRP sheet rupture at ultimate. The predicted load-deformation relationships were compared with experimental results of this study and past studies to show its reliability and proofs that PET FRP sheet with a large rupturing strain (LRS) can achieve a better ultimate deformation than conventional FRP sheet.

EXPERIMENTAL PROGRAMS

Details of Specimens

Ten simply-supported RC beams designed to fail in shear were subjected to four-point bending loads, as shown in Figure 1(a). The locations of strain gauges and Linear Variable Differential Transformers (LVDTs) are
illustrated in Figures 1(b) and 1(c). 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 to 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 of 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 of 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 Standard Specification (JSCE 2010).

The overall shear force vs. mid-span deflection responses of specimens SP1 to SP6 and SP7 to SP10 are presented in Figures 2(a) and 2(b), respectively; the shear force ($V_t$) is presented using a nominal shear stress ($\nu_t$) by dividing the shear force by the effective cross section (i.e., $\nu_t = V_t / bd$). The mid-span deflection is presented by the drift ratio ($\delta$), which is defined as the ratio of the mid-span deflection ($\Delta_{mid}$) 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 (Figure 2(a)). 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 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 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.

PROPOSED MODEL

Flexural Strength and Deformation

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 Figure 3.

In the flexural strength model, the enhancement of the flexural strength is a consequence of the confined concrete stress-strain relationship. For a given flexural cross section, Eq. 1 and Eq. 2 give the force and moment equilibrium conditions, respectively. The corresponding shear force, $V_{\text{max}}$, is obtained using Eq. 3.
\[
P = \sum_{i=1}^{n} \sigma_{ci} A_{ci} + \sum_{j=1}^{m} \sigma_{sj} A_{sj}
\]
(1)
\[
M = \sum_{i=1}^{n} \sigma_{ci} A_{ci} d_i + \sum_{j=1}^{m} \sigma_{sj} A_{sj} d_j
\]
(2)
\[
V_{pu} = M / a
\]
(3)

where \(\sigma_{ci}\) and \(\sigma_{sj}\) is stress in \(i^{th}\) concrete layer and \(j^{th}\) longitudinal reinforcement, respectively, \(d_{i}\) and \(d_{j}\) is distance from top fiber to the centroid of \(i^{th}\) concrete layer and \(j^{th}\) steel layer, respectively, \(A_{ci}\) and \(A_{sj}\) is area of \(i^{th}\) concrete layer and \(j^{th}\) longitudinal reinforcement, \(P\) is axial force (N), \(M\) is moment at the considered cross section (N-mm), \(a\) is shear span (mm), and \(i, j = 1, 2, 3\ldots n\) or \(m\).

As expressed in Figure 3(b), the strain compatibility equations of the \(i^{th}\) concrete and the \(j^{th}\) longitudinal reinforcement are given by the following equations:

\[
\varepsilon_{ci} = \varepsilon_{el} + (\varepsilon_{cc} - \varepsilon_{el}) \frac{d_i}{h}
\]
(4)
\[
\varepsilon_{sj} = \varepsilon_{el} + (\varepsilon_{cc} - \varepsilon_{el}) \frac{h - d_j}{h}
\]
(5)

From the section analysis, the secant modulus of material \((E_{se}, E_{je}, E_{we}, E_{ce})\), the effective strength of concrete \((f'_{ce})\), strain at the extreme fiber \((\varepsilon_{cc})\) 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 (Figure 4) because of the shear crack opening.

Flexural deformation is calculated by doubly integrating strain obtained by the section analysis. The tension shift, which is explained by the truss mechanism (see the section of Shear Strength and Deformation) and causes increase in strain of tension reinforcement, increases flexural deformation.

Shear Strength and Deformation

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. (1997) is combined with the section analysis to predict the shear strength more precisely.

Figure 4 illustrates the concept of the shear strength model based on the truss mechanism. Previous experimental observations by Sato et al. (1997) 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 relationships of materials, which satisfy the compatibility and equilibrium conditions in the flexural strength model. 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 \((V_{s+j})\), as shown in Eq. 6. This shear reinforcement consists of contribution from steel shear reinforcement and jacket.

\[
V_{su} = V_c + V_{s+j}
\]
(6)
The parameters of the shear strength model are based on the experimental shear force component in the post-peak region, because the observed shear force for a given deformation can be considered as the remaining shear strength (or potential shear strength) for that deformation. The shear strength can be assumed to depend on four parameters: the secant stiffness of the flexural reinforcement \(\rho_s E_s\), the steel shear reinforcement and jacket \(\rho_w E_w + \rho_j E_j\), the shear-span to effective-depth ratio \((a/d)\) and the effective concrete strength \(f'_{ce}\). Using non-linear regression analysis, the concrete contribution to shear strength can be written as:

\[
V_c = \beta_d \cdot \beta_p \cdot \beta_s \cdot \beta_w \cdot f_{cd} \cdot b \cdot d
\]

where \(f_{cd} = 0.2 (f'_{ce})^{0.5} \cdot \beta_d = \sqrt{a/d} \cdot \beta_p = \sqrt{\frac{P}{2.5A_p f_{tu}}} \cdot \beta_s = \sqrt{\rho_s E_s} \) and \(\beta_w = \sqrt{\rho_w E_w + \rho_j E_j}\).

Shear deformation becomes significant after shear cracking and is obtained by the truss mechanism approach as shown in Figure 5 (Ueda et al. 2004). The original shear deformation model is expanded for post-yielding range of tension and shear reinforcement.

Verification

Steel jacketing is a more common retrofit technique than concrete jacketing. Aboutaha et al. (1999) 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. Figure 6 demonstrates the load-deformation responses from the experiment carried out by Aboutaha et al. (1999) and from the analytical method. With rectangular jacket (Specimen SC10 in Figure 6(b)), the column showed subsequent improved behavior compared to the control column (Specimen SC9 in Figure 6(a)).

The load-deformation responses of specimens tested by Anggawiddjaja et al. (2006) are shown in Figure 7. For specimens SP1-4, the effectiveness of Aramid, PEN and PET fiber jacket in strength and ductility enhancement is examined, as shown in Figures 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 proposed 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 after the intersection point of flexural and shear strengths.

CONCLUSIONS

An analytical method is presented taking into account the interaction of flexural and shear strength/deformation models. The conclusions of this study can be summarized as follows:
(1) The presented method can successfully predict the load-deformation responses of RC columns with various jacketing materials: steel plate, concrete and FRP materials. FRP materials include both conventional FRP and large rupturing strain (LRS) FRP. The method is also applicable to both rectangular and elliptical cross-sections of steel jacket.

(2) The flexural strength/deformation model connects with the shear strength/deformation model through the reduced neutral axis depth and tension shift in the shear crack region, whereas the shear strength/deformation model connects with the flexural strength/deformation model through the yielding of tension reinforcement and concrete strength degradation in compression softening.

(3) The good agreement with experimental data implies that the potential shear strength concept in the presented method can represent post-peak behavior in load-deformation response.

(4) The presented method can prove that LRS FRP materials provide better load-deformation characteristics.

REFERENCES


