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Plastic hinge analysis of FRP confined circular concrete columns

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ABSTRACT

Experimental tests have identified that FRP confinement affects the plastic hinge length of reinforced concrete columns. Some tests found that the confinement increased the plastic hinge length; whereas others showed otherwise. The plastic hinge length as well as the drift capacity of FRP confined circular concrete columns are studied in this work through a combination of numerical simulation, experimental study, and mechanism analysis. Data regressions are employed to formulate the plastic hinge length, which is found to be closely related to the confinement ratio of FRP. The obtained plastic hinge model shows that FRP confinement increases the plastic hinge length at low confinement ratio; however, it has an opposite effect when the confinement ratio is high. The accuracy of the proposed model is verified by test results from 29 large-scale FRP-confined circular concrete columns.

1. Introduction

Deformation capacity is critical for structures expected to withstand seismic activity and must be carefully designed to satisfy explicit deformation demands in performance-based design [1]. Reinforced concrete (RC) columns are often required to undergo a large number of inelastic deformation cycles for a design earthquake while maintaining a certain strength to ensure the stability of the structure. A sufficient deformation capacity for RC columns can usually be achieved by providing adequate confining reinforcement at a potential plastic hinge region [2,3]. The deformation-based approach to the design of confining steel for RC columns has been extensively investigated [4], and is accepted by many current design codes. In recent years, fiber-reinforced polymer (FRP) jackets have become popular in providing confinement to RC columns [5]; however, extensive analytical modeling of the plastic hinge deformation capacity of FRP-confined RC columns under seismic load is limited [6]. Although nonlinear numerical simulation can be used for calculating the deformation capacity of FRP-confined RC columns [7,8], a simple design-based procedure is necessary for engineering use.

Curvature capacity at the cross-sectional level and drift capacity at the member level are often used as criteria for evaluating the deformation capacity of columns. A procedure for calculating the curvature capacity of FRP-confined columns has been proposed [9,10], but studies calculating deformation capacity at the member level are scarce in the open literature. Calculation of that deformation capacity requires a model for the plastic hinge length, which can be combined with the model for ultimate curvature to give the drift capacity of the member [11,12].

The literature reveals contradictory conclusions about the plastic hinge length of FRP-confined columns. One suggestion is that the plastic hinge length of an FRP-confined RC column is smaller than that of a normal RC column [13]. This conclusion derives from the test observation of steel jacketed RC columns in which the jacket restricted the spread of plastic yielding [14]. Other researchers have accepted this position [15]. However experimental tests have also shown that the plastic hinge lengths of most FRP-confined columns are larger than those of normal RC columns [16]. Some researchers have proposed that to ensure simplicity, the plastic hinge length of an FRP-confined column be considered equal to that of a normal RC column [10,17]. Given these findings, it is clear that the plastic hinge length of FRP-confined RC columns needs further investigation.

The parameters that affect the ultimate curvature and plastic hinge length of FRP-confined circular concrete columns are extensively studied in this paper, and models are proposed to predict the ultimate drift ratio of those columns.

2. Deformation relationships

Using the well-known plastic hinge concept (Fig. 1), Park and Paulay [3] proposed an expression for the ultimate displacement, Δ_u, at the tip of a cantilever column

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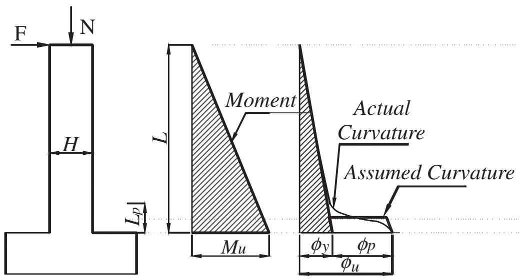

Fig. 1. Plastic-hinge analysis.

$$\Delta_u = \Delta_y + \Delta_p = \frac{\phi_y L^2}{3} + (\phi_u - \phi_y) l_p (L - 0.5 l_p), \tag{1}$$

where Δ_y is the yield displacement, Δ_p is the plastic displacement, ϕ_u is the ultimate curvature at the column base, ϕ_y is the yield curvature, and L and l_p are the lengths of the cantilever column and the plastic hinge, respectively. The ultimate drift ratio, θ_u, can be expressed as:

$$\theta_u = \frac{\Delta_u}{L} = \frac{\phi_y L}{3} + \frac{(\phi_u - \phi_y) l_p (L - 0.5 l_p)}{L}. \tag{2}$$

The yield curvature of a reinforced concrete column can be given as [13,18]:

$$\phi_y = \lambda \frac{\varepsilon_y}{D}, \tag{3}$$

where $\lambda = 2.45$ for spiral-reinforced columns, ε_y is the yield strain of the longitudinal reinforcement, and D is the diameter of the column. The confinement of FRP does not significantly affect the column yield state, and thus the equation is also applicable to FRP-confined columns [10]. In Eq. (2), ϕ_u and l_p are the two parameters that need to be determined.

3. Ultimate curvature

The ultimate curvature ϕ_u can be expressed as

$$\phi_u = \varepsilon_{cu}/c, \tag{4}$$

where c is the depth of the neutral axis (or the depth of the compression zone) when the strain at the extreme compressive fiber reaches the ultimate strain, ε_{cu}. The neutral axis depth c of RC sections has been comprehensively studied in the literature. Although many factors such as axial load, longitudinal steel and transverse reinforcement can affect the value of c in general, the following simplified equation has been adopted by many researchers for evaluation of c [19,20]:

$$\frac{D}{c} = \frac{\phi_c D}{\varepsilon_c} = \frac{G_0}{1 + G_1 n} + \frac{\phi_y D}{\varepsilon_c}, \tag{5}$$

where ϕ_c is the curvature at a given extreme compression fiber strain ε_c, n is the axial load ratio (defined as $n = N/A_g f'_c$, where A_g is the gross cross-sectional area, f'_c is the concrete compressive strength, and N is the axial force), and G_0 and G_1 are two parameters depending on the value of ε_c. By substituting Eq. (3) into Eq. (5), the following equation is obtained for circular columns:

$$\frac{D}{c} = \frac{G_0}{1 + G_1 n} + 2.45 \frac{\varepsilon_y}{\varepsilon_c}. \tag{6}$$

From the moment–curvature analyses of flexure-dominant columns, G_0 and G_1 are calculated to be 5.3 and 9.4, respectively, for $\varepsilon_c = 0.004$ [19,20]. Similar expressions have been adopted by others to calculate c/D at $\varepsilon_c = 0.004$, such Eq. (7a) by Kowalsky [18] and Eq. (7b) by Jiang et al. [21]:

$$\frac{c}{D} = \frac{1}{3.8 - 5.0n}. \tag{7a}$$

$$\frac{c}{D} = 0.19 + 0.9n. \tag{7b}$$

For RC columns with a significant confinement by steel stirrups, the ultimate failure strain is greater than 0.004 and closely related to the degree of confinement. Therefore, apart from the parameter n in the above equations, another parameter involving confinement should be included in the model. From moment–curvature analyses, the following equation was proposed by Jiang et al. [21] for calculation of the compression zone depth at ultimate failure:

$$\frac{c}{D} = 0.12 + (1.07 - 1.05 K_e \omega_s) \times n, \tag{8}$$

where ω_s is the confinement ratio of transverse steel, and K_e is the shape factor related to the effective area of confinement.

In this work, the same approach is adopted to derive a model similar to Eq. (8) for the calculation of the compression zone depth of FRP-confined circular columns. As the confinement provided by FRP is different from that by steel stirrups, and also the concrete cover does not spall in FRP confined columns, the details of the model will be different from those for RC columns.

3.1. Numerical simulations

In this section a numerical procedure using the conventional layered method [18–22] is adopted to study the moment–curvature relationships of FRP-confined circular sections. The moment–curvature relationship is generally insensitive to the constitutive model for confined concrete, and the change in the yield curvature and ultimate moment is insignificant when different constitutive models are used [23,24]. The constitutive model for FRP-confined concrete proposed by Lam and Teng [25] is adopted for the numerical simulation. No tensile strength of the concrete is considered. The Giuffre–Menegotto–Pinto model is used as the stress–strain relationship for the longitudinal steel bars [26].

3.2. Numerical simulation parameters

A typical bridge pier with a diameter of one meter, a clear cover of 40 mm, and a cylinder compressive strength f'_c of 30 MPa is used for the analysis. The yield stress of the longitudinal reinforcement, f_y, is 420 MPa. The axial load ratio n varies from 0.1 to 0.4 in increments of 0.1. The longitudinal reinforcement ratio ρ_s (defined as $\rho_s = A_s/A_g$, where A_s is the area of longitudinal reinforcement) changes from 1% to 4% in 1% increments. The confinement ratio, λ_f (defined as $\lambda_f = f_l/f'_c = 2E_f t_f \varepsilon_f / D f'_c$, where f_l is the lateral confining pressure exerted by the FRP, E_f is the modulus of the FRP, t_f is the thickness of the FRP jacket, and ε_f is the FRP rupture strain from flat coupon test), varies between 0.1 and 0.3 in increments of 0.1. Two types of FRP are considered in the numerical simulation: carbon FRP (CFRP) and glass FRP (GFRP). The ultimate tensile strength and rupture strain of the CFRP are 3500 MPa and 0.013, respectively, while those for the GFRP are 1380 MPa and 0.023, respectively. As non-dimensional parameters are adopted in the analyses, the conclusions derived are more general.

3.3. Numerical results

The ratio of the neutral axis depth to the sectional diameter, c/D, is plotted versus the extreme compressive fiber strain for one group of CFRP confined sections in Fig. 2. Clearly, the compression zone depth approaches a stable value when ε_c is greater than 0.004. As the ultimate strain of FRP confined columns is generally much greater than 0.004, the calculation of compression zone depth is insensitive to the ultimate strain. Therefore, the compression zone depth is