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Numerical modeling of externally prestressed steel–concrete composite beams

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ABSTRACT

A finite element model to analyze externally prestressed steel–concrete composite beams under short and long-term loads is developed. The nonlinear geometric effect is considered by introducing flexural and axial interaction in the finite element formulation and by updating the eccentricities of external tendons in the numerical procedure. A layered technique is employed to describe varied material properties across the composite section. The time-dependent effects are also introduced in the model. External prestressing is considered to contribute to equivalent nodal loads. The analysis is able to simulate the short-term behavior of externally prestressed composite beams at all ranges of loading up to failure and also to model the long-term behavior of these beams at service loads. The proposed model is validated by comparisons with available experimental data as well as other analysis results. Typical short and long-term responses of steel–concrete composite beams with and without external prestressing are evaluated.

1. Introduction

External tendons have been widely employed to strengthen or erect steel–concrete composite beams. Composite beams with external prestressing possess many attractive merits such as excellent crack resistance, low service-load deflection, high ultimate flexural capacity and favorable fatigue behavior [1]. There are two features that bring difficulties in the analysis of externally prestressed steel–concrete composite beams. Firstly, because of the unbonded nature of external tendons, the increase in tendon strain is dependent on the deformation of the whole member and, therefore, the tendon stress cannot be determined by a single section analysis. Secondly, the effective tendon depths would change when a member deflects, except at deviator and anchorage points, causing what is termed as second-order effects [2].

A number of works were reported on nonlinear analysis of externally prestressed steel–concrete composite beams. Ayyub et al. [3] used a strain compatibility method based on the force equilibrium and deformation compatibility to determine the complete response of externally prestressed composite beams. By using an empirical equation for computing the tendon stress, Zhang and Fu [4] proposed an analytical method to determine the ultimate moment resistance of externally prestressed composite box beams. Based on the observation that the strain and curvature distributions at ultimate would not vary substantially under given conditions, Zona et al. [5] developed a simplified approach for evaluating the moment capacity of externally prestressed composite beams. General numerical methods of analysis have also been available in the literature. Refs. [6,7] presented a set of theoretical works on the development of finite element models for steel–concrete composite beams with external prestressing. Some investigators [8,9] carried out finite element studies on the behavior of externally prestressed composite beams using the software packages such as ABAQUS and ANSYS.

Although some analytical and numerical models have been reported, most of the available models dealt only with the analysis of composite beams under short-term loads, neglecting the time-dependent effects which are recognized to be critically important in practice. In a previous study by the authors [10], the time-dependent effects were introduced in a finite element model for internally unbonded prestressed concrete beams. More recently, the authors [11] outlined a numerical procedure for externally prestressed concrete beams, taking into account second-order interaction with time-dependent effects. The above-mentioned time-dependent models [10,11], however, were limited to reinforced concrete beams with external or internal unbonded prestressing. These models are extended in this study to the short-term and long-term analysis of steel–concrete composite beams prestressed with external tendons. Numerical examples are given to illustrate the reliability and applicability of the proposed model.

2. Material stress–strain curves and concrete creep

2.1. Stress–strain curves of materials

The stress–strain curve of concrete in short-term uniaxial compression consists of parabolic ascending and linear descending portions [12],

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while the stress–strain curve of concrete in tension consists of elastic and linear tension-stiffening portions, as illustrated in Fig. 1(a), where σ_c and ε_c^m = concrete stress and mechanical strain, respectively; $f_c^{'}$ = concrete cylinder compressive strength; $\varepsilon_u = 0.0033$; f_t = concrete tensile strength; ε_{t0} = strain at the end of tension-stiffening, taken as 10 times the cracking strain.

The stress–strain equation for prestressing steel proposed by Menegotto and Pinto [13] is used here. The stress–strain curve is shown in Fig. 1(b), where σ_p and ε_p = stress and mechanical strain of prestressing steel, respectively; E_p and f_{py} = elastic modulus and yield stress of prestressing steel, respectively; and K, Q and R = empirical coefficients. For Grade 270, 7-wire strands as used in the experimental beams analyzed in the present study, the values of K, Q and R are 1.0618, 0.01174 and 7.344, respectively.

The nonprestressed steel in tension and compression is linear elastic up to yielding, followed by linear strain-hardening, as illustrated in Fig. 1(c), in which σ_s and ε_s = stress and mechanical strain of nonprestressed steel, respectively; and f_y = yield strength. The modulus of strain-hardening E_{sh} is assumed to be 1.5% of the steel modulus of elasticity E_s.

2.2. Creep of concrete

The present analysis assumes that the concrete nonmechanical stain consists of the shrinkage strain ε_c^{sh} and the creep strain ε_c^{cr}. Concrete creep is associated with stress history. At service loads, a linear creep law may be applied and, therefore, creep of concrete may be expressed as follows

$$\varepsilon_c^{cr}(t) = \sigma_c(t_0)C(t,t_0) + \int_{t_0}^{t} C(t,\tau)\frac{\partial\sigma_c(\tau)}{\partial\tau}d\tau \tag{1}$$

where $\sigma_c(t_0)$ = initial stress applied at time t_0 and $\sigma_c(\tau)$ = stress applied at time τ; $C(t,\tau)$ = creep function, expressed here by [14]

$$C(t,\tau) = \sum_{k=1}^{m}\phi_k(\tau)\left[1-e^{-r_k(t-\tau)}\right] \tag{2}$$

in which m, $\phi_k(\tau)$ and r_k = creep coefficients.

Denote by Δt_n the time interval from time t_{n-1} to t_n. The creep strain increment $\Delta\varepsilon_c^{cr}$ at Δt_n is given by [15]

$$\Delta\varepsilon_c^{cr} = \varepsilon_c^{cr}(t_n) - \varepsilon_c^{cr}(t_{n-1}) = \eta_n + C(t_n, t_{n-1/2})\Delta\sigma_n \tag{3}$$

$$\eta_n = \sum_{k=1}^{m}(1-e^{-r_k\Delta t_n})\omega_{kn} \tag{4}$$

$$\omega_{kn} = \omega_{k(n-1)}e^{-r_k\Delta t_{n-1}} + \Delta\sigma_{n-1}\phi_k(t_{(n-1)-1/2})e^{-r_k\Delta t_{n-1}/2} \tag{5a}$$

$$\omega_{k1} = \sigma_c(t_0)\phi_k(t_0) \tag{5b}$$

where $t_{n-1/2}$ = intermediate time between t_{n-1} and t_n; $\Delta\sigma_n$ = stress increment at time interval Δt_n; Eqs. (5a) and (5b) are recursive formula by which the concrete creep can be effectively determined by storing the value of $\omega_{k(n-1)}$ only, instead of recording the entire stress history.

3. Finite element method

The finite element method is formulated based on the following simplified assumptions: (1) a beam element is divided into discrete layers so as to describe varied material properties across the depth of a composite section; (2) plane sections remain plane after deformation; (3) the relative slip between steel beam and concrete slab, as well as between reinforcing steel and surrounding concrete, is negligible; (4) the effect of shear deformation is negligible (i.e., Euler–Bernoulli beam theory is applied). A plane beam element as shown in Fig. 2 is considered here. The local coordinate system (x, y), in which the element properties are to be described, is defined by the two end nodes of the beam element. There are three degrees of freedom for each node: x, y-displacements u, v and rotation θ. Assume there are linear and cubic variations of u and v with respect to x, respectively. The x and y-displacements are related to the element nodal displacements by

$$u = N_1u_i + N_4u_j \tag{6a}$$

$$v = N_2v_i + N_3\theta_i + N_5v_j + N_6\theta_j \tag{6b}$$

where $N_1 = 1-\xi$; $N_2 = 1-3\xi^2+2\xi^3$; $N_3 = l(\xi-2\xi^2+\xi^3)$; $N_4 = \xi$; $N_5 = 3\xi^2-2\xi^3$; $N_6 = l(-\xi^2+\xi^3)$; $\xi = x/l$ in which l = element length.

The incremental strain–displacement relationship for an element can be expressed as follows

$$\Delta\varepsilon = \left(\boldsymbol{B} + \Delta\boldsymbol{u}^{eT}\boldsymbol{J}^T\boldsymbol{J}/2\right)\Delta\boldsymbol{u}^e \tag{7}$$

$$\boldsymbol{B} = \left[N_1^{'} \ \ -N_2^{''}y \ \ -N_3^{''}y \ \ N_4^{'} \ \ -N_5^{''}y \ \ -N_6^{''}y\right] \tag{8}$$

$$\boldsymbol{J} = \left[0 \ \ N_2^{'} \ \ N_3^{'} \ \ 0 \ \ N_5^{'} \ \ N_6^{'}\right] \tag{9}$$

where $\boldsymbol{u}^e$ = element nodal displacements written as $\{u_i, v_i, \theta_i, u_j, v_j, \theta_j\}^T$.

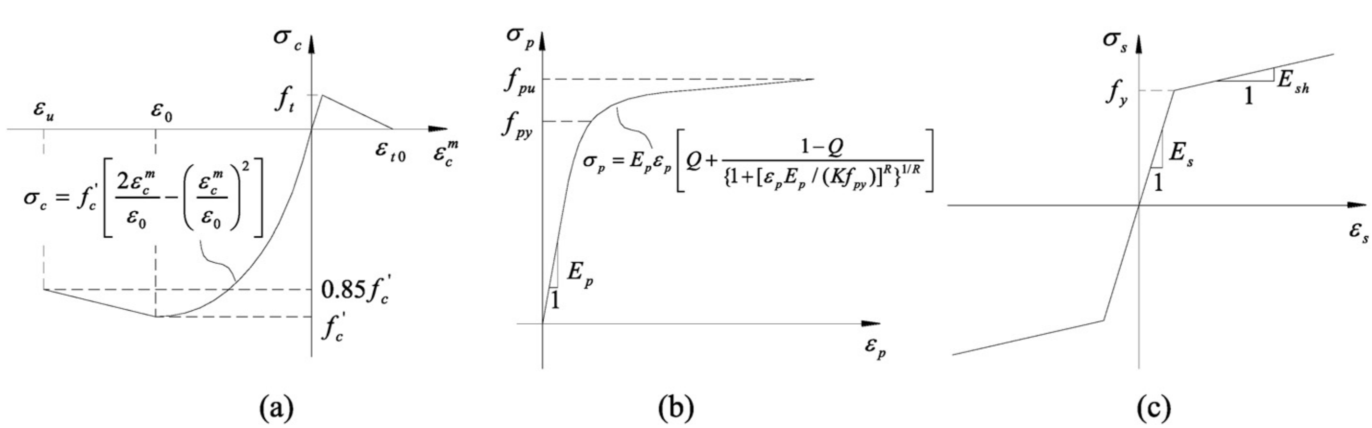

Fig. 1. Material stress–strain curves. (a) Concrete; (b) prestressing steel; (c) nonprestressed steel.