Intelligent control on long-run deflection for prestressed concrete bridges

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ABSTRACT: An approach to deduce characteristics of concrete creep and shrinkage from short-term test in pre-stressed concrete bridges during construction is present, and when the automatically step-up method and finite element method to analyze time-dependent effect are introduced, a novel model of intelligent control of long-run deflection for pre-stressed concrete bridges is put forward. The model includes five sub-models, dada collection, deduction of characteristics for creep and shrinkage , time-dependent structural analysis, solution of needed force to compensate surplus deflection and activating force system, which bear the capacity to accumulate time-dependent data on deflections and strains of concrete, to revise the input computing parameters to improve the predicting precision, to adjust the structural deflection automatically by tensioning the intelligent tendons if the real deflection is out of limitation. This prediction and control system considers the effect of sectional characteristics, steel ratio, steel distribution, relaxation of tendons and time-dependent modulus of concrete. The theoretical analysis shows that the model is feasible, holding practical meaning for future smart bridge.

1 INTRODUCTION

Creep and shrinkage are natural properties of concrete which lead to time-dependent redistribution of stresses between concrete and steel in section in prestressed concrete bridges, thus to time-dependent deflections. In hyperstatic continuous girder bridges, redistribution of structural force occurrs simultaneously which also produces time-dependent deflections. In the past years the mechanics property of concrete is improved tremendously due to adopting newly developed materials as concrete components and renewing mixture design, and the strength of reinforced materials such as steel and carbon fiber increases too, all of which make possible to construct larger span with smaller height, in which how to predict precisely and control assuringly the long-run deflection becomes one of the most important key techniques for needed innovative structural design. The precision in predicting time-dependent effect in prestressed concrete bridges lies mainly in four aspects, precise model to describe properties of creep and shrinkage, precise structural analysis without considering creep and shrinkage, precise time-dependent structural analysis and reasonable construction.

Theories on mechanism of creep and shrinkage (Bažant et al. 1979,1989,1995, Cervera et al. 1999, Wittmann 1982) have been put forward, but even the most precise model B3 still holds large scale of uncertainty factor, as Bazant(1995) pointed out that B3 has to be considered uncertainty factors 23% for creep and 34% for shrinkage in design respectively, for too many varying aspects contributes to creep and shrinkage. For specific deflection-controlled structure, to some extent, B3 is not enough, so engineers turn to obtain properties of creep and shrinkage of plain concrete adopted in real structure by testing specimens in laboratory. As researches show, in some cases, the deduced characteristics of creep and shrinkage from indoors experimental results may not fit well with real values in working ambient environment. Thus, we face the problem how to derive authentic expressions for creep and shrinkage, the base to predict precisely and control time-dependent deflection of structure.

In most cases, the result of structural analysis without considering creep and shrinkage is reliable if the adopted parameters are reasonable, while the precision of time-dependent structural analysis is more difficult to guarantee. Numerous approaches on time-dependent structural analysis have been developed, which can be categoried into two groups, one is step-up approach based on finite element method (Bažant 1972, Bradford et al.1999, Buragohain et al.1997, Hu & Chen 2004), the other is simplified approach based on steel restraint coefficient method (Bandyopadhyay &Sengupta 1999, Branson 977,Gilbert 1999, Hu et al. 2004), and the former is always preferred for precise analysis. Nowadays the commonly used step-up approach treats the effect due to creep and shrinkage obtained from last step as initial strain in up-date equations, in which creep and shrinkage effect cant not be separated from unified equations, this will lead to be difficult to ascertain the fault or unacceptable discrepancy coming whether from elastic analysis or from time-dependent analysis, for the parameters such as concrete modulus and existed stress in tendons used in elastic analysis are time-dependent variables. Hu & Chen (2004) established automatically step-up method (ASUM), in which the control of calculating precision is fulfilled through precision control of finite element method(FEM) for elastic analysis and ASUM for time-dependent analysis, respectively. ASUM bears the capacity to achieve anticipated precision that can be used in intelligent system.

Construction also has serious impact on long-run deflection control, for creep behavior is affected by loading age and load duration relative to the time of tensioning individual steel group, the time the first-term dead load (self weight) begins to function and the time to apply second-term dead load (deck system and handrail). The most important is that the properties of creep and shrinkage vary with concrete components and curing condition, all of which correspond to construction in site. To some extent, uncertainty in construction is inevitable and it's difficult to estimate in structural analysis.

As discussion above, we face the problems in long-term deflection control, how to derive authentic expressions of creep and shrinkage of plain concrete adopted in real structure working in site, how to guarantee the precision of time-dependent structural analysis and how to consider the uncertainty effect of construction. On one hand, they are difficult to deal with; on the other hand, we still face how to adjust deflection if it surpasses the limited value occurring possibly even though above problems are disposed theoretically. In this paper, the main purpose is to present concept of an intelligent system on controlling long-run deflection for prestressed concrete bridges, and to discuss the structure and key techniques.

2 STRUCTURE OF INTELLIGENT CONTROL ON LONG-RUN DEFLECTION

The model of intelligent control on long-run deflection includes five sub-models, data collection, deduction of authentic expressions of creep and shrinkage of plain concrete, time-dependent structural analysis, solution of the needed force to adjust surplus deflection, and applying force. The structure of intelligent system can be described in Figure 1.

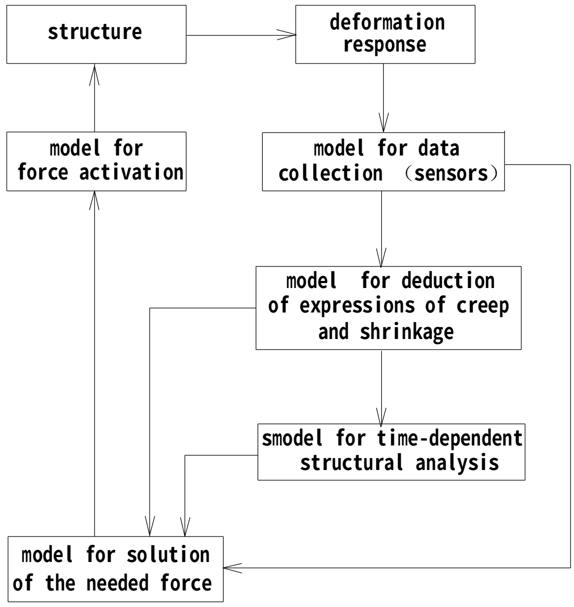

Figure 1. Schematic diagram of structure of intelligent control on long-run deflection

Data collection system accumulates data on time-dependent deflection of structure and strain of concrete. At least six observation points are needed along a span to observe the deflection, which two points are located at mid-span and two points at each end of span, these two points are parted at two sides of the girder in the symmetric location at the same section. At least ten strain sensors are needed to observe the strain of concrete at five sections located at two end of span, mid-span, one-fourth span and three-fourth span. Two sensors at a section, one is located near the top fiber while the other near the bottom fiber. A temperature sensor is needed near each strain sensor to delete the effect of temperature from observed data.

There are relationship between observed total strains and strains due to creep and shrinkage in real structure in site, so the followed model is to deduce authentic expressions of creep and shrinkage of plain concrete from results of short-term observation (Hu & Chen,2003).The deduced expressions are fundamental for precision-controllable time-dependent structural analysis to predict long-run deflection. Even the analysis model bears the capacity to adjust computational precision by further dividing time interval, the theoretical values with large discrepancy from observed data still possibly occur for too many factors affecting structure's time-dependent behavior beyond control and pre-ponderation, so the analysis model has to amend it's computing parameters to make theoretical values fit well with observed data. If the predicted long-run deflection based on revised analysis model surpasses the limitation, the needed force to adjust surplus deflection should be solved, which is the function of fourth model. The last model is to apply solved force on structure.

3 MODEL FOR DEDUCTION OF AUTHENTIC EXPRESSIONS OF SHRINKAGE AND CREEP

3.1 *Authentic expression of shrinkage*

Creep and shrinkage function simultaneously in structures, and shrinkage is a behavior without load, so the authentic expression of shrinkage can be derived firstly when the structure is in load-free condition (Hu & Chen 2003). A concrete beam always stands over the formwork before bearing load after casting, and the effect of shrinkage and temperature is not enough to product camber, therefore only the longitudinal deformation due to shrinkage and temperature occurs (transverse deformation can be neglected)

$$e_c(t,t) = e_{sh}(t,t) + a_e \cdot \Delta T(t,t) \tag{1}$$

where $e_c(t,t)$ = strain increment from time t to t, which can be observed from sensors ; $e_{sh}(t,t)$ = shrinkage strain; $a_e = \dfrac{a_s + m_s n_s a_c}{1 + m_s n_s}$ = equivalent thermal coefficient of temperature for whole section; $n_s = E_s / E_c(t)$; $m_s = A_s / A_c$; A_s = area of non-prestressed steel; A_c = net cross area of concrete; E_s = modulus of non-prestressed steel; $E_c(t)$ = modulus of concrete at time t.

Solution of compatibility and equilibrium gives

$$e_{sh}(t,t) = e_{sh,c}(t,t)\frac{m_s}{m_s + n_s\left[1 - e_{sh,c}(t,t)\right]} \tag{2}$$

where $e_{sh,c}(t,t)$ = shrinkage strain of plain concrete.

Setting the discrepancy x_s of shrinkage between experimental result and theoretical value given by CEB-FIP Model Code 1990, the shrinkage expression turns to

$$e_{sh,c}(t,t_s) = x_s e_{sh,\infty} b_s(t,t_s) \tag{3}$$

where $e_{sh,\infty}$ = nominal shrinkage; $b_s(t,t_s)$ = developing function of shrinkage; t_s = the time for concrete to begin to shrinkage.

Solution of Eq.1~ Eq.3 works out

$$x_s = \frac{n_s + m_s}{e_{sh,\infty}\left[b_s(t,t_s) - b_s(t,t_s)\right]} \cdot \frac{e_c(t,t) - a_e \cdot \Delta T(t,t)}{m_s + n_s\left[e_c(t,t) - a_e \cdot \Delta T(t,t)\right]} \tag{4}$$

Different x_s obtained at different time t due to observed $e_c(t,t)$ and theoretical data. Letting $\overline{x_s}$ the expected value of regression analysis, authentic expression of shrinkage can be derived as follow

$$e_{sh,c}(t,t_s) = \overline{x_s} e_{sh,\infty} b_s(t,t_s) \tag{5}$$

3.2 *Authentic expression of creep*

For a static prestressed concrete bridge, strain increment of concrete in section relates to the creep of plain concrete, so the creep coefficient can be given as

$$j(t,t_0) = \left[\Delta e(t,t_0)(1 + r\frac{m_s E_s + m_p E_p}{E_c}) - e_{sh.c}(t,t_0) - a_e \cdot \Delta T(t,t_0) + m_p r\, l_r \frac{s_r(t,t_0)}{E_c}\right]$$
$$\div \left[e_e(t_0) - r c(t,t_0) l_r m_p s_r(t,t_0) + r c(t,t_0)(m_s E_s + m_p E_p)\Delta e(t,t_0)\right] \tag{6}$$

where t_0 = the time the stress is applied ; $e_e(t_0)$ = instantaneous elastic strain of concrete ; e = distance from centroid of total steel area to centroid of concrete; $m_p = A_p / A_c$; r = radius of gyration of concrete section; $r = (1 + \dfrac{e^2}{r^2})$; e = distance from centroid of total steel to centroid of concrete; $c(t,t_0)$ = aging coefficient of concrete; $s_r(t,t_0)$ = intrinsic relaxation of the prestress steel; l_r = reduced coefficient of intrinsic relaxation considering the effect of creep and shrinkage. The scope of $c(t,t_0)$ varies from 0.6 to 0.9, 0.82 is commonly preferred, or can be calculated by equations (Lacidogna & Tarantino 1996). $s_r(t,t_0)$ and l_r can be calculated by equations (Ghali &Trevino 1985).

In Eq.6, $\Delta e(t,t_0)$ and $e_e(t_0)$ can be read from sensors, $e_{sh,c}(t,t_0)$ is provided by Eq.5. Setting the discrepancy x_{cr} of creep between experimental result and theoretical value given by CEB-FIP Model

Code 1990, the expression of creep coefficient turns to

$$j(t,t_0) = x_{cr} j(\infty,t_0) \cdot b_c(t,t_0) \quad (7)$$

where $j(\infty,t_0)$ = nominal creep; $b_c(t,t_0)$ = developing function of creep.

Thus we can obtain authentic expression of creep coefficient as follows

$$j(t,t_0) = \overline{x_{cr}} j(\infty,t_0) \cdot b_c(t,t_0) \quad (8)$$

where $\overline{x_{cr}}$ is the expected value of regression analysis.

4 MODEL FOR PRECISION-CONTROLLABLE TIME-DEPENDENT STRUCTURAL ANALYSIS

This model covers three functions, predicting time-dependent deformation (deflections and strains) of prestressed concrete bridge adopting derived authentic expressions of creep coefficient and shrinkage, adjusting computational model by comparing tested data and theoretical values, and predicting long-run deflection caused by applied force for compensating surplus deflection based on revised time-dependent structural analysis model.

Here to introduce briefly the precision-controllable time-dependent structural analysis based on automatically step-up method (ASUM) and finite element method (FEM), which includes four steps:

Step1—Compute instantaneous elastic responses of bridge at loading age t_0 by FEM, in which the built-up beam element composed of concrete, prestressed steel and non-prestressed steel is adopted, and the equivalent stiffness considers contribution of concrete, prestressed steel and non-prestressed steel, including the effect of time-dependent modulus of concrete and incessantly tensioning steels. The equivalent stiffness at time t_0 is

$$[K_{b,0}]^e = [K_c]_0^e + [K_s]^e + [K_p]^e \quad (9)$$

and at any time turns to

$$[K_{b,t}]^e = [K_{b,0}]^e + \frac{E_c(t) - E_c(t_0)}{E_c(t_0)} [K_c]_0^e \quad (10)$$

where the subscripts "b"、"c"、"s"、"p" represent compound element, concrete element, prestressed steel element and non-prestressed steel element.

The elastic response includes $e_c^e(t_0)$, $s_c^e(t_0)$, curvature $f^e(t_0)$ and deflection $f^e(t_0)$ at any element nods, where the stress and strain are the values at the centroid of concrete.

Step2—Use ASUM to compute the hypothetical strain increment Δe_c and curvature increment Δf due to creep and shrinkage in the period $t_0 \rightarrow t$ under constant force N_0^e 、 M_0^e without restraint.

The artificial stress needed to prevent occurrence of Δe_c is given as

$$\Delta s_c = -E_c(t)\Delta e_c \quad (11)$$

Step3—Compute the elastic response $\Delta\Delta e_c$, $\Delta\Delta s_c$, $\Delta\Delta f$ and $\Delta\Delta f$ caused by artificial stress at every element nods by FEM, in which the stiffness is given by Eq.10.

Step4—Calculate the total response including elastic value and time-dependent variation at time t, which is given as

$$e_c(t) = e_c^e(t_0) + \Delta\Delta e \quad (12)$$

$$s_c(t) = s_c^e(t_0) + \Delta s + \Delta\Delta s \quad (13)$$

$$f(t) = f^e(t_0) + \Delta\Delta f \quad (14)$$

$$f(t) = f^e(t_0) + \Delta\Delta f \quad (15)$$

In above computation the FEM is only used to calculate elastic responses which the time-related influences are not considered, while the ASUM is only used to calculate time-dependent effect due to creep and shrinkage for structures under free-constraint, thus the control of calculating precision is fulfilled through precision control of FEM and ASUM, respectively.

5 MODEL FOR SOLUTION OF NEEDED FORCE TO ADJUST DEFLECTION

In real bridges it's possible to occur that the tested real deflection surpasses the predicted time-dependent value resulting in long-run deflection over the limited value, in this case we have to adjust the internal force to delete or compensate the surplus. For any structures the mid-span deflection can be calculated through curvatures at different sections, so adjusting deflection can be achieved by adjusting curvatures. Assuming the needed force N_k, M_k at a specific section at time t_k can generate long-run curvature f_∞, setting elastic strain $e_c(t_k)$ at reference point O (centroid of concrete) and elastic sectional curvature $f(t_k)$ caused by N_k, M_k, thus we have

$$\int E_c(t_k) e_c(t_k) dA + \int E_c(t_k) f(t_k) y dA$$
$$+ E_s \sum_{i=1}^{ns} A_{si} \left[e_c(t_k) + e_{si} f(t_k) \right]$$
$$+ E_p \sum_{i=1}^{ns} A_{pi} \cos q_{pi} \left[e_c(t_k) + e_{pi} f(t_k) \right] = N_k \quad (16a)$$

$$\int E_c(t_k)e_c(t_k)ydA+\int E_c(t_k)f(t_k)y^2dA$$
$$+E_s\sum_{i=1}^{ns}A_{si}e_{si}\left[e_c(t_k)+e_{si}f(t_k)\right]$$
$$+E_p\sum_{i=1}^{ns}A_{pi}e_{pi}\cos q_{pi}\left[e_c(t_k)+e_{pi}f(t_k)\right]=M_k \qquad (16b)$$

Which can be rewritten as

$$\begin{Bmatrix}N_k\\M_k\end{Bmatrix}=\begin{bmatrix}\mathrm{H}_1 & \mathrm{H}_2\\ \mathrm{H}_2 & \mathrm{H}_3\end{bmatrix}\begin{Bmatrix}e_c(t_k)\\ f(t_k)\end{Bmatrix} \qquad (17)$$

or

$$\begin{Bmatrix}e_c(t_k)\\ f(t_k)\end{Bmatrix}=\begin{bmatrix}\mathrm{H}_1 & \mathrm{H}_2\\ \mathrm{H}_2 & \mathrm{H}_3\end{bmatrix}^{-1}\begin{Bmatrix}N_k\\M_k\end{Bmatrix} \qquad (17')$$

where $\mathrm{H}_1=E_c(t_k)A_c+aA_c$; $\mathrm{H}_2=E_c(t_k)S_c+bA_c$; $\mathrm{H}_3=E_c(t_k)I_c+cA_c$; $A_c=\int dA$; $S_c=\int ydA$;

$I_c=\int y^2dA$; $a=E_s\sum_{i=1}^{ns}m_{si}+E_p\sum_{i=1}^{np}m_{pi}\cos q_{pi}$;

$b=E_s\sum_{i=1}^{ns}m_{si}e_{si}+E_p\sum_{i=1}^{np}m_{pi}e_{pi}\cos q_{pi}$;

$c=E_s\sum_{i=1}^{ns}m_{si}e_{si}^2+E_p\sum_{i=1}^{np}m_{pi}e_{pi}^2\cos q_{pi}$; ns = the total number of layers of non-prestressed steel; np = the total number of bundles of prestressed steel; m = steel ratio; q_{pi} is the acute angle between tangent line of prestressed steel and beam axis.

The relationship between strain and stress of concrete considering creep and shrinkage is given as

$$e_c(t)=\frac{s_c(t_0)}{E_c(t_0)}\left[1+j(t,t_0)\right]+\int_{t_o}^{t}\frac{1+j(t,t)}{E_c(t)}ds_c(t)+e_{sh}(t,t_0)$$
$$=s_c(t)J(t,t)-\int_{t_o}^{t}s_c(t)dJ(t,t)+e_{sh}(t,t_0) \qquad (18)$$

where $J(t,t)=\dfrac{1+j(t,t)}{E_c(t)}$ = creep flexibility.

Dividing duration (t_0,t) into small time interval （t_{i-1}，t_i）（$i=1,2,...n$）, assuming the stresses of concrete vary linearly during（t_{i-1}，t_i）, setting $s_c(t_i)=s_i$, $e_c(t_i)=e_i$, $e_{sh}(t,t_0)=e_{sh,n}$, $J(t,t_i)=J_{n,i}$, Eq.18 can be rewritten in the form

$$e_n=\frac{1}{2}s_n(J_{n,n}+J_{n,n-1})+\frac{1}{2}s_0(J_{n,0}-J_{n,1})$$
$$\frac{1}{2}\sum_{i=1}^{n-1}(J_{n,i-1}-J_{n,i+1})s_i+e_{sh,n} \qquad (19)$$

Above equation can be written as following form

$$[A]\{s\}+s_0\{B\}+\{e_{sh}\}=\{e\} \qquad (20)$$

where s_0 = initial stress of concrete at computational point; $a_{ii}=\frac{1}{2}(J_{i,i}+J_{i,i-1})$; $a_{ij}=\frac{1}{2}(J_{i,j-1}-J_{i,j+1})$; $b_i=\frac{1}{2}(J_{i,0}-J_{i,1})$; $(i=1,2,\mathrm{L\,L},n;\quad j=1,2,\mathrm{L\,L}\,i-1)$; $\{s\}=[s_1\ \ s_2\ \ \mathrm{L}\ \ s_n]^T$; $\{e\}=[e_1\ \ e_2\ \ \mathrm{L}\ \ e_n]^T$; $\{e_{sh}\}=[e_{sh,1}\ \ e_{sh,2}\ \ \mathrm{L}\ \ e_{sh,n}]^T$ 。

According to the equilibrium of internal force and deformation compatibility, the stress of concrete at any point is provided by

$$\{s_y\}=k_{1,y}\{e\}+k_{2,y}\{f\} \qquad (21)$$

where y = distance from computational fiber to reference point; $\{e\}$ are the values at reference point; $k_{1,y}=-(a+\frac{y}{r^2}b)$; $k_{2,y}=-(b+\frac{y}{r^2}c)$; $s_{c,y}(t_i)=s_{i,y}$; $\{s_y\}=[s_{1,y}\ \ s_{2,y}\ \ \mathrm{L}\ \ s_{n,y}]^T$

Let $y_1=0$, $y_2=d$ and put them into Eq.20, Eq.21, the time-dependent curvature is derived by

$$\{f\}=[C][(s_{01}\{W_1\}+s_{02}\{W_2\}+\{W_3\})] \qquad (22)$$

where $[C]=\left[k_{2,1}[T]_1^{-1}[A]-[T]_1^{-1}(dI+k_{2,2}[A])\right]^{-1}$; $[T_1]=I-k_{1,1}[A]$; $[T_2]=I-k_{1,2}[A]$; $\{W_1\}=-[T_1]^{-1}\{B\}$; $\{W_2\}=[T_2]^{-1}\{B\}$; $\{W_3\}=([T_1]^{-1}+[T_2]^{-1})\{e_{sh}\}$; $s_{01}=e(t_k)E_c(t_k)$; $s_{02}=[e(t_k)+df(t_k)]E_c(t_k)$.

Substitution of s_{01} and s_{02} into Eq.22, and select the last equation in Eq.22, we have

$$f_n=\sum_{i=1}^{n}c_{ni}(w_{1,i}+w_{2,i})E_c(t_k)e_c(t_k)+\sum_{i=1}^{n}c_{ni}w_{2,i}dE_c(t_k)f(t_k)$$
$$+\sum_{i=1}^{n}c_{ni}w_{3,i} \qquad (23)$$

which can be rewritten as

$$f_n=E_c(t_k)\left[\sum_{i=1}^{n}c_{ni}(w_{1,i}+w_{2,i})\quad \sum_{i=1}^{n}c_{ni}w_{2,i}d\right]\begin{bmatrix}e_c(t_k)\\ f(t_k)\end{bmatrix}+\sum_{i=1}^{n}c_{ni}w_{3,i} \qquad (24)$$

Substitution of Eq.17 into Eq.24, considering $M_k=N_ke(y)$, Eq.24 turns to

$$f_n-\sum_{i=1}^{n}c_{ni}w_{3,i}=E_c(t_k)[\Gamma_1\quad\Gamma_2]\begin{bmatrix}N_k\\ N_ke(y)\end{bmatrix} \qquad (25)$$

where $[\Gamma_1\quad\Gamma_2]=\left[\sum_{i=1}^{n}c_{ni}(w_{1,i}+w_{2,i})\quad\sum_{i=1}^{n}c_{ni}w_{2,i}d\right]\begin{bmatrix}\mathrm{H}_1 & \mathrm{H}_2\\ \mathrm{H}_2 & \mathrm{H}_3\end{bmatrix}^{-1}$;

$e(y)$ = distance from centroid of concrete to acting point of N_k .

In Eq.25, $e(y)$ and N_k are unknown variables needed to solve. How to determine $e(y)$ and N_k lies in the characteristics of system to applying force on structure. For instance, provided the eccentric distance $e(y)$ is set, the needed force N_k to generate long-run curvature $f_\infty(=f_n)$ at time t_k can be calculate by

$$N_k = \frac{f_n - \sum_{i=1}^{n} c_{ni} w_{3,i}}{E_c(t_k)[\Gamma_1 + \Gamma_2 e(y)]} \qquad (26)$$

6 DISCUSSION ON SYSTEM OF APPLYING FORCE

Several kinds of systems on applying force can be used to adjust deflection, Figure 2 illustrates the typical intelligent prestressed steel located outside of beam after applying force in a simply supported prestressed concrete bridge. Line AB represents the centroid of concrete in section along the span, ADB represents the up-dating profile of prestressed steel after applying extra force, in which two approaches can achieve this condition: one is that the original profile of prestressed steel is ACB, ADB results from activating upward force in point C; the other is that the original profile of prestressed steel is ADB, the extra force apply s on point A or B along direction DA or DB, in which one end of steel A or B is fixed. So two different systems of applying force can be adopted to fulfill this same purpose. The following is only to discuss on determining the applied force N_k and eccentric distance

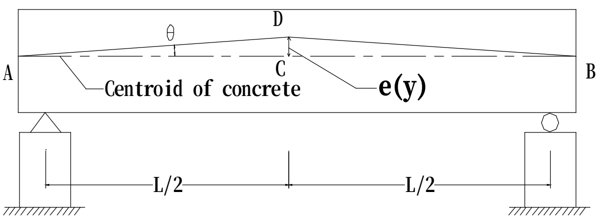

Figure 2. Schematic diagram of typical system for applying force

$e(y)$ based on first approach in Figure 2.

If the long-run curvature needed to be adjusted is f_∞, then horizontal component of N_{AD} is equal to N_k, solution of equations of force equilibrium gives

$$N_K = E_p A_p \left(1 + \frac{s_r(t_k,t)}{E_p} - \frac{L}{\sqrt{4e^2(y) + L^2}} \right) \qquad (27)$$

where L = length of span.

If point C is not in the controid of concrete, existing eccentric distance e_0 above the controid, assume the existed pre-tension in steel ACB stretched initially at time t is N_0 at time t_k, we have

$$N_K = \frac{(N_0 + s_r(t_k,t)A_p + E_p A_p)L}{\sqrt{4e_0^2 + L^2}} - \frac{E_p A_p L}{\sqrt{4e^2(y) + L^2}} \qquad (28)$$

From Eq.26 and Eq.27 or Eq.28 we obtain

$$a_1 N_k^4 + a_2 N_k^3 + a_3 N_k^2 + a_4 N_k + a_5 = 0 \qquad (29)$$

where a_i = coefficient relative to $s_r(t_k,t)$ determined by N_0.

Therefore the applied upward force is given by

$$N_{CD} = N_K tgq = N_K \frac{2e(y)}{L} \qquad (30)$$

Actually, there are three variables in Eq.30, N_0, N_k and $e(y)$, in which the results of N_k and $e(y)$ are affected by N_0 and the value of N_0, to some extent, plays a decisive role in workability of system of applying force for the maximum of force and displacement provided by system is limited, thus the value of N_0 should be in a reasonable range.

If the bridges given in Figure 2 is a hyperstatic system, the needed forces along span to adjust deflection can be derived on a set of equations considering the optimum design, in which the calculation is more complicate than discussed above.

7 EXAMPLE

Hu & Chen (2003) have fulfilled a 630-day time-dependent experiment on behavior of simply supported T-section prestressed concrete bridge with 6 meters long. The initial time to stretching steel is 10 days after curing concrete and the self-weight of beam begins to act and first extra load 7.7 KN/M is applied simultaneously, two months later second extra load 9.33 KN/M is applied over beam. The experimental results is given in Figure 3. To illustrate theoretically the feasibility of intelligent system on long-run deflection control presented in this paper the applied extra load is replaced equivalently by tendons (thick reinforcement bars).

The deduced formulae on creep and shrinkage based on MC90 from short-term observation are

$$j(t,t_0) = 0.919 j(\infty,t_0) b_c(t,t_0) \qquad (31)$$

$$e_{sh,c}(t,t_s) = 1.023 e_{sh,\infty} b_s(t,t_s) \qquad (32)$$

The firstly added load can be replaced by tendon with initial prestress 300 MPa and eccentric distance $e_0 = 5.3cm$ above centroid of concrete. The diameter of tendon is 40mm and the standard strength is 540 MPa. The theoretical values based on replaced tendon before adjusting is showed in Figure 3. To keep the long-run deflection derived from adjusted system almost the same as original system, the needed $N_k = 330.5KN$ calculated by Eq.29 should be applied and the relative forced upward displacement of tendon is 6.4cm ($e(y) = 11.7cm$). Figure 3 compares the experimental results and theoretical values based on adjusted system.

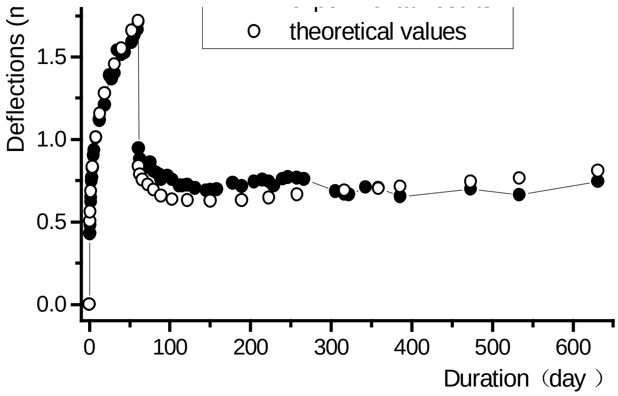

Figure 3. Comparison of deflections between experimental results and theoretical values at midspan

Figure 3 shows that the theoretical deflections based on adjusted system fit well with experimental results in original system during load duration, which means the deflection-adjustable system through adjusting the location of intelligent tendon is feasible and theoretical value will achieve the predicted value. Therefore, the presented intelligent system on long-run deflection control is practical theoretically.

8 CONCLUSIONS

This paper discusses mainly on the concept and structure of intelligent system on long-run deflection control for prestressed concrete bridges, and introduces the functions of sub-models and their key techniques. Numerous novel ideas are put forward and sets of equations on time-dependent structural analysis are established considering the effect of creep, shrinkage, sectional characteristics of concrete, steel, relaxation of steel and time-dependent modulus of concrete. An example is introduced to illustrate the feasibility of the intelligent system, showing the system practical theoretically.

Because of the complicity of the intelligent system many key techniques, such as the detailed system for applying force or the mechanics system to make tendons displace and theories on optimum design for applied forces in hyperstatic structure, left to be resolved are not discussed in this paper, much work need to do to make intelligent system on long-run deflection control for prestressed concrete bridges real.

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