Long-term Behavior of Composite Bridges with Full-Depth Precast Decks

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The long-term behavior of composite bridges with precast decks due to the time-dependent behavior of concrete such as creep and shrinkage is of fundamental concerns in maintenance of in-service state. It is necessary therefore to evaluate exactly the variations due to time-dependent structural behaviors. In this study, the time-dependent brhavior of a full-scaled composite beam with full-depth precast decks were measured for 430 days and the results are compared with analysis results. On the basis of these results, the long-term behavior of a real designed composite bridge with precast decks can be evaluated and it is analyzed specially on the loss of compressive stress of precast decks, which must be taken into consideration in determining the initial prestressing force. Finally, the effects of parameters considered on the long-term behavior in design of the precast deck system are investigated based on the analytical studies.

Key Words: long-term behavior, creep, shrinkage, precast deck,, compressive stress, prestress

1. Introduction

For the rapid replacement of the deteriorated concrete deck in steel-concrete composite bridges as well as new construction of composite bridges with concrete decks of wider span, the precast deck system is very effective and has been widely used in several countries. However, there are many problems in serviceability, such as crack and water leakage in transverse joints, in several bridges. In the case of the full-depth precast deck system as shown in Fig. 1, which is female-to-female type joint system, there is no reinforcement in the transverse joints except longitudinal prestressing steel. Therefore, it is necessary to prevent the initiation of crack in transverse joints and basic criterion to determine the longitudinal prestress is to prevent tensile stress in transverse joints, where bonding strength of the transverse joint is neglected.

Because of time-dependent behavior of concrete, the precast deck system has long-term variations, which are prestress loss, increase of deflection and especially loss of compressive stress of concrete slabs due to the constraint of steel girders. Time-dependent behavior of composite bridges with precast decks is of fundamental concern in determining initial prestressing force and in-service maintenance of prestressed concrete bridges. Actually, the loss of compressive stress at the joints is the most important factor and that amount due to the time-dependent behavior is very large.

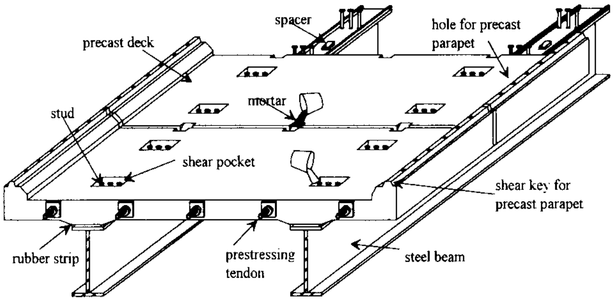


Fig. 1 Full-depth precast deck bridge system

Although several design codes specify the minimum value of prestress for joints without reinforcement, they did not consider creep and shrinkage. Dezi et al.(1995) evaluated the long-term behavior of the post-tensioned composite bridge, but the results cannot be applied to the precast deck system directly because of the characteristic peculiarities.

In this paper, the real time-dependent behavior will be evaluated through the experimental works on the full-scale simply supported composite beam with precast decks. Also, experimental results will be compared to the analytical results using the developed program. From these comparisons, the long-term behavior focused mainly on the compressive stress of transeverse joints is estimated analytically and some design parameters will be discussed through the analysis of the bridge model in the design example.

2. Experimental Works

Precast deck system requires step-by-step construction sequence as following: preparing the girder and placing the rubber strip and the spacer, placing precast decks, filling the transverse joints with grout, applying longitudinal post-tensioning, and filling the bedding layer and the pockets for stud shear connectors with non-shrink mortar.

Two simply supported full-scale composite beams were designed and manufactured according to internal effective T-beam part of the real three-girder composite bridge designed. These two specimens are the same in detials and just one composite beam, LTCB, was measured for a long time, 430days, because the other specimen, STCB, was used in fatigue test after 8day's measurement. Studs of 19mm-diameter were arranged uniformly at 400mm pitch and three studs were used in each pocket. The degree of shear connection, defined as the strength of the shear connection in a shear span as a proportion of the strength required for full shear connection, was anticipated to be 1.11. In order to eliminate the effect of compression in the precast slab on the steel beam, composite action was achieved by filling the shear pocket for stud connectors with mortar after applying the longitudinal prestress. Fig. 2 shows the specimen and its details.

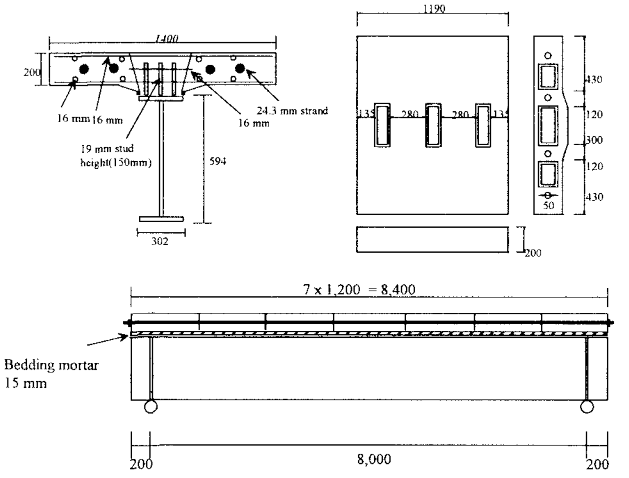


Fig. 2 Composite Beam

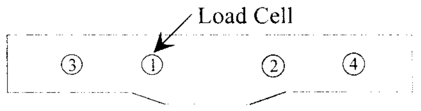
The time schedules of experiment and environments are listed in Table 1. The main parameter of two specimens is the connection time. To evaluate the effect of the connection time and transverse joints in this period, the time from prestressing to injecting mortar to shear pockets was made different in two specimens.

	STCB	LTCB
Age of concrete at loading (t_0) (day)	28	38
Time from pestressing to injecting mortar to shear pockets (Hr)	24	3
Measurement period (day)	8	430
Temperature range (℃)	23 ~ 28	10 ~ 30
Average relative humidity (%)	75	60

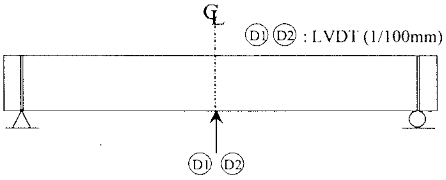
Table 1. Test Conditions

The material property of concrete, non-shrink mortar and re-bars were tested, and that of prestressing tendons were offered by manufacturer. The cylindrical compressive strengths of concrete for the precast slab and non-shrink mortar for the transverse joint are 34.6MPa and 38.2MPa for STCB, and 37.5MPa and 49.8MPa for LTCB.

The main items measured are prestressing force and deflection. For the measurement of prestressing force, a load cell of which the capacity is 50 tons was installed at the location of number 1 in Fig 3(a). The numbers mean the order of prestressing. One 1/100mm linear variable differential transformer(LVDT) was installed at the bottom of mid-span of the beam and the other LVDT was apart by 5 centimeters transversely as Fig. 3(b).



(a) Prestressing Force



(b) Deflection

Fig. 3 Measurements

Table 2 shows the losses due to anchorage set and elastic shortening of the composite beam. In the case of using the bolt-type anchorage, loss due to the anchorage set can be neglected generally, but a little loss occurred in this experiment because of not using the device of tightening bolts. The loss due to elastic shortening is dependent on the number of identical prestressing tendons, in other words, the number of times of additional tensioning, which was three times in this experiment.

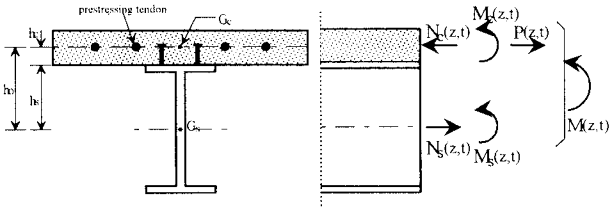
	Prestressing force at measured point (kN)		Prestress loss (%)	
	STCB	LTCB	STCB	LTCB
Initial	201.8	200.4		
After anchorage set	197.4	197.6	2.2	1.4
After additional tensioning (3 times)	193.9	195.3	1.74	1.14

Table 2. Initial Prestress Losses

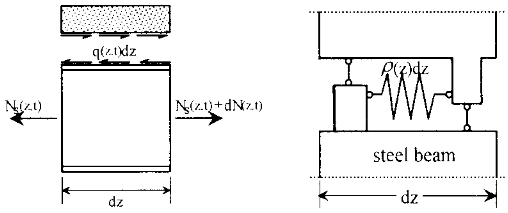
3. Time-Dependent Analysis

Many researchers(Soliman and Kennedy,1986, Takenada et al.,1986) have performed time-dependent analysis of composite beams with prestressed slab by disregarding the flexibility of the shear connection. Partial interaction theory was used in time-dependent analysis by several researchers(Tarantino and Dezi,1992, Dezi et al.1995). By assuming a flexible shear connection, shear force distribution at beam-slab interface could be obtained. Based on these numerical studies, the analytical studies using general step-by-step procedure are performed in this paper.

The concrete slab and the steel beam are considered separately, and linear elastic relationship is assumed for the connection, which is considered as continuously distributed along the beam axis. Fig. 4 shows the cross section of the composite beam and internal actions.



(a) Cross section and Internal Actions



(b) Shear Force per Unit Length and Connection Device

Fig. 4 Modeling Scheme

With reference to structures composed by a concrete slab and a steel beam that can slide with respect to each other, and in order to consider the prestressing of the slab with internal tendons, the following hypotheses are applied: (1) preservation of the plane cross section for the two elements; (2) no vertical separation between the parts; (3) The stiffness of the shear connections per unit length, ρ varies in this way [ρ =0, for t<t* ; ρ =R/s, for t>= t*, where, R and s are the stiffness and the spacing of connectors, and t* is the connection time between slabs and steel beams]; (4) The prestressing tendons are positioned at the centroidal level of the concrete slab; (5) The real time evolution of the prestressing steel relaxation is not considered. Assumption (5) is justified by the use of low-relaxation steel and on the basis of the observation that steel relaxation occurs in time more rapidly (very few months) with respect to the effect produced by concrete creep (several years) (Soliman and Kennedy, 1986). This hypothesis, which involves negligible numerical errors, considerably simplifies the mathematical formulation of the problem.

The analysis is performed considering the construction steps of the precast deck system and partial interaction behavior. The formulation is based on the force method so that the internal actions and the interface actions are assumed as unknowns. The internal actions consist of axial forces and bending moments in the slab(N_c , M_c) and in the steel beam (N_s , M_s); the interface actions are the longitudinal shear force per unit length [$q(z,t)$]. The time-dependent analysis of precast deck beams with flexible shear connectors is governed by a system of the following equilibrium and compatibility conditions. The equilibrium conditions of internal axial force and moment force for the whole section at any time are as follows.

$$N_c(z,t) - N_s(z,t) - P(z,t) = 0 \qquad (1a)$$

$$M_c(z,t) + M_s(z,t) + N_s(z,t)h_0 = M(z,t) \qquad (1b)$$

Under the assumption that there is no vertical separation between the slab and the girder, we can derive the following equation (2) for the same curvature.

$$\frac{M_s(z,t)}{E_s I_s} = \frac{M_c(z,t_0)}{I_c} J(t,t_0) + \frac{1}{I_c}\int_{t_0^+} J(t,\tau) dM_c(z,\tau) \qquad (2)$$

Where $J(t,\tau)$ is the specific creep compliance function which is defined as the sum of the instantaneous and creep strains at time t produced by a sustained unit stress applied at τ .

The compatibility condition at concrete-slip interface becomes the following:

$$u_{s,top}(z,t) - u_{c,bottom}(z,t) = \frac{1}{\rho}\frac{\partial N_s(z,t)}{\partial z} + \Gamma(z,t^*) \qquad (3)$$

Where the first term on the right side represents the slip obstructed by the shear connection (developed after t^*), and the second term denotes the free slip (developed before t^*). By differentiating the foregoing relation with respect to variable z, we can obtain the equation about the axial force of a steel beam and the strains of a slab and a steel beam. Finally, the compatibility equation using internal forces can be expressed as equation (4).

$$\frac{1}{\rho}\frac{\partial^2 N_s(z,t)}{\partial z^2} = -\varepsilon^{sh}(t_s,t) + \frac{N_s(z,t)}{A_s E_s} - \frac{M_s(z,t)}{I_s E_s} h_s + \left[\frac{N_c(z,t_0)}{A_c} - \frac{M_c(z,t_0)}{I_c} h_c\right] J(t,t_0)$$

$$+ \frac{1}{A_c}\int_{t_0^+}^{t} J(t,\tau) dN_c(z,\tau) - \frac{h_c}{I_c}\int_{t_0^+}^{t} J(t,\tau) dM_c(z,\tau) - \frac{\partial \Gamma(z,t_{con})}{\partial z} \qquad (4)$$

The prestressing force, acting at the cross section z and time t, can be expressed as equation (5) considering that the real experimental condition is free-slip between the concrete slab and the prestress tendon.

$$P(z,t) = P(z,t_0) + A_{sp}E_{sp}\int_{0^+}\frac{1}{l}\sum_{i=1}^{divide} l_i d\varepsilon_c(z_i,\tau) \qquad (5)$$

Where l is the length of the beam and *divide* is the number of segments into which the beam is divided for analysis.

From the foregoing five equations, the basic governing equation about the increment of the internal axial force of a steel beam at any time interval $\Delta t_k (= t_k - t_{k-1})$, which is inverted from the integral-differential type to algebraic form using the general step-by-step method, can be derived as equation (6).

$$\frac{\partial^2 N_s(z,t_k)}{\partial z^2} = C_1(t_k)\Delta N_s(z,t_k) + C_2(t_k)\sum_{i=1}^{divide}\Delta N_s(z_i,t_k) + C_3(t_k)\Delta M(z,t_k) + C_4(z,t_k) \qquad (6)$$

Where $C_1(t_k)$, $C_2(t_k)$, $C_3(t_k)$ and $C_4(t_k)$ are constants which can be obtained at each time step. Also, the change of the unknowns in a time interval, $\Delta f(z,t_k)$ is defined as the equation (7).

$$\Delta f(z,t_k) = f(z,t_k) - f(z,t_{k-1}) \qquad (7)$$

Equation (6) is so complex that it must be solved numerically. By introducing discretization of the time domain, application of the step-by-step general method reduces the integral differential problem to a set of differential problems. Introducing also a discretization for the beam axis, such problems can be solved with the finite difference method.

4. Comparisons

4.1 Behavior during the initial time interval(t_0 ,t^*)

The experimental results of two specimens were compared for 8 days that is the measured period of the specimen STCB. The prestressing forces of two specimens are represented in Fig. 5, and those are described at the same scale on the basis of the initial prestressing force of the specimen LTCB. The effects of the connection time and non-shrink mortar injected to transverse joints are evaluated from the comparison with the analysis solution ignoring the effect of transverse joints. In the analysis, it is assumed that the material property of non-shrink mortar is the same as the precast panels and the injection time of non-shrink mortar is also considered as the same as the manufactured time.

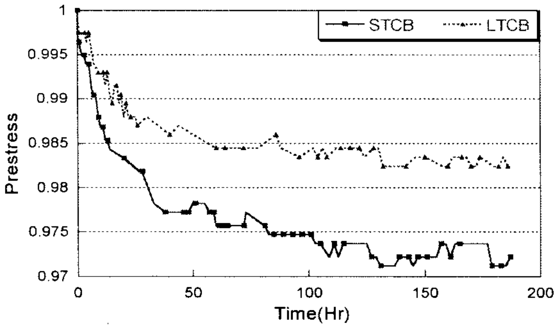


Fig. 5 comparison of prestressing force for initial 8 days

Specimen	STCB	LTCB
initial prestressing force	193.9 kN	195.3 kN
Total prestress loss for 8 days	2.8%	1.8%
Expected connection time After prestressing	2 days	1 day
Prestress loss until the connection time	2.2%	1.2%
Analytical prestress loss until the connection time	0.6%	0.4%

Table 3 comparison of prestressing force

From Fig. 5 and Table 3, considerable difference of prestress loss between two specimens occurred during the initial time interval in the case of the different connection time. Compared to the prestress loss of the analytical solution in which the effects of transverse joints is not considered, the experimental results of two specimens are over three times. Furthermore, in case of the experiment, the difference of prestress loss between two specimens is by about 1%. This value is very large considering that the total prestress loss after 430 days of LTCB is about 7.1%. The reason of this large difference can be judged because the non-shrink mortar is of early ages so the creep strain by applied prestressing force occurs very suddenly during the initial time interval. In case of different connection times like these two specimens, the effect of early-age mortar during the times free to slip may influence the long-term behavior very much. Therefore, in the analysis of the long-term behavior of composite beam, the effects of transverse joints must be considered by means of adjusting the result to the connection time.

4.2 Long-term behavior of composite beam LTCB

Fig. 6 and 7 show the variation of the prestressing force and deflection by test compared to the analysis result using step-by-step method. The results at the final time of test are also arranged, compared to the analysis results classified by the models of time-dependent behavior of concrete in each specification in table 4.

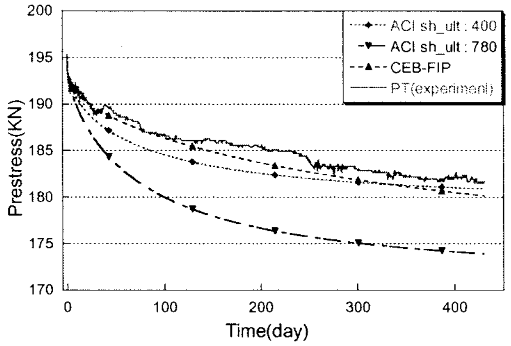


Fig. 6 long-term variation of prestressing force

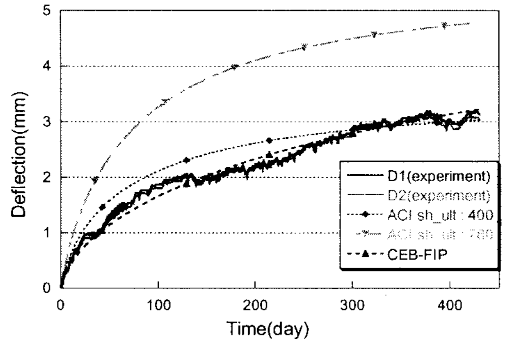


Fig. 7 long-term variation of deflecrion

			Prestressing force (kN)	Deflection (mm)
Result of experi-ment	Initial value		195.31	0
	Final value		181.50 (loss : 7.1%)	3.13
Result of analy-sis	ACI-209 model	Final value	180.92	3.04
		error	4.2%	2.9%
	CEB-FIP model	Final value	180.18	3.23
		error	9.5%	3.2%

Table 4 Comparison of the result of test and analysis

As the model of time-dependent behavior of concrete, ACI-209 model(1994) and CEB-FIP model(1990) were used in the analysis. The prestress loss until the connection time considered as 1 day after prestressing was adjusted in the analysis using the result of test because of the effects of non-shrink mortar injected to transverse joints.

In case of the ACI-209 model, time-dependent behavior of concrete is decided according to the ultimate creep coefficient and the ultimate shrinkage strain. Because the level of applied prestress is not so high, the long-term behavior of composite beam is influenced dominantly by the shrinkage strain. Therefore, the ultimate creep coefficient was used as the fixed value 2.35 in the analysis, which is the generally accepted average value in normal concrete.

In case of composite beams, it can be equivalently considered that the shrinkage strain of concrete panels decreases because of the constraint of steel girder. Therefore, the long-term behavior of composite beam was compared with the analytical solution according to the variation of the ultimate shrinkage strain in using ACI-209 model and 400 μ strains as the ultimate value was most closely fitted to the test result. The analysis result using CEB-FIP model was also fitted to the test result well. Table 4 shows the test results compared with analytical solution and its errors at the final time, 430 days. When the results just at the final time are compared, the analysis results using ACI-209 model, for which case the ultimate shrinkage strain is 400 μ strains, are fitted better than those using CEB-FIP model. However, the trend of the real time-dependent behavior of composite beam is similar to the analysis results using CEB-FIP model as shown Fig. 6 and 7.

4.3 Expected variation of compressive stress of concrete

In determining the magnitude of prestress to prevent cracks at joints, the losses of compressive stress are the most important in precast deck system because the loss of compressive stress is relatively greater than that of prestressing force. From the analysis results compared in the section 4.2, the expected variation of the compressive stress at each point of concrete slab was evaluated. It is based on the analysis result by the step-by-step method using ACI-209 model and Fig. 8 shows the variation of the compressive stress. In Fig. 8, top_slab, bottom_slab and bottom_hunch denote the top of the slab, the bottom of the slab and the bottom of the hunch respectively.

The loss of compressive stress at the top of concrete slab is about 20%. However, at the bottom of the slab and the part of hunch, the loss of the compressive stress is over 50%. From these results, longitudinal stress at the bottom surface of the joint should be a design criterion for the prevention of cracks at joints.

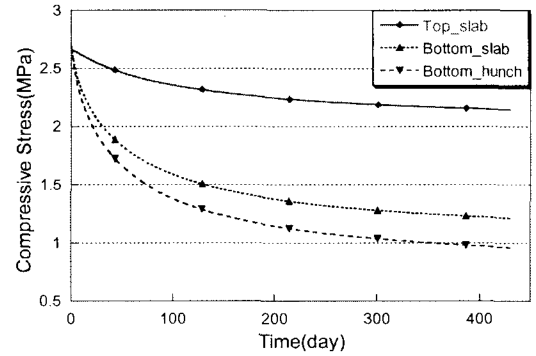
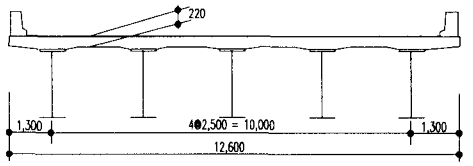


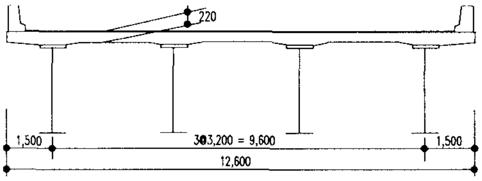
Fig. 8 the variation of compressive stress of concrete slab

5. Design Examples

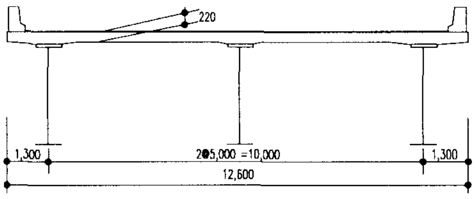
Based on the previously evaluated results longitudinal prestresses of three bridge models shown in Fig. 9 are evaluated on the effect of long-term behavior to make sure the serviceability of precast deck bridges.



(a) S2500



(b) S3200



(c) S5000

Fig. 9 Bridge Models

Design / Model	Initial Compression	Number of Tendon
S2500	4.92 MPa	14
S3200	5.80 MPa	16
S5000	7.07 MPa	20
Design Value	Area of the Deck : 2772000 mm^2 Bearing Strength of the Deck = 1500 kN Prestress Tendon : Ap = 691mm^2, Pu= 1260 kN	

Table 5. Assumed Initial Longitudinal Prestress

Firstly, effective longitudinal prestress is evaluated considering the longitudinal tensile stresses in the bridge deck due to appended dead load, live load including impact, and temperature differential, where the first transverse joint is assumed to be located 2.4m from the support. For each bridge model the effective prestresses are 2.10MPa, 2.83MPa, and 3.89MPa respectively(Shim, 2000). Assuming the prestress loss due to shrinkage as 2.0MPa, and 20% due to creep, initial longitudinal prestress is evaluated as shown in Table 8.

And then, prestress losses are evaluated using the analytical solution derived before. Assuming the construction schedule as followings: applying the longitudinal prestress 60 days after casting precast decks and 1 day after casting the transverse joints with non-shrinkable mortar, connecting steel beam and precast decks with filling the block-outs for studs shortly after applying the prestress.

Service life of the bridge is assumed as 50 years. Relative humidity used in the analysis is 70 %, and full-interaction behavior of the bridge is assumed.

The losses of prestressing force for S2500, S3200, and S5000 are 5.3%, 4.8%, and 3.8%, respectively. And the change of compressive stress at the top and bottom surface of the deck is shown in Fig. 10. Loss of compressive stress at the joint is greater than 40% of initial prestress. Because the level of compressive stress is so high that creep effect on the loss increases. Judging from the results, the initial prestress should be increased. Iterative procedure must be done to determine the initial prestress.

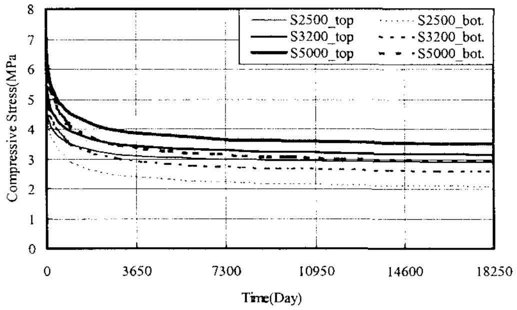


Fig.10 Change of Compressive Stress at Concrete Deck

To reduce the loss of compressive stress, it is favorable to delay the prestressing time. Reducing restrained shrinkage strain affecting the loss will also decrease the loss due to creep. The loss of compressive stress at the bottom surface of the joint can be reduced by adopting partial composite action or non-composite action. In case of non-composite action, the loss of compressive stress will be about 5%, which is similar to that of prestressing force.

Longitudinal prestressing causes the increase of deflection. Therefore, it should be considered in determining the initial camber. Based on the results of the analyses performed before, the increase of deflection due to creep and shrinkage is presented in Fig. 11.

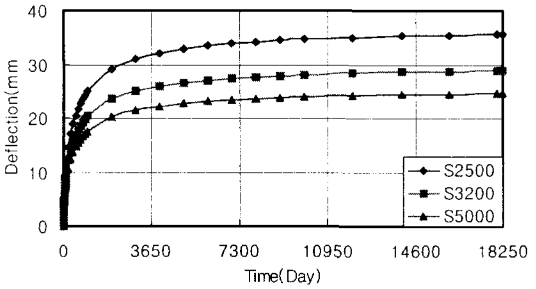


Fig. 11 Increase of Deflection

6. Conclusion

In this study, the long-term behavior of composite bridge with precast decks are evaluated by experiments and the program for long-term behavior of precast deck bridges has been developed using the analytical solution. The results of the analysis are compared with those of experimental works on a composite beam, and they show good agreements. The analysis using CEB-FIP model(1990) is well fitted to the long-term behavior of the test specimen, and it is very similar to the trend of the real behavior of the test specimen. Also, ACI-209 model(1994) shows good agreement with test results when the ultimate shrinkage strain is adjusted as 400 μ strains.

From the long-term measurement of the manufactured composite beam for 430 days, the prestress loss and the increase of deflection of the specimen can be evaluated. By fitting the analytical solution to the test results, the long-term behavior of the whole structure system can be judged, then the variation of compressive stress of concrete slab for the precast deck system can be evaluated through analytical solution. The loss of compressive stress at the bottom part of concrete slab occurs over 50% during the test period. Therefore, the loss of compressive stress is the most important criterion in determining the magnitude of longitudinal prestress.

In the design example, long-term losses of prestress for three bridge models are evaluated using the program. In determining the initial prestress, iterative procedure is used. Judging from the results, it is favorable to reduce shrinkage strain for reducing the loss of compressive stress at joints. And, partial composite or non-composite bridges can be considered as the method of reducing the losses.

For the application of the precast deck system to continuous bridges, prestressing method for the negative moment region should be investigated. There are mainly three methods, internal tendon, external tendon, jack up and down the internal support. Based on the concept of this paper, it is necessary to perform experimental works and analytical studies for the continuous composite bridges with precast decks.

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