Prevention of Slab Cracking at Interior Supports in Composite Girder Bridge

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ABSTRACT This study deals with cracking control of concrete deck in steel-concrete composite bridges according to the concrete slab casting sequences. In correlation studies between casting sequences, time-dependent effects of concrete creep and shrinkage are implemented in the analytical model. Finally, the methods of cracking control in terms of concrete slump and relative humidity are suggested to prevent early transverse cracking of concrete slab.

KEYWORDS Cracking control, Concrete deck, Steel-concrete composite bridges

1. INTRODUCTION

Steel-concrete composite bridges are constructed by placing slab concrete on steel girders with shear connectors to guarantee monolithic behavior. In the case of these bridges, the section type, casting deck concrete and so on, affect the behaviors of the bridges because of their weak stiffness of girders. Especially, there are two ways of casting deck concrete. The first method is sequential casting and the other is continuous casting. Sequential casting is generally recommended in which deck concrete is poured first in the middle portion of the interior span and then poured at the interior supports based on the influence line. But it produces many construction joints and increases the construction time and cost resulting from the movement of equipment and concrete forms at each construction stage. In continuous casting, deck concrete is cast from one end to the other end of a bridge. Even though this method can reduce significantly the construction time and cost, it is not generally recommended because some potential structural disadvantages such as development of transverse cracking in the earlier-placed deck can be expected due to the negative moment caused by pouring concrete in the adjacent span.

In Korea, closed-top box girder sections are generally used for steel box girder bridges because of the convenience of delivery and installation. As expected, those sections give much larger flexural rigidity than open-top box girder sections used in many countries, before composite action is developed. In this study, the effect of slab concrete placing sequence on the transverse cracking of slab decks has been investigated theoretically.

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Based on the ACI Code, the ultimate shrinkage strain is expressed as a function of concrete slump and the relative humidity. Finally, field recommendations in terms of concrete slump and relative humidity are suggested to prevent early transverse cracking of concrete slabs.

2. Analytical Model

For continuous placing of concrete, concrete is generally poured in a day and therefore, it can be assumed that there are no time-dependent effects such as creep and shrinkage during the placing. In general, after the concrete is placed, the time-dependent effects due to creep and shrinkage occur with the aging of the concrete. Shrinkage strain can be evaluated directly by utilizing the shrinkage model proposed in the design codes[1] since it is defined as the volume change which occurs independently of imposed stresses, whereas creep is defined as the increase in strain under sustained stress.

In this paper, the first-order recursive algorithm based on expansion of creep compliance, proposed by Scordelis and Kabir, has been adopted because that model can simulate the stress history effectively in spite of its simplicity in application. In describing the uniaxial stress-strain behavior of concrete, the model proposed by Hognestad is used after some modifications in this paper. To simulate the cracking of concrete and to describe the tension stiffening effect, the strain softening in tension is included [2]. Reinforcing Steel is assumed to be a linear elastic-perfect plastic and thermal strain, which is the only nonmechanical strain expected for steel, is not considered in this study.

Based on the assumed displacement field formulation, a two-dimensional beam element stiffness matrix is constructed [3]. The nonlinear solution scheme selected in this study uses the tangent stiffness matrix at the beginning of the load step in combination with a constant stiffness matrix during the subsequent correction phase, that is, the incremental-iterative method. More details for analytical model can be found in Ref. 2.

3. Field Test and Analysis

To verify the analytical results, field examinations for a bridge are also performed. The test bridge is located on Korea National Route 23 and is 6-span continuous. As shown in Fig. 1, this bridge has a total length of 300m and a width of 19.5 m. It has 3 girders with variable web depth. Considering the symmetry, strain gages were installed on the left three spans of the middle girder (locations A to F in Fig. 1).

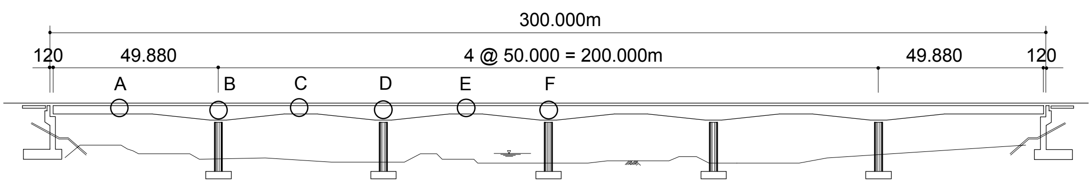
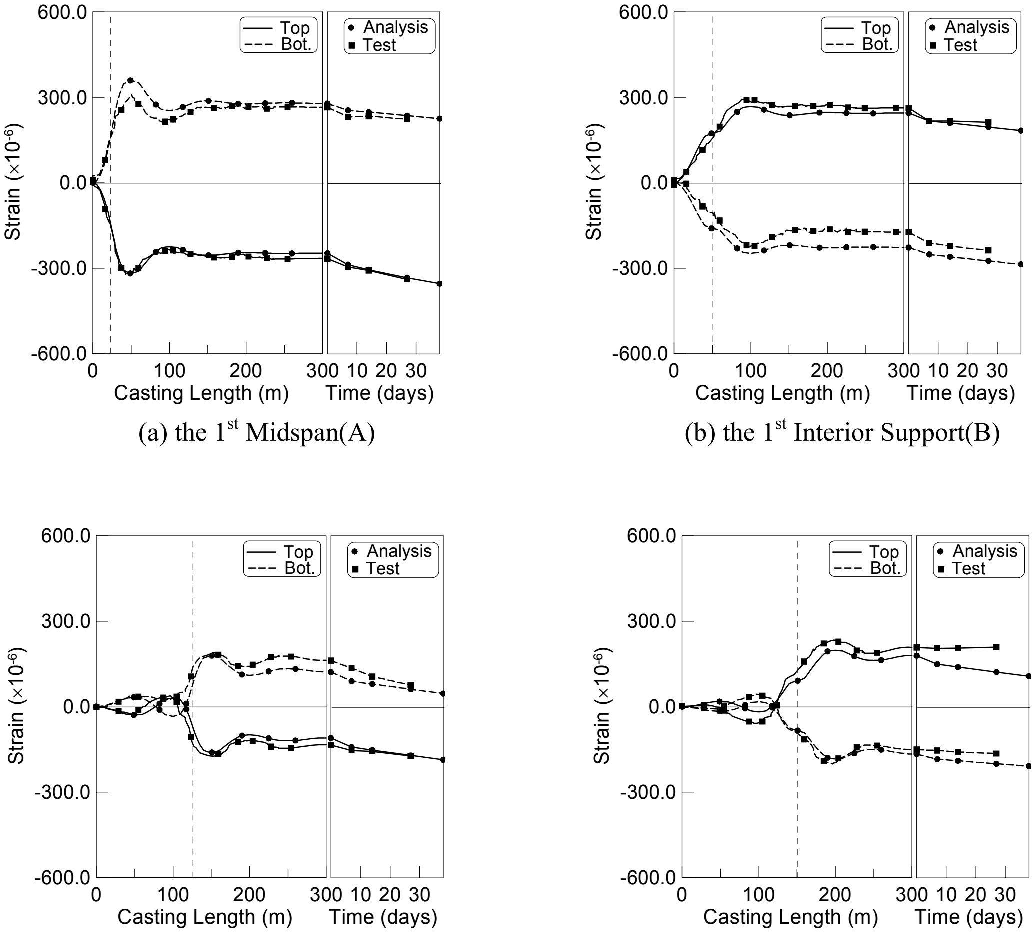


Fig. 1 Test Bridge

Deck concrete was placed according to the continuous placing sequence. For the short-term behavior, strain gages were measured simultaneously about every 10 minutes from the beginning of placing concrete to the end of construction. For the long-term behavior, strain gages were measured once a week for one month after completion of construction. Fig. 2 shows the analytical and measured strains for test bridge 1. The strains on the top and bottom fibers during the placing of the concrete are shown on the left side. The right side indicates the time-dependent strains due to the creep and shrinkage after the placement of the deck concrete is finished. The dotted vertical line represents the specific gage location.



(a) the 1st Midspan(A)

(b) the 1st Interior Support(B)

(c) the 3rd Midspan(E)

(d) the 3rd Interior Support(F)

Fig. 2 Measured and Calculated Strains for Test Bridge

As a whole, both field measurements and analytical results agree well except for the third interior support (Fig. 2(d)). It was supposed that these differences might come from inappropriate gage attachments. Besides, waterproof gages were not used at some gages on the third interior support. This can explain why the measured strain did not agree surprisingly with the calculated strain in the area of long-term behavior.

The ultimate creep coefficient and the ultimate shrinkage strain recommended in the Korea Highway Specification [1], are 4.0 and $150\sim200\times10^{-6}$, respectively. In this study, the ultimate creep coefficient ϕ is fixed as 4.0 because the support conditions during construction do not change. Otherwise, the rate of drying shrinkage affects the stress in concrete deck. From the parametric study, it is found that the ultimate shrinkage strain ε_{sh} of 600×10^{-6}, which is calculated values by SRSS[4], gives the best correlation to the measured results. All numerical results shown in Fig. 2 were calculated based on the ultimate shrinkage strain ε_{sh} of 600×10^{-6}. This value is much larger than the recommended value for design purposes in the Specifications. There are possibly differences between the rate of drying shrinkage assumed in the design stage and the actual state on the site. Therefore, it is required that more effort to reduce the drying shrinkage rate during construction be made.

4. EFFECTS OF CASTING AND SHRINKAGE OF DECK CONCRETE

In order to isolate the effect of slab casting sequences, test bridge 1 is analyzed using the ultimate shrinkage strain of 600×10^{-6}. The time gaps for each stage in the sequential casting are assumed to be 0, 3, 7 and 15 days, and the sequential casting is shown in Fig.3. Also, in order to consider the effect of the ultimate shrinkage strain, a similar analysis has been conducted and compared when the ultimate shrinkage strains are 200×10^{-6}, 600×10^{-6}, and 800×10^{-6}, assuming no time gap.

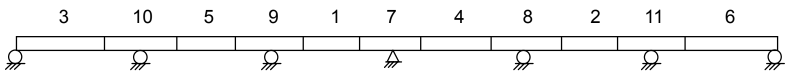
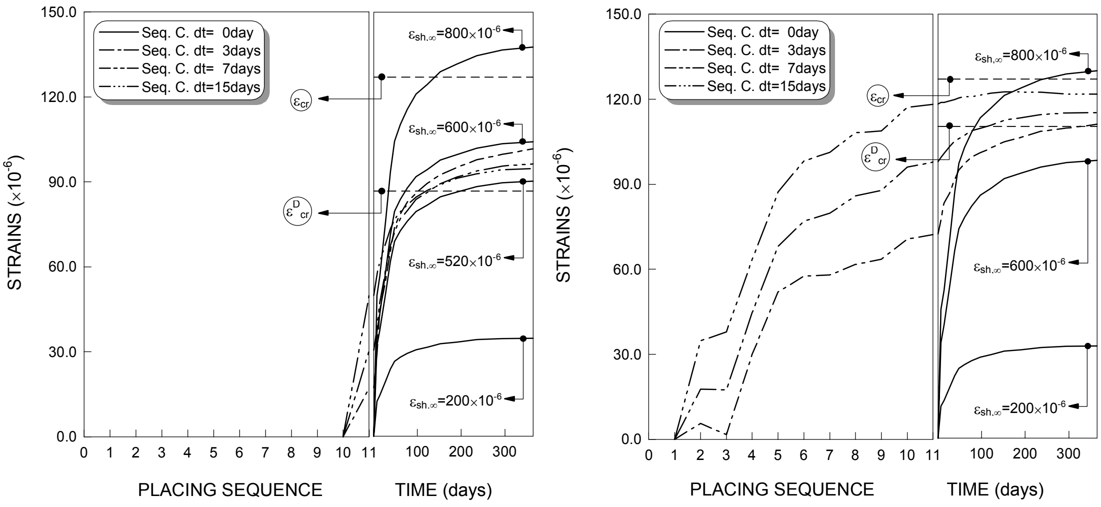


Fig. 3 Sequential Casting

In Fig. 4, the result for the mechanical strain of the concrete deck, which is closely related to the cracking of concrete, is shown. The compressive strain due to the drying shrinkage at the midspan increases as the time gap increases. This means more restraint on the shrinkage of concrete and higher tensile stress develops in the slab concrete at the midspan. As expected, the mechanical strains are always tensile both at the midspans and at the interior supports. As the time gap increases, the mechanical strain of concrete also increases. This means more restraint on the shrinkage of concrete and higher tensile stress develops in the slab concrete at the midspans that are placed earlier. Therefore, in order to control the possible cracks at the midspan during construction, it is required to reduce the time gap in sequential.

Otherwise, since the tensile strength of concrete is about 3.13 N/mm^2 for normal concrete, and its corresponding strain is about 127.51 $\mu\varepsilon$, the cracking occurs when the mechanical strain exceeds 127.51 $\mu\varepsilon$ marked ε_{cr}. Also ε_{cr}^{D} is the strain level corresponding to cracking with live loading defined at Korean Specification [1]. So, if live loads are applied, cracking will develop at the third midspan when the ultimate shrinkage strain $\varepsilon_{sh,\infty}$ exceeds about 655 $\mu\varepsilon$, and at the first interior support when the ultimate shrinkage strain $\varepsilon_{sh,\infty}$ exceeds about 520 $\mu\varepsilon$.



(a) the 1st Interior Support (b) the 3rd Midspan

Fig. 4 Mechanical Strain of Deck with the Casting Sequences and the Ultimate Shrinkage Strains

5. CRACK CONTROL

The ultimate shrinkage strain in the ACI model[5] is determined based on the next formula.

$$\varepsilon_{sh,\infty} = 730\times10^{-6}\cdot k_H \cdot k_T \cdot k_S \cdot k_C \cdot k_F \cdot k_A \cdot k_{thk} \quad (1)$$

where the coefficients determined by the relative humidity, the curing time, the slump and the unit weight of cement, the ratio of sand to aggregate, the air content, and the thickness of the member. Since the slab thickness is usually 0.25m in test bridge, the coefficient k_{thk} determined by the average thickness of a member is 0.86. k_F, k_C, and k_A are 0.86, 0.994, and 0.99 if the air content, the ratio of sand to aggregate, and the unit weight per cubic meter are assumed to be 5%, 40%, and 400kg/m^3, respectively, which are the typical values in mixed concrete. k_T is 1.0, assuming 7 days of moisture curing period. Substituting all these values into Eq. (1), the ultimate shrinkage strain becomes

$$\varepsilon_{sh,\infty} = 565.4\times10^{-6}\cdot k_H \cdot k_S \quad (2)$$

If the relative humidity and the slump are determined, depending on the field conditions, the ultimate shrinkage strain can be calculated from Eq. (2) and listed in Table 1. As mentioned before, cracks will develop at the first interior support of the test bridge when the ultimate shrinkage strain $\varepsilon_{sh,\infty}$ exceeds 520με under service loading conditions. If the average relative humidity is 70%, cracks will not develop with a slump of up to 18cm according to Table 1. If the average relative humidity is 40% corresponding to dry atmosphere, cracks are inevitable unless special measures are taken such as a longer curing period.

Table 1 Ultimate Shrinkage Strains to Relative Humidity and Slump ($\times 10^{-6}$)

Slump (cm)	Humidity (%)						
	40	50	60	70	80	90	99
8	*576*	*518*	*461*	*403*	*346*	*173*	*17.3*
9	*585*	*527*	*468*	*410*	*351*	*176*	*17.6*
10	*594*	*535*	*475*	*416*	*357*	*178*	*17.8*
11	*603*	*543*	*483*	*422*	*362*	*181*	*18.1*
12	*613*	*551*	*490*	*429*	*367*	*184*	*18.4*
13	*621*	*559*	*497*	*435*	*373*	*186*	*18.6*
14	*631*	*568*	*505*	*441*	*378*	*189*	*18.9*
15	*640*	*576*	*512*	*448*	*384*	*192*	*19.2*
16	*649*	*584*	*519*	*454*	*389*	*195*	*19.5*
17	*658*	*592*	*526*	*461*	*395*	*197*	*19.7*
18	*667*	*600*	*534*	*467*	*400*	*200*	*20.0*

6. CONCLUSIONS

The analytical results showed that the effect of slab casting sequence is negligible for both short-term and long-term behavior of bridges because transverse cracks may be developed when the tensile strain of concrete in the plastic state exceeds 1500με[6]. Therefore, the continuous casting for closed box sections used in Korea is recommended for easy and fast construction without any danger of increasing transverse cracking. The effect of drying shrinkage is the most critical for long-term behavior of a bridge and transverse cracking. Cracks may develop at interior supports without any live loading if the ultimate shrinkage strain is near 800×10^{-6}. A field recommendation for crack control was suggested on the basis of the ACI model. In this recommendation, concrete slump and relative humidity are the important factors. If the average relative humidity is 60%, the slump should be less than 13 cm to avoid transverse cracks at interior supports with live loading. If the average relative humidity is 40% corresponding to dry atmosphere, cracks are inevitable unless special measures are taken such as a longer curing period.

REFFERENCES

[1] Korea Society of Civil Engineers, "Korea Highway Specification", Korea Ministry of Construction & Transportation, Seoul, 1996 (in Korean).

[2] Kwak, H.G., Seo, Y.J. and Jung, C.M., "Effects of the Slab Casting Sequences and the Drying Shrinkage of Concrete Slabs on the Short-term and Long-term Behavior of Composite Steel Box Girder Bridges. Part 1", Engineering Structures, Vol.22, No.11, pp. 1453-1466, 2000.

[3] Kang, Y.J., "Nonlinear Geometric, Material and Time Dependent Analysis of Reinforced and Prestressed Concrete Frame", Report No. UC–SEEM 77–1, UNIV. of California, Berkeley, 1977.

[4] Clough, R. A. and Penzien, J., "Dynamics of Structures", McGraw-Hill, New York, USA, 1993.

[5] ACI Committee 209, "Prediction of Creep, Shrinkage and Temperature Effects in Concrete Structure", 2nd Draft, Detroit, USA, 1992.

[6] Hilsdorf, H.K. and Lott, J.L. Revibration of Retarded Concrete for Continuous Bridge Decks, NCHRP , Report 106, Washington DC, 1970.