

Effects of Base Course Restraint on Early Age Transverse Cracking in CRCP

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Abstract

Asphalt Interlayer (AIL) is planning to be inserted between a concrete slab and base course in continuously reinforced concrete pavement (CRCP) in a super express highway in Japan. In this study, the effects of AIL are investigated based on observations and measurements on a test CRCP and simulations with three dimensional finite element method (3DFEM). The test pavement has two sections with and without AIL and strains and temperatures in the slab were measured. Investigation of the test pavement revealed that air temperature at the time of concrete casting affected the strains in the slab and crack pattern. Also, three closely spaced cracks were observed in the AIL section. To examine how AIL affects cracking situations, numerical simulations were conducted using a 3DFE model that was developed in this study. The model employs an incremental algorithm to consider changes of free contraction strain (FCS) and material properties of concrete at the early age. Mechanical interaction between steel and concrete was modeled with three nonlinear springs. The model was validated by simulating early age cracking of the slab and comparing the predicted strains and the strains measured in the test pavement. Effects of AIL on spacing and width of transverse crack were investigated with the model. The simulations of the 3DFE model revealed that the crack spacing does not change by using AIL, while AIL narrows the crack width. It was found that a small peak in tensile stress on the top of the slab near the crack appears in the AIL section, which might cause “secondary” crack near “primary” crack. The peak disappears after the secondary crack occurs near the primary crack, resulting in a cluster cracking observed in the test pavement.

Introduction

Continuously reinforced concrete pavement (CRCP) eliminates joints by forming small transverse cracks with a short spacing to provide a smooth surface. Spacing and width of the transverse cracks are controlled by longitudinal steel bars embedded at a third or half thickness of concrete slab. Composite pavements with CRCP overlaid with porous asphalt pavement are used in a super express highway that connects Tokyo and Nagoya. The express highway is planning to extend west beyond Nagoya. The extended express highway is planning to use a new pavement structure, in which an asphalt interlayer (AIL) is inserted between concrete slab and cement treated base course (CTB). In this study, the effects of AIL on early age cracking are investigated based on observations on a test CRCP and numerical simulations with three dimensional finite element method (3DFEM).

The test pavement that has two sections with and without AIL was constructed in an express highway (Taketsu, H., et al., 2014). Strains and temperatures in the concrete

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slab were measured continuously from just after casting concrete and cracking situation was observed periodically. In the observations three closely spaced cracks were found in the AIL section. To examine the effects of AIL on the cracking situation, numerical simulations were conducted using a 3DFE model that was developed in this study. The model employed an incremental algorithm to consider changes of free contraction strain (FCS) and material properties of concrete in the hydration process. Mechanical interaction between steel and concrete was modeled with three nonlinear springs. The model was validated by simulating early age cracking of the slab and comparing the predicted strains with the strains measured in the test pavement. Effects of AIL on spacing and width of crack were investigated with the model. Causes of the cluster cracking observed in the AIL sections were also explored.

Measurement on Test Pavement

Overview of Test Pavement. To compare mechanical behavior of CRCPs with and without AIL, a test CRCP was constructed. The test pavement has two sections, Section A and Section B, with different base course structures as shown in Figure 1. Base course of Section A was a 40 mm AIL and 160 mm CTB. That of Section B was only a 200 mm CTB. The length of each section was 280 m long. CTB had an axial compressive strength of 3 MPa. The AIL used a stone mastic asphalt concrete of which properties are presented in Table 1. The concrete slab with a thickness of 250 mm was placed over both sections. Longitudinal steel bars with a diameter of 16mm were embedded at a third depth from the top of the slab with a spacing of 125mm (0.64% steel reinforcement of cross section area). Transverse steel bars with a diameter of 13 mm were installed at a 300 mm spacing skewed with a 60 degree with respect to the longitudinal direction. The concrete mixture proportion of the slab is presented in Table 2. The flexural strength of the concrete was 5.17 MPa.

Measurement Instrumentation. Temperatures and strains in the slab had been continuously measured at every one hour since the slab was placed. The time of the placement was 18th April in Section A and 16th April in Section B. At the middle of each section, six measurement points were installed as shown in Figures 2(a) and 2(b). Each measurement point had six strain gauges with temperature sensors at 25mm from the top, middle and 25mm from the bottom as shown in Figure 2(c). Strains in the longitudinal and transverse directions and temperatures were measured at the points. Also, strain gauges were attached on steel bars at the same positions and strains of the steel bars were measured similarly.

Few transverse cracks were observed near the measurement points a month after the placement of concrete as shown in Figures 2(a) and 2(b). Width of the transverse crack was 0.15mm in Section A, and 0.35mm in Section B.

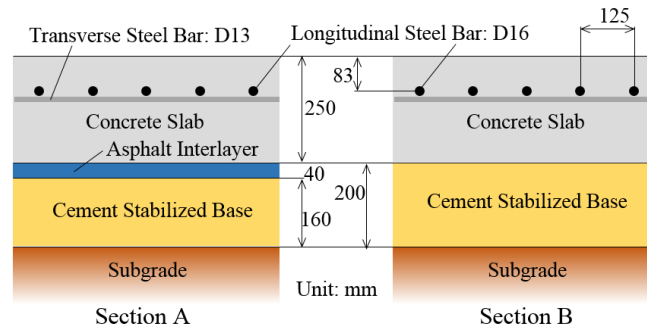


Figure 1. Cross Section of Test CRCP.

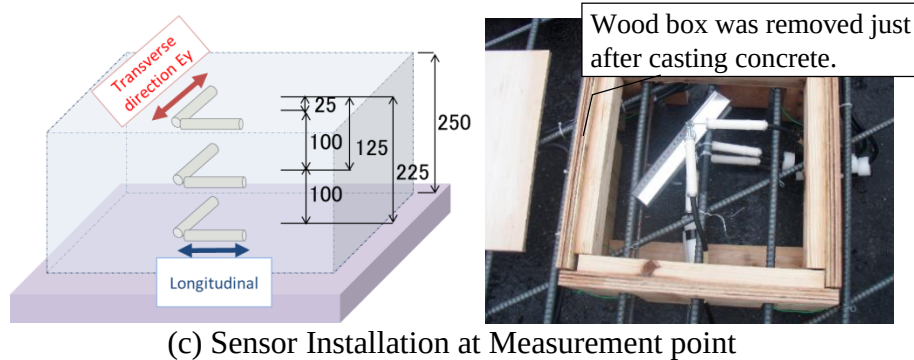
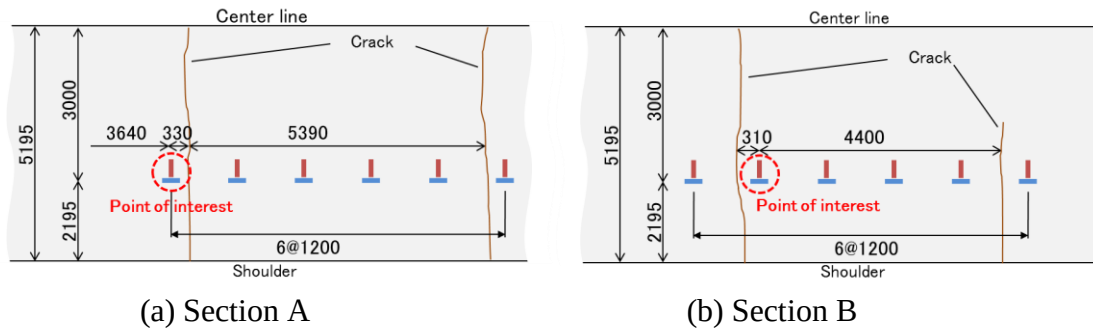


Figure 2. Test Pavement and Sensor Installations.

Table 1. Properties of Stone Mastic Asphalt Mixture (SMA).

Item	Value
Optimum asphalt content [%]	6.5
Density [g/cm ³]	2.388
Air void [%]	2.5
Marshal stability [kN]	9.70
Flow [1/100cm]	38
Residual stability [%]	90.8

Table 2. Mixture Proportion of Concrete.

Gmax [mm]	Slump [cm]	W/C [%]	s/a [%]	Air [%]	Unit content						Ad [%]
					W	C	S1	S2	G1	G2	
40	4.5	46.1	40	6.0	140	304	293	435	610	499	1.1

Note: Gmax: maximum aggregate size, W: water, C: Cement, S1& S2: Fine aggregate, G1& G2: Coarse aggregate, Ad: Admixture

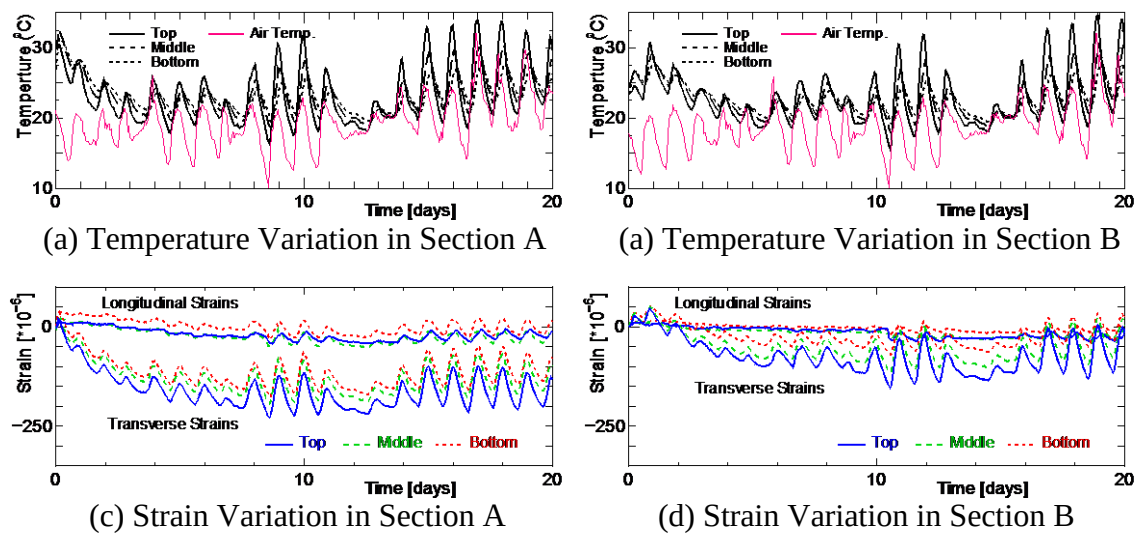


Figure 3. Measured Temperature and Strain Variations in Concrete Slab.

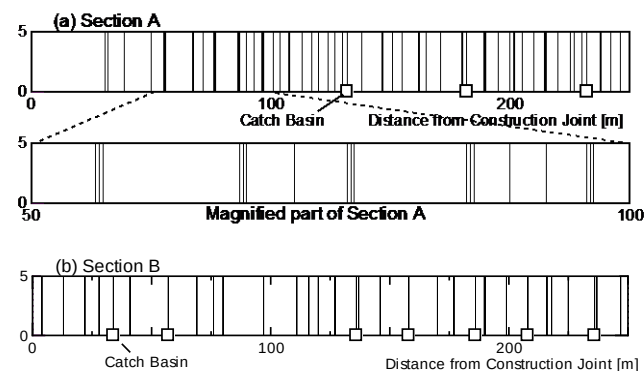


Figure 4. Observed Transverse Cracks.

Measurement Results. Figures 3(a) and 3(b) show the temperatures measured at the top, middle and bottom of the slab along with the air temperature. Since the air temperature at the concrete casting time in Section A was 20.3 °C that was higher than that of 16.5 °C in Section B, the concrete temperature in Section A was slightly higher than that in Section B. Figures 3(c) and 3 (d) show total strains measured at the top, middle and bottom of the slab. The strains are plotted as a relative value from the one at the highest concrete temperature. Since the slab was strongly restrained in the longitudinal direction, the longitudinal strains were small. On the other hand, the transverse strain was large, because the transverse restraint was relatively weak. The strain in Section A was larger than that in Section B, which was due to the higher temperature of concrete in Section A at the concrete casting time. This difference resulted in wider transverse crack spacing in Section B than that in Section A as shown in Figure 4, which shows the cracking on 29th August, 4.5 months after the casting. Three cracks with a narrow spacing of 30 cm to 40 cm were often observed at several places in Section A. Similar narrowly spaced cracks were reported in Japan and Belgium (Sato, et al. 1990 and Ren, et al. 2013).

Prediction of FCS. Free contraction strains (FCS) are required to calculate restraint strains. In the test pavement, however, FCSs were not able to be measured, because strains were measured with the strain sensors directly embedded in the slab. In this study the FCSs were estimated with the method proposed in Japanese standard specification for concrete structures, which was developed to predict cracking in mass concrete structures (JSCE Committee on Concrete Engineering 2013). FSC, ε_{FSC} , was assumed as

$$\varepsilon_{FSC} = \varepsilon_{sh} + \varepsilon_d + \varepsilon_T + \varepsilon_{cp}, \quad (1)$$

where, ε_{sh} =autogenous shrinkage strain, ε_d =dry shrinkage strain, ε_T =temperature strain, ε_{cp} =creep strain, which were estimated by equations as summarized in Table 3.

Table 3. Equations to Estimate Free Contraction Strains (JSCE Committee on Concrete Engineering 2013).

Autogenous Shrinkage	Dry Shrinkage	Temperature	Creep
$\varepsilon_{sh}(t_e) = \varepsilon_{sh,\infty} \left[1 - \exp \left\{ -a_{sh}(t_e - t_{e,sett})^{b_{sh}} \right\} \right]$ $t_e = \sum_{i=1}^n \Delta t_i \exp \left\{ 13.65 + \frac{4000}{273 + T_i} \right\}$ $\varepsilon_{sh,\infty} = 2350 \exp \{ -5.8(W/C) \}$ $+ 80[1 - \exp \{ -1.2 \times 10^{-6} (T_{max} - 20)^4 \}]$, $(20 < T_{max} < 70)$ $a_{sh} = 3.7 \exp \{ -6.8(W/C) \} \cdot (0.06T_{max} - 0.2)$ $b_{sh} = 0.25 \exp \{ 2.5(W/C) \} \cdot (-0.0075T_{max} + 1.15)$	$\varepsilon_d(t) = \frac{1 - RH/100}{1 - 60/100} \varepsilon_{d,\infty} (t - t_{d0})$ $\left(\frac{d}{100} \right)^2 \beta_d + (t - t_{d0})$ $\varepsilon_{d,\infty} = \left(1 + \frac{\beta_d}{182} \right) \varepsilon_{d0}$, $\beta_d = \frac{30}{\rho} \left(\frac{120}{-14 + 21(C/W)} - 0.7 \right)$	$\varepsilon_T = \alpha T$	$\varepsilon_{cp} = \phi \sigma_{cp} / E$
Note: t_e = maturity time (day), $t_{e,sett}$ = 0.3, Δt_i = time increment, T_i = concrete temperature at time, t_i , T_{max} = the maximum concrete temperature, W/C = water cement ratio, RH = relative humidity, t = age of concrete (day), $t_{d0} = 7$, $\varepsilon_{d0} = 780$ micro, ρ = concrete density, α = expansion coefficient of concrete, ϕ = creep coefficient of concrete, σ_{cp} = stress (MPa), E = elastic modulus of concrete			

Comparison of measured and calculated strains. Figures 5(a) and 5(b) show FCSs calculated by the equations in Table 2. The top strains are the highest in contraction and the bottom strains are the lowest. The strains in Section A are higher than those in Section B, which is similar tendency observed in the measurement. Figures 5(c) and 5(b) show variations of peak strains at midnight or early morning when the temperature is the lowest on a day. The difference between the measured and calculated strains represents the restraint strain that causes stress in the slab.

To look into the difference in degree of restraint between the base courses, a restraint ratio was defined as

$$\text{Restraint Ratio} = \left(1 - \frac{\text{Measured Strain}}{\text{Calculated Strain}} \right) \times 100 (\%). \quad (2)$$

If the measured and calculated strains are the same, the ratio is 0%, meaning no restraint. If the measured strain is 0, the ratio becomes 100%, meaning the complete restraint. Figure 6 shows variations of the restraint ratios of the peak strains at the top, middle and bottom of the slab. The ratio in Section A is around 20 % to 30 %, while the ratio in Section B ranges much wider: 30 % at the top to 70 % at the bottom of the slab. The low ratio in Section A may be resulted from stress relaxation of AIL.

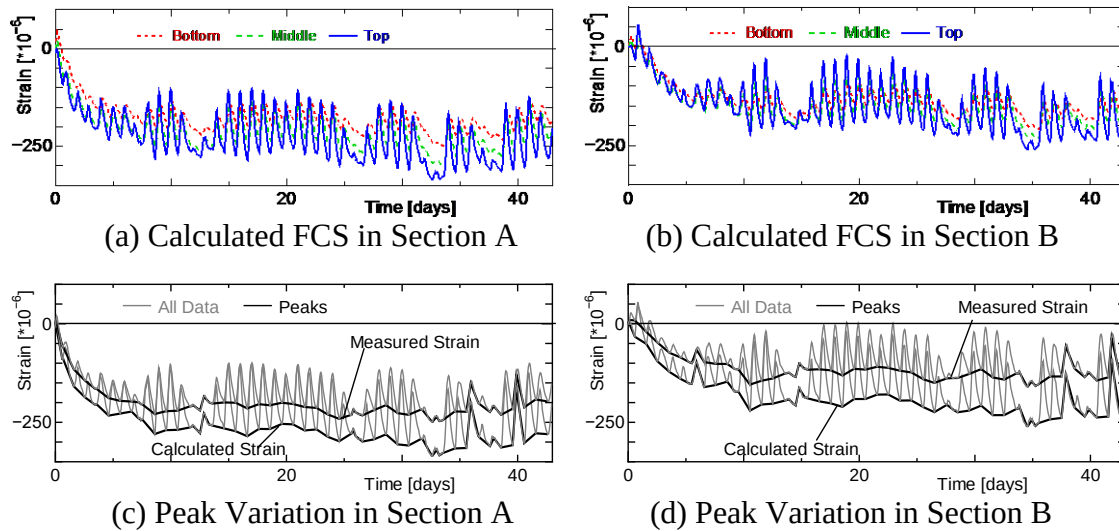


Figure 5. Calculated FCS and Its Peak Variations.

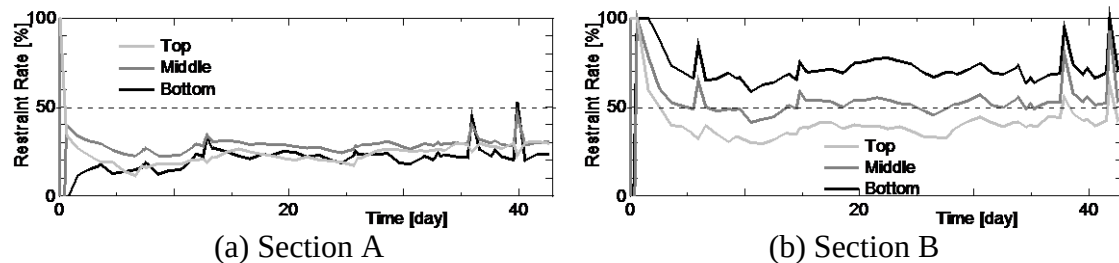


Figure 6. Variations of Restraint Ratio.

ANALYSIS METHOD

3DFEM Structural Model. In USA, a group of researchers at the University of Texas at Austin developed a prediction method based on the Finite Element Method, which allows pavement engineers to understand the complex CRCP cracking phenomena (Kadiyala and Zollinger 1993, Kim et al. 1998 and Kim et al. 2000). They have shown that interaction modeling between steel and concrete is crucial, and that effects of steel embedding position and its amount on spacing and width of crack are significant. Another 3DFE model was developed by the authors (Nishizawa, et al. 2001), which successfully simulated steel stress distributions measured in a test CRCP (Nishizawa, et al. 2013). However, the model was assumed to confine the slab transversely (2D plain strain model) and was not able to predict the stresses in that direction.

A structural model employed in this study is shown in Figure 7(a). Concrete slab, longitudinal steel bars, base course and subgrade were divided with solid elements. Considering the symmetry of the structure, a quarter region of pavement between two transverse cracks was meshed. Horizontal displacements of the steel bars, base course and subgrade were fixed but vertical displacements were free on four sides of the region. All displacements at the bottom of the region were fixed. Horizontal displacements of the concrete slab were fixed at the mid-span of cracks but are free at the crack edge.

Interface elements were inserted between steel bars and concrete and between concrete slab and base course as shown in Figure 7(b). Interaction between the steel bars and concrete was modeled by three-springs, of which relationship between force and displacement can be illustrated in Figure 7(c), where, k_0 is the initial stiffness of bond, f_0 is the bond strength, and Δu_0 is the critical amount of slip. In this study, it was assumed $k_0 = 3136 \text{ GPa/m}$, $f_0 = 6.0 \text{ MPa}$, $\Delta u_0 = 0.2 \text{ mm}$ based on our previous study (Nishizawa, et al. 2013).

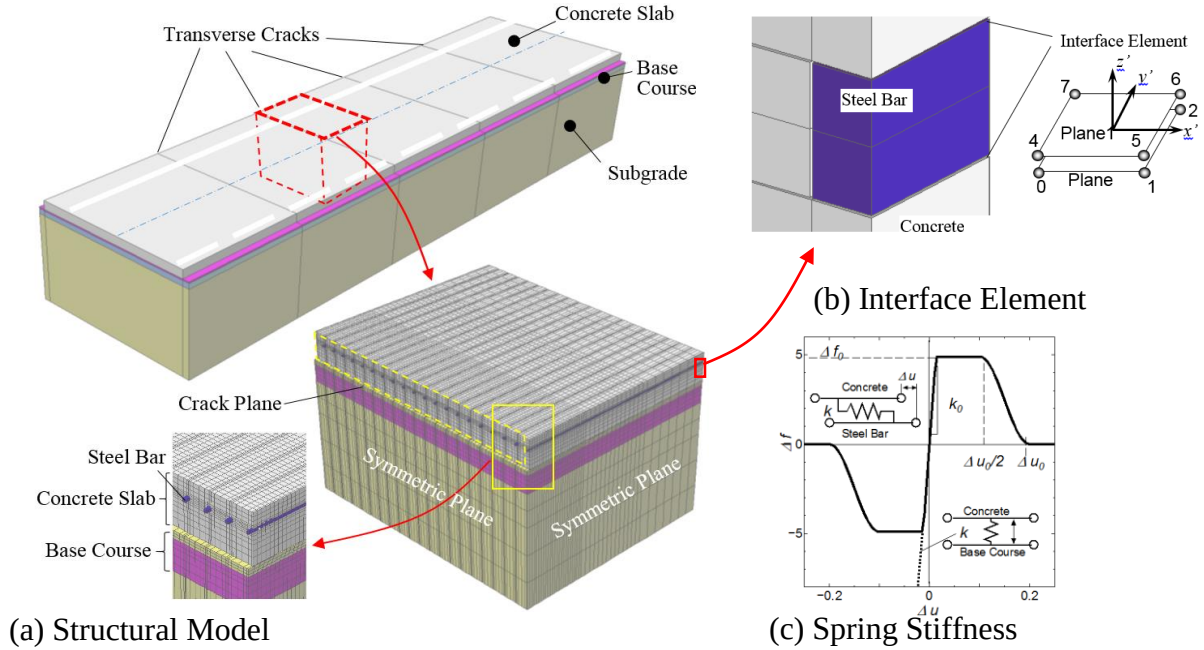


Figure 7. 3DFEM Structural Model.

Incremental Method. An incremental method was employed to simulate the change of concrete properties and the development of FCS in early age. The hardening process of concrete was divided into several time steps. It was assumed that the properties and strains of concrete and bond stiffness were constant during each step. At i step, total strain increment, $\{\Delta \varepsilon_i\}$ can be written as

$$\{\Delta \varepsilon_i\} = \{\Delta \varepsilon_{ei}\} + \{\Delta \varepsilon_{shi}\} + \{\Delta \varepsilon_{di}\} + \{\alpha \Delta T_i\} + \{\phi / E_i \Delta \sigma_i\}, \quad (3)$$

where, $\{\Delta \varepsilon_{ei}\}$ is the elastic strain increment, $\{\Delta \varepsilon_{shi}\}$ is the autogenous strain increment, $\{\Delta \varepsilon_{di}\}$ is the dry shrinkage strain increment, α is the thermal expansion coefficient, ΔT_i is the temperature increment, ϕ is the creep coefficient, σ_{cp} is the acting stress, and E_i is the Young's modulus of concrete. Stress increment becomes

$$\{\Delta \sigma_i\} = \frac{1}{1 + \phi_i} [D_i] (\{\Delta \varepsilon_i\} - \{\Delta \varepsilon_{shi}\} - \{\Delta \varepsilon_{di}\} - \{\alpha \Delta T_i\} - \{\phi / E_i \Delta \sigma_i\}), \quad (4)$$

where, $[D_i]$ is the elastic matrix at i step.

Load increment due to FCS increment can be calculated based on the virtual work theory as follows

$$\{\Delta F_i\} = \frac{1}{1+\phi_i} \int_V [B]^T [D_i] (\{\Delta \varepsilon_{shi}\} + \{\Delta \varepsilon_{di}\} + \{\alpha \Delta T_i\}) dV \quad (5)$$

where, $[B]$ is strain-displacement matrix. Incremental stiffness equation including the load increment due to FCS was solved and increments of displacements, strains and stresses were calculated. The increments of the responses were cumulated to obtain the current responses at i step.

Young's modulus, E , and tensile strength of concrete, f_t , were assumed to be functions of compressive strength of concrete f_c as proposed in the Japanese Standard Specification for Concrete Structures (JSCE Committee on Concrete Engineering 2013) as follows:

$$E = 6300 \cdot f_c^{0.45}, \quad f_t = 0.13 \cdot f_c^{0.45}. \quad (6)$$

f_c can be estimated by

$$f_c = (t_e - 0.42) \frac{46.6}{3.35 + 0.878(t_e - 0.42)}, \quad (7)$$

where, t_e is the effective age of concrete (day). Other material parameters are presented in Table 4.

Table 4. Material and Interface Properties Used in the Analysis.

Material / Interface	Property	Value
Concrete Slab	Elastic Modulus(MPa), E_c	Eq. (6)
	Poisson's Ratio, μ_c	0.2
	Density(kg/cm ³), γ_c	2.4
	Thermal Expansion Coef.(/°C), α_c	0.000012
Steel Bar	Elastic Modulus(MPa), E_s	210,000
	Poisson's Ratio, μ_s	0.3
	Density(kg/cm ³), γ_s	3
Concrete-Base Course Interface	Initial Stiffness(GPa/m), k_0	147
	Bond Strength(MPa), f_0	0.98
Asphalt Interlayer	Elastic Modulus(MPa), E_b	98
	Poisson's Ratio, μ_b	0.35
Cement Treated Base Course	Elastic Modulus(MPa), E_d	2940
	Poisson's Ratio, μ_d	0.35
Subgrade	Elastic Modulus(MPa), E_g	78
	Poisson's Ratio, μ_g	0.35

Validation of the Model

The 3DEF structural model developed in this study was validated by comparing the calculated responses with the ones observed in the test CRCP.

Figure 8 shows an example of entire deformation of the pavement and contour of concrete stress in the slab. The slab contracts due to FCSs, resulting in a large horizontal displacement at the crack edge. Twice amount of the horizontal displacement corresponds to the crack width. Since surface FCS is larger than bottom ones, the edges and corners of the slab deform upward and the steel bars and base course restrain the deformation producing the tensile stress at the top of the slab.

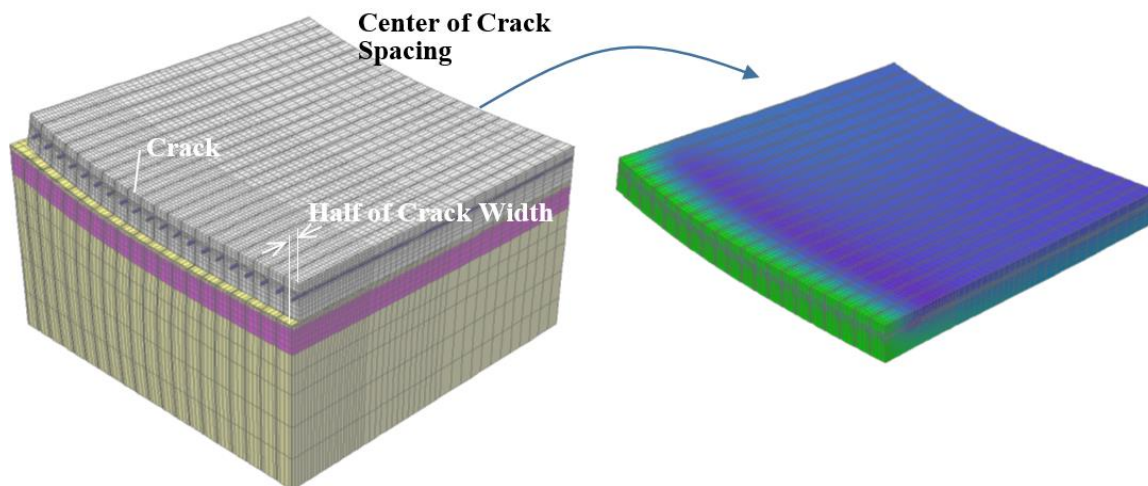


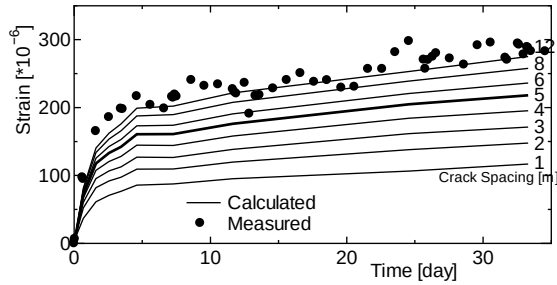
Figure 8. Example of Slab Deformation and Contour of Slab Stress.

Figure 9 shows measured and calculated restraint strain variations with the age of concrete. Since the slab is extended without transverse joint and strongly restrained in the longitudinal direction, the longitudinal restraint strains are much larger than the transverse ones. The crack spacing was varied from 1 m to 12 m in the calculation. The crack spacing at the measurement site was 5m in Section A and 4m in Section B as shown in Figure 2. These particular cases are plotted by thick lines in the figure. Comparing the calculated strain with a crack spacing of 5 m and the measured strains in Section A, the measured strains are larger than the calculated ones in the longitudinal direction, while in the transverse direction the calculated strains are very close to the measured ones. The comparison for a crack spacing of 4 m in Section B found the similar tendency.

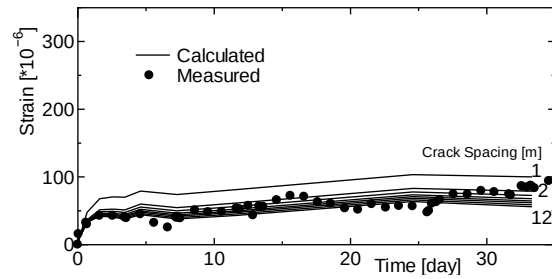
Figure 10 plots calculated tensile stresses at the top of the slab and tensile strength of concrete with the age of concrete. If the tensile stress exceeds the tensile strength, a crack would occur. The stresses in Section A are larger than the ones in Section B. To discuss the possibility of crack occurrence, a crack index (CI) is defined according to the Japanese specification (JSCE Committee on Concrete Engineering 2013) as follows:

$$CI = \frac{\text{Tensile Strength of Concrete}}{\text{Tensile Stress of Slab}} \quad (8)$$

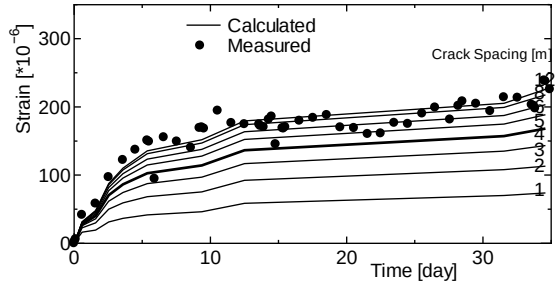
If CI is less than 1.0, the possibility is more than 50%. Figure 11 shows changes of CI with the age of concrete. Assuming that a crack occurs at a CI of 1.0, the minimum crack spacing can be determined 4 m in Section A and 5 m in Section B from the figures. The age of the crack occurrence would be less than one day in Section A and round 11 days in Section B. The predicted crack spacing in Section A is less than the observed 5m, while in Section B it is longer than the observed 4m.



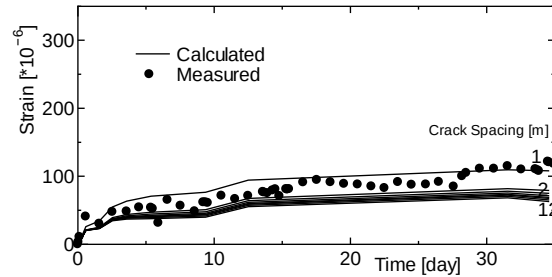
(a) Longitudinal Strain in Section A



(b) Transverse Strain in Section A

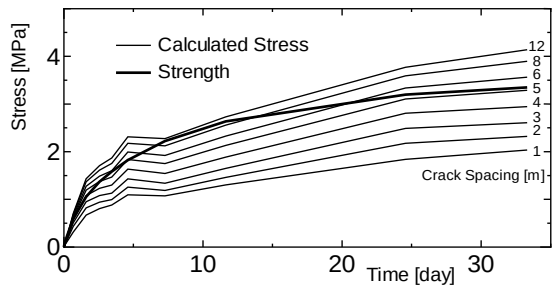


(c) Longitudinal Strain in Section B

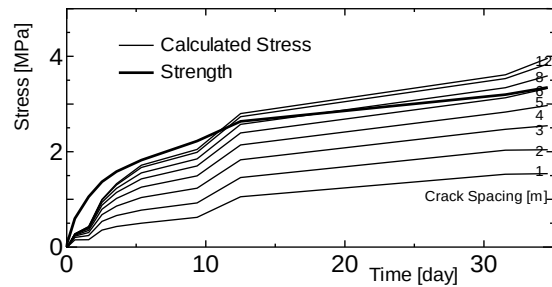


(d) Transverse Strain in Section B

Figure 9. Comparison of Measured and Calculated Restraint Strains.

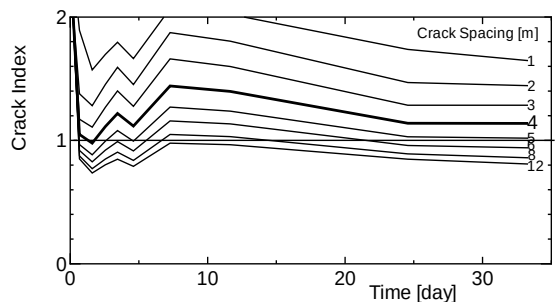


(a) Top Tensile Stress in Section A

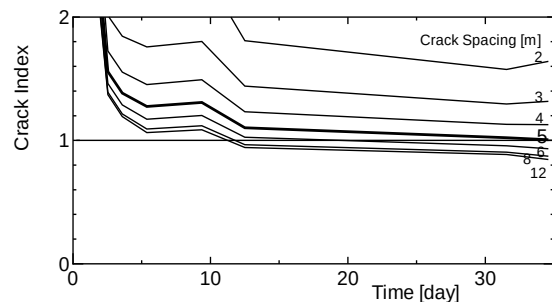


(b) Top Tensile Stress in Section B

Figure 10. Tensile Stress Development at the Top of Concrete Slab.



(c) Crack Index in Section A



(d) Crack Index in Section B

Figure 11. T Crack Index of Concrete at Top of Concrete Slab.

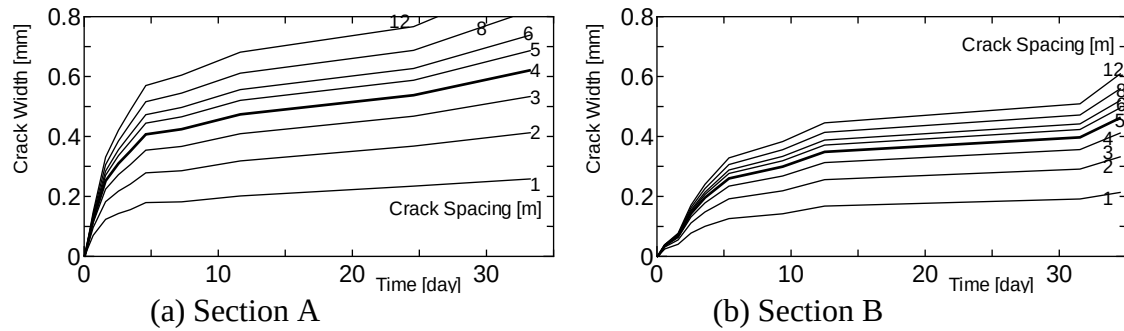


Figure 12. Crack Width at Top of Concrete Slab.

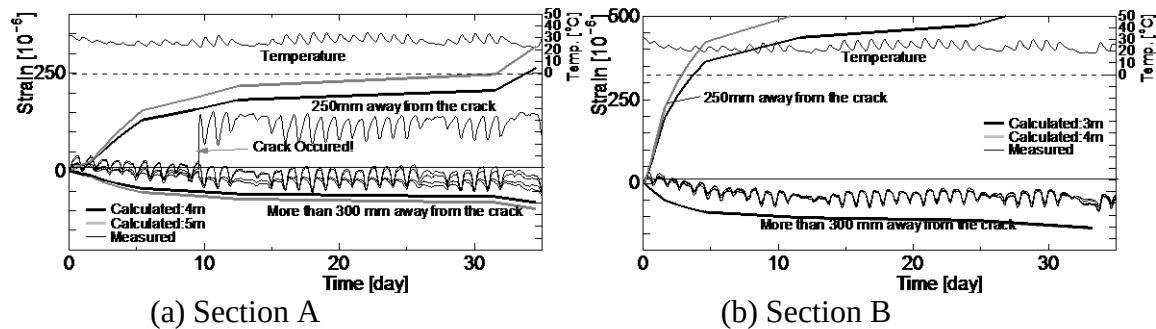


Figure 13. Comparisons of Measured and Calculated Steel Strains.

Figure 12 shows calculated changes of crack width. Assuming that the crack spacing is 4 m in section A and 5 m in Section B, the crack width at one month after casting concrete can be obtained from the figure as 0.62 mm in Section A and 0.50 mm in Section B. These values are the ones at midnight or early morning when the concrete temperature is lowest. If the width is observed in day time, they become narrower. The actual crack widths measured at day time on 1st June were 0.15mm in Section A and 0.35 mm in Section B. The daily differences in temperature were 9.7 $^{\circ}\text{C}$, 3.4 $^{\circ}\text{C}$ and -0.2 $^{\circ}\text{C}$ at the top, middle and bottom of the slab, respectively, on that day. These temperature differences were input into 3DFE model and calculated daily crack width difference at the top of the slab, which was 0.15 mm with a 4 m crack spacing in Section A and 0.14 mm with a 5 m crack spacing in Section B. Therefore the crack width calculated for the day time would be $0.62 - 0.15 = 0.47\text{mm}$ in Section A and $0.50 - 0.14 = 0.36\text{mm}$ in Section B. The calculated crack width is much wider in Section A than the measured one, while it is close to the measured in Section B.

Figure 13 shows comparisons of calculated and measured steel bar strains. The measured strains fluctuate with daily temperature change and slightly decreased at first and converged at a certain level later. The calculated and measured strain variations depend on the distance from the crack. The strain is tension near the crack while it becomes compression a distance away from the crack. The strain increases as the crack spacing increases. In Section B, the measured strain near the crack rapidly increased at age of 10 days, which might be caused by cracking near the measurement point. After the cracking, the measured strain becomes close to the one calculated near the crack.

From the comparisons mentioned above, it is concluded that the 3DFE model is able to reasonably predict the cracking phenomena in early age concrete slab considering restraints of reinforcement steel bar and base course.

Effects of Inserting AIL between the Slab and CTB

Using the 3DFE model developed in this study, effects of inserting AIL on the cracking of CRCP was examined. In this case, FCSs obtained in Section A were used to compare the responses of both sections under the identical condition.

Cracking Spacing and Width. Figure 14(a) shows the change of top concrete stress in Section B under the condition of Section A. The calculated stress is larger than that in Section A. *CI* estimated from Figure 14(a) was shown in Figure 14(b). From the figure, the minimum crack spacing can be determined to be 4 m, which is the same as that of Section A. The crack width at a crack spacing of 4 m obtained from Figure 14(c) is 0.66 mm, which is wider than that of Section A.

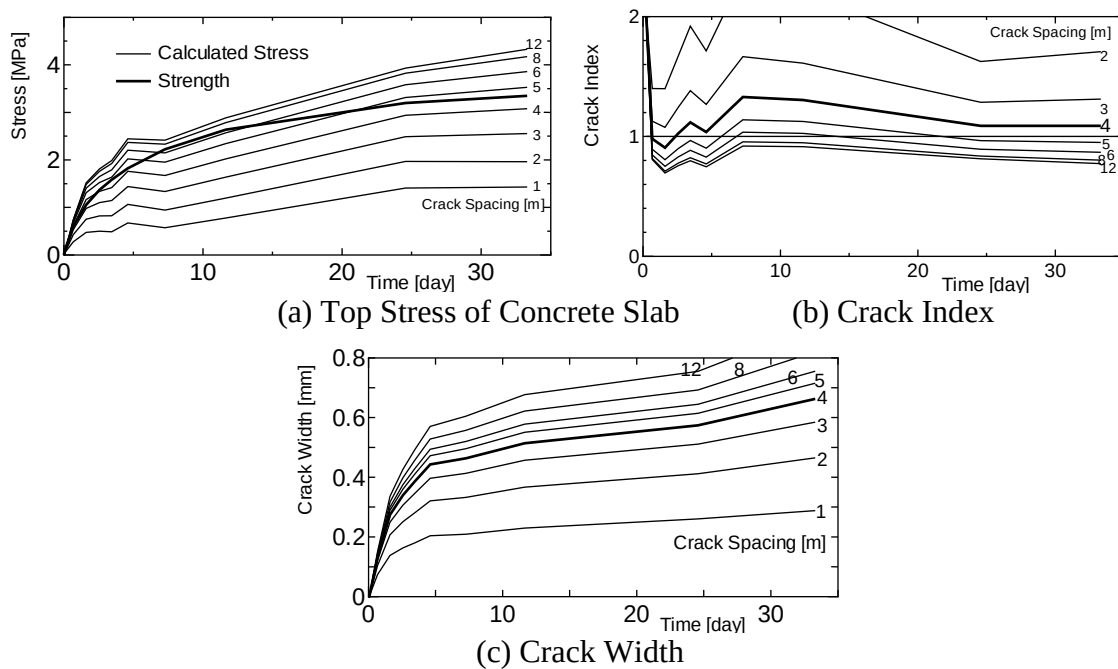


Figure 14. Crack Spacing and Width of Section B under Condition of Section A.

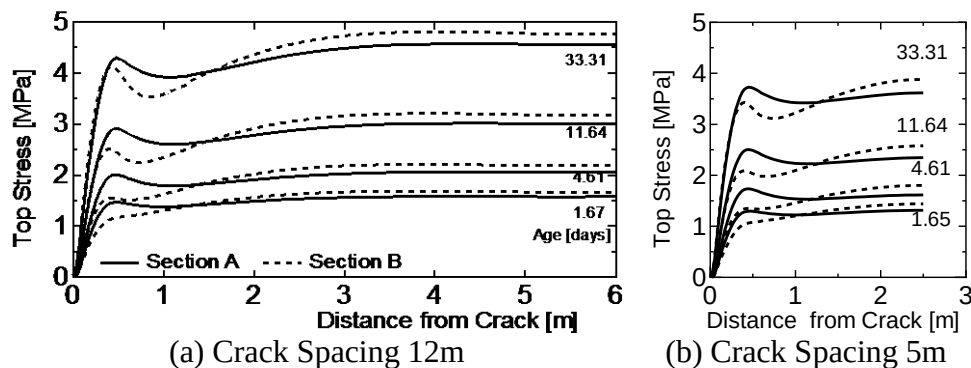


Figure 15. Top Tensile Stress Distributions in Longitudinal Direction.

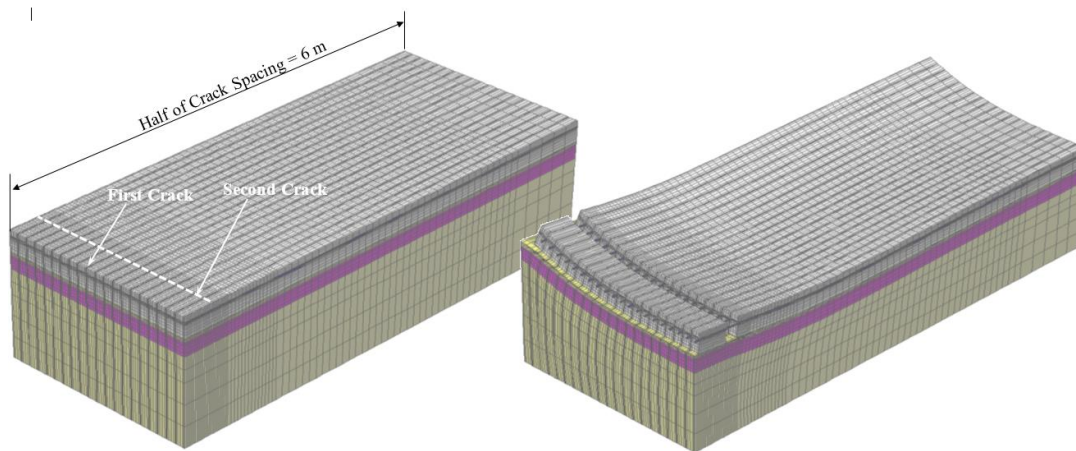


Figure 16. Modeling of Secondary Crack and Its Effect on Deformation.

Secondary Cracking. Narrowly spaced cracks with a spacing of 30 to 40 cm were often observed in Section A as mentioned earlier. The cause of the closely spaced cracks will be investigated in the following.

Figure 15 shows longitudinal stress distributions at the top of the slab along the center line of the slab. The stress rapidly increases near the crack and reaches to its peak at the midspan between cracks. A small peak appears near the crack at early days. This peak value increases with age and become larger than the one at the midspan in Section A, while the peak value in Section B is less than the one at the midspan. This means that crack would occur earlier near crack than that at the midspan in Section A, resulting in narrowly spaced cracks as observed in Section A (Figure 4). Sato et al. (1990) also reported “secondary” cracks that appeared 0.3m to 0.5 m away from “primary” cracks. The crack due to the stress peak at the midspan is called the first crack and the crack due to the peak near the first crack is called the second crack in this study.

To look at what happens after the second crack occurs, a simulation with the second crack was conducted using the 3DFE model as shown in Figure 16. The interface elements with very small spring constants were inserted 40 cm away from the first crack and the FCS was input into the model. Figure 16 also shows the entire deformation of the pavement and the second crack opening.

Figure 17 shows longitudinal stress distributions after the second crack occurs. The stress decreases significantly at the second crack position. Although the small peak appears again near the second crack, its value becomes less than that at the midspan. Therefore, after the second crack, the next crack would occur at the midspan, resulting in the three narrowly spaced cracks as observed in Section A.

Figure 18 shows crack width profiles through the slab depth. Before the second crack occurs, the profile of Section A is uniform through the depth, which might be the results of unbonding at the interface between the steel bar and concrete as well as between the slab and base course near the crack. In Section B, the width is larger at the top than that at the bottom. After the second crack occurs, widths of all cracks decrease by a half.

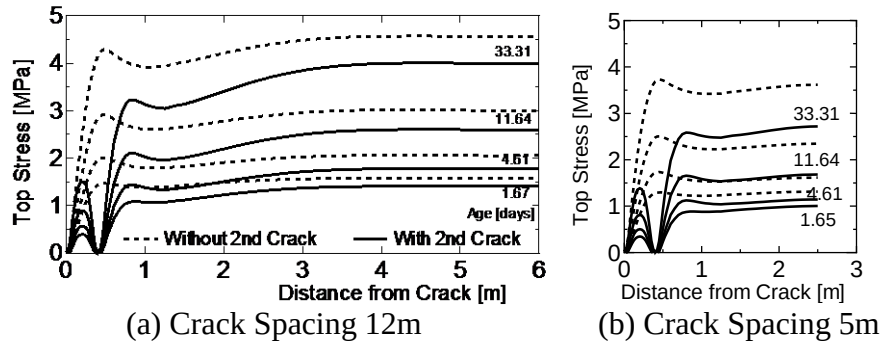


Figure 17. Top Stress Distributions in Longitudinal Direction after Secondary Crack Occurred.

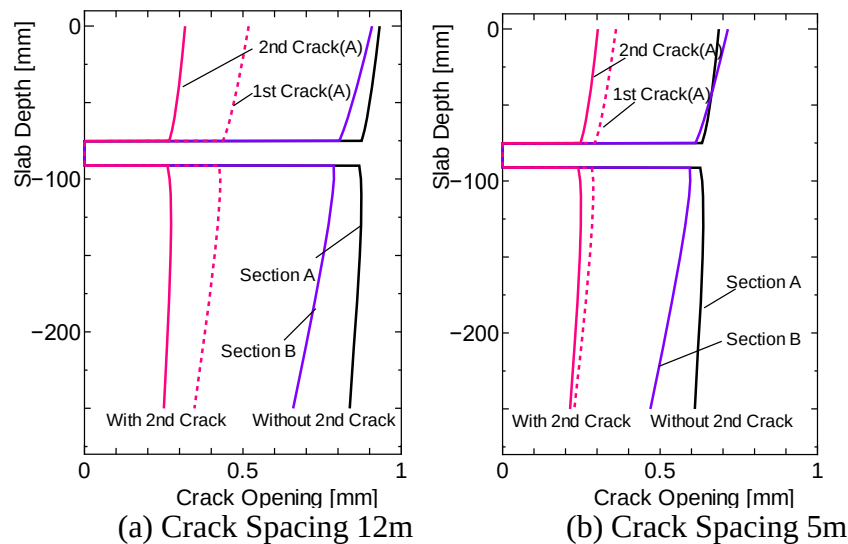


Figure 18. Crack Width throughout Slab Depth with and without Secondary Crack.

Conclusions

In this study, the effect of AIL on early cracking in CRCP was investigated based on the crack observations and strain and temperature measurements on a test CRCP and numerical simulations with 3DFE models that was validated by comparing the computed responses with the measured ones on the test CRCP. Obtained results can be summarized as follows:

1. From the strain and temperature measurements, it was found that AIL reduced the restraint to the contraction of concrete slab.
2. It was observed that the average crack spacing was shorter in AIL section than that in CTB section. This is probably due to the difference in the temperature at concrete casting. Three narrowly spaced cracks appeared at several places in AIL section.
3. From the 3DFE simulations with the same temperature condition, the crack spacing is slightly shorter in CTB section than that in AIL section and the crack width is a little wider in CTB section than that in AIL section.
4. A small peak of tensile stress at the top of the slab appeared near the crack edge. The peak stress was larger than that at the midspan between transverse cracks in AIL

section, suggesting a possibility of the second crack. After the second crack occurred, the peak tensile stress significantly decreased and the next crack location would be the midspan.

5. The second crack width was about a half of the first crack. The first crack width also decreased a half after the second crack occurred.

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