Full Scale Thermal Stress Simulation of Multiple Span Steel Box Girder Bridge Evaluating Early Age Transverse Cracking Risk of Durable RC Deck Slab

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Abstract

The present research aimed at evaluating early age thermal cracking risk of durable RC slabs incorporating slag cement and expansive additive on multiple span steel box girder bridges utilizing full-scale 3D FEM simulation. First, laboratory investigations were conducted to calibrate the material models of durable concrete. Second, the material models were utilized in several member level FEM models and the simulation procedure was verified regarding early age volume changes calibrating parameters for expansion energy and reduction factors for creep. Third, thermal and volumetric changes in RC slab were monitored and the simulation procedure was further validated in structural level utilizing full-scale FEM model of the real bridge. The simulated maximum tensile stress along bridge axis in RC slab signify the risk of early age transverse cracking where the accumulated stepping construction stress is comparatively large. The effectiveness of expansive additive in reducing the risk of transverse cracking is revealed from the simulation. However, parametric studies of the validated model indicate that the RC slab on the permanent form of seven span steel box girder bridge is vulnerable to early age thermal cracking regardless of ambient conditions and placing temperatures when coefficient of thermal expansion of concrete is larger than 6 $\times$ 10$^{-6}$/\textdegree C.

1. Introduction

Transportation infrastructures in cold and snowy regions in Japan are susceptible to sever deterioration due to the combined actions of freezing and thawing, chloride attack from deicing agent, alkali silica reaction, cracking, fatigue, and so on. Moreover, the occurrence of early age thermal and shrinkage cracks in RC deck slabs restrained by girders provide rapid routes for harmful agents aggravating deterioration. In order to ensure the high durability performance of RC decks of bridges along Revival Roads in cold and snowy Tohoku region, a durable concrete design accounting multiple protection countermeasures against deterioration is formulated incorporating low water-to-cement ratio, blast furnace slag and/or fly ash, expansive additive, anti-corrosion rebar and increased entrained air (Tanaka et al. 2017). It is to be noted that such a highly durable concrete specification may conversely increase the risk of early age cracking in RC slabs owing to the influence of large amount of cement and mineral admixtures (Ishida and Iwaki 2017). Eventually, early age transverse cracks appeared after completion of 28 days special curing period of the durable RC slabs of several multiple span continuous steel box girder bridges such as Shinkesen Ohashi Bridge and Kosano Viaduct in Iwate Prefecture and Koori Viaduct in Fukushima Prefecture along Revival Roads. In case of Shinkesen Ohashi RC slab crack width reached 0.2 mm or more after two years of construction.

Several investigations were conducted in the past regarding transverse cracking of bridge decks (PCA 1970; Cady et al. 1971; Purvis et al. 1995; Schmitt and Darwin 1995; Krauss and Rogalla 1996; Eppers et al. 1998; Altoubat and Lange 2000; ACI 2001; Saadeghvaziri and Hadidi 2002; Frosch et al. 2003; Xi et al. 2003). Based on the field investigations, several structural factors influencing transverse cracking were identified: (1) steel girder bridges had more cracks than prestressed concrete bridges, (2) continuous span bridges showed more cracking than simply supported span bridges and (3) transverse crack intensity increased as the span length increased (PCA 1970; Cady et al. 1971). Again, environmental factors such as variations of temperature and moisture in early age concrete were recognized as major causes of transverse cracking (Mohsen 1999; Ramey et al. 1997; Springenschmid 1998). Purvis et al. (1995) revealed that transverse cracks intersected coarse aggregates indicating the crack occurrence in hardened concrete was likely caused by drying and thermal shrinkage. Eppers et al. (1998) identified the dominant design fac-
tors and material parameters affecting transverse cracking. Design factors were longitudinal restraint, deck thickness, and top transverse bar size. Most affecting material parameters were cement content, aggregate type and quantity, and air content. Accordingly, Eppers et al. (1998) recommended reducing restraints by using bridge expansion joints, constructing simply supported spans, increasing girder spacing, and providing fewer shear connectors. Krauss and Rogalla (1996) reported that stiffer decks and larger amounts of reinforcement are effective in reducing cracks. Again, Frosh and Bice (2006) evaluated that the amount and spacing of reinforcement in the deck influenced the extent of cracking. They recommended providing adequate amount of reinforcement to prevent localized yielding at cracks. In 2011, Choi et al. assessed the early age temperature and relative humidity distribution in a composite bridge utilizing heat transfer and moisture distribution models. Again, Tanaka et al. (2017) and Ishida et al. (2017) conducted numerical simulations of early age deformation and cracking of multiple protection durable RC deck utilizing multiscale multi-chemo-physical integrated analysis (Maekawa et al. 2008). However, early age thermal stress combined with construction stress causing early age transverse cracking was not explicitly addressed. Zerin et al. (2018) performed thermal stress simulation of full-scale multiple span bridge; however, detailed modeling assumptions and quantitative evaluation of influential factors causing transverse cracking were not explicated in detail. Hence it is important to establish a full-scale FEM model of multiple span girder bridges to conduct thermal stress simulation of RC slab incorporating construction stresses. The validated FEM model can be effectively utilized in parametric studies for quantitative evaluation of risk and the influential factors causing transverse cracking.

Against this background, the present research aimed at evaluating the cracking risk of the durable RC slab on continuous multiple span steel box girder Shinkesen Ohashi Bridge along the Revival Roads. A three leveled systematic full-scale 3D FEM analysis scheme was followed in the numerical simulation of early age thermal and volumetric stresses in the corresponding RC slab. First, material level laboratory investigations were conducted to calibrate the material models of the durable concrete. Second, the material models were given inputs in the small-scale member level specimen FEM models and the simulation procedure was verified for early age volume changes. In this level, parameters were calibrated for expansion energy and reduction factors for effective Young’s modulus considering creep. Third, thermal and volumetric changes were monitored in the real RC slab on Shinkesen Ohashi steel box girder bridge and the simulation scheme was further validated in structural level. The effect of external stresses induced by the stepping construction of continuous RC slab was considered in the investigation as well. The cracking risk and the influential factors such as restrained expansion produced by expansive additive, coefficient of thermal expansion (CTE) of concrete, ambient and concrete placement temperature are evaluated based on the structural level simulation and extensive parametric studies utilizing the validated FEM bridge model.

2. Cracking scenario of RC slab on multiple span steel box girder bridge

Construction and cracking scenario of RC slab on seven continuous span steel box girder Shinkesen Ohashi Bridge (Figs. 1, 2 and 3) is discussed in the following sections.

2.1 Application of highly durable concrete in Shinkesen Ohashi RC slab

The seven continuous span steel box girder Shinkesen Ohashi Bridge (length = 438 m) along Revival Roads in Tohoku region is about three kilometres from the estuary of Kesengawa River in Iwate Prefecture. The multiple protection highly durable concrete incorporating blast furnace slag cement, low water to binder ratio (W/B = 0.44), expansive additive (20 kg/m^3), entrained air (6%) and anti-corrosion rebar were applied in the RC slab during June to July in 2016. Three layers of special curing mats were applied on the RC slab to control thermal changes and prevent evaporation. Twenty-two days of wet curing was followed by initial seven days sealed curing of the RC slab.

2.2 Accumulation of stresses in RC slab induced by stepping construction

Systematic stepping construction method was applied in
the construction of continuous RC slab considering 13 concrete placement lots to minimize adverse structural tensile stresses. Concrete placement order review analysis was performed based on the non-linear stress analysis incorporating non-linear material models (Maekawa et al. 2003; CEB-FIP 1990) and layered fiber model for steel girder and RC slab (MHPS Engineering Co. Ltd. 2006). In the layered model, RC slab sections are con-

![Graph](image1)

**Fig. 4** Stepping construction stress of concrete in different lots of RC slab on the final day of concrete placement of Shinkesen Ohashi Bridge.

![Graph](image2)

**Fig. 5** Schematic diagram showing the effect of stepping construction stress on thermal cracking.

![Diagram](image3)

**Fig. 6** Cross-section details and location of plywood forms in Shinkesen Ohashi RC slab.

![Diagram](image4)

**Fig. 7** Crack patterns in Lot 1 of Shinkesen Ohashi RC slab according to the dates of observation.
sidered as layers and steel girders as beams. Calculation of stress is performed only in the longitudinal direction (along bridge axis) at the event of placing of each concrete lot. **Figure 4** represents the estimated tensile strength and cumulative stepping construction stress of each concrete lot along the bridge axis on steel girder G2 (Fig. 6) at final concrete placement day of Lot 13. The cumulative tensile stresses in concrete were comparatively large in Lot 1, Lot 2 and Lot 3. Minor transvers cracks were observed in Lot 1, Lot 2, Lot 3, Lot 8 and Lot 11 primarily after removal of the curing mats at 28th days of curing. It is anticipated that the cracks might have been induced by imposed stepping construction stresses (illustrated in Figs. 4 and 5). Efflorescence observed in the bottom surface of the slab confirmed that some cracks were penetrating (Fig. 3).

### 2.3 Initiation and propagation of cracks in Shinkesen Ohashi RC slab

Initially, primary transverse cracks were generated in the RC slab on permanent plywood forms along main girders (Zone P in Figs. 6 and 7). Further, the cracks were propagated towards the cantilever part as well as the central axis of the bridge across the RC slab on temporary plywood forms (Zone T). The maximum crack width was 0.08 mm in Lot 1 observed after completion of wet curing. However, some cracks were reported to be widened (0.2 mm or more) along Zone T observed from the exposed bottom surface of the bridge deck after two years of construction.

### 3. Application of JCI guidelines in thermal stress simulation of RC deck slab

Guidelines for Control of Cracking of Mass Concrete (JCI 2016a) recommends verifying thermal cracking probabilities utilizing 3D FEM thermal stress simulation of concrete structures undergoing large temperature drops. The multiple protection durable concrete incorporates high cement contents contributing to considerable temperature drops. Hence, present investigation focuses on evaluating early age thermal cracking risk of highly durable RC slabs based on JCI 2016 guidelines.

#### 3.1 Time dependent material models of concrete

Guidelines for Control of Cracking of Mass Concrete (JCI 2016a) incorporates time dependent material models of concrete for adiabatic temperature rise, compressive strength, tensile strength, Young’s modulus, chemical expansion and shrinkage considering the temperature history of the material incorporating effective material age of concrete as Eq. (1).

$$ t_e = \sum_{i=1}^{n} \left\{ \Delta t_i \cdot \exp \left[ 13.65 - \frac{4000}{273 + T(\Delta t_i)/T_e} \right] \right\} \quad (1) $$

where, $t_e$: effective material age (day) considering the effect of temperature history, $\Delta t_i$: period of constant temperature in concrete (day), $T(\Delta t_i)$: concrete temperature for $\Delta t_i$ (ºC), $T_e$: 1ºC (value that makes temperature non-dimensional).

Further, the influence of creep and micro-cracking is considered in a simple way by calculating the effective Young’s modulus implying reduction factors at temperature increasing and decreasing period of concrete. Additionally, the provision of heat transfer coefficients of concrete structures depending on formwork type, form removal time, curing method and duration, ambient temperature and wind velocity has simplified the FEM thermal and structural stress analysis procedure.

#### 3.2 FEM thermal stress simulation software

FEM thermal stress analysis tool JCMAC3 has been utilized in the present research. In JCMAC3, three dimensional linear hexahedral isoparametric heat generating and non-heat generating solid elements are adopted for material models (JCI 2016b). Reinforcing bars are modeled with one dimensional embedded ETRUSS elements. However, the effect of reinforcing bar is considered in the form of steel ratio in the analysis. Further, two dimensional heat elements are applied as the heat transfer surfaces upon the 3D solid elements. JCMAC3 consists of four analysis modules: temperature module, moisture transfer module, stress module and crack analysis module. The temperature distribution according to each time is calculated in temperature analysis module. In stress module, normal stress and cracking index defined by the ratio of tensile strength of concrete to the tensile stress are calculated based on simultaneous linear analysis. As cracking is not considered in the stress module, perfect bond is implied at the steel concrete interface. In crack analysis module, smeared crack model is applied where crack width is expressed by crack equivalent strain and stress release after crack initiation is considered. However, the present research intends to evaluate the early age thermal stress and cracking index of the restrained RC slab during the wet curing period utilizing stress module. Hence, the moisture transfer (drying shrinkage) and crack analysis are not pursued in the present scope.

### 4. Three leveled systematic FEM thermal stress simulation

The 3D FEM simulation procedure followed three leveled systematic analytical schemes comprising material, member, and full-scale structural levels to simulate early age restrained volume changes in the RC slab on multiple span steel box girder Shinkesen Ohashi Bridge. The FEM bridge model is validated with respect to monitored temperature, volumetric strains and cracking scenario ensuring appropriate thermal and structural boundary conditions along with adopting precise modeling assumptions. The simulation procedure is illustrated in Fig. 8 and in following sub-sections.
4.1 Level 1: Laboratory investigations and calibration of material models

In Level 1, laboratory investigations were performed to calibrate the material models of Shinkesen Ohashi RC slab concrete. The detailed concrete mix design is summarized in Table 1 where, C = type B blast furnace slag cement, Ex = expansive additive, S1 = limestone crushed sand, S2 = land sand, G1 = coarse aggregate from crushed lime stone (Ryushin area), G2 = coarse aggregate from crushed lime stone (Waga area), Ad = admixture for achieving workability and AE = air entraining admixture. Time dependent compressive strength and Young’s modulus, setting time and free autogenous shrinkage without expansive additive were measured both under 20°C and 40°C constant room temperature confirming the effect of temperature on concrete properties (Zerin et al. 2018). Adiabatic temperature rise of concrete was measured to calibrate the parameters of adiabatic temperature rise model (JCI 2016a). Average value of the coefficient of thermal expansion of the concrete was measured as $8.4 \times 10^{-6}/°C$.

Table 1. Concrete mix proportion of Shinkesen Ohashi Bridge RC slab.

| W/B % | s/a % | Air % | W | C | Ex | S1 | S2 | G1 | G2 | Ad | AE |
|-------|-------|-------|---|---|----|----|----|----|----|----|----|
| 44    | 37.8  | 6     | 160| 344| 20 | 334| 328| 558| 554| 2.18| 0.1|

4.1.1 Adiabatic temperature rise model

JCI 2016a adiabatic temperature rise equation (Eq. (2)) is regressed from the multicomponent model (Shima et al. 2007) for hydration heat of cement. The parameters are formulated as functions of unit cement content and placing temperature. Time dependent compressive strength and Young’s modulus of concrete were measured. The parameters were calibrated based on the laboratory investigation as $Q_\infty = 56°C$, $r_{AT} = 1.0$ and $S_{AT} = 0.85$ and $t_{0,AT} = 0$ day as concrete placing time (Fig. 9(a)). The origin of the time scale is kept same throughout the current analysis scheme.

\[
Q(t) = Q_\infty \left[ 1 - \exp\left(-r_{AT} (t-t_{0,AT})^{S_{AT}}\right) \right] \tag{2}
\]

where, $Q(t)$: adiabatic temperature rise at age of $t$ days ($°C$), $Q_\infty$: ultimate adiabatic temperature rise ($°C$) as a function of unit cement content and initial concrete placement temperature, $r_{AT}$ and $S_{AT}$: parameters representing the rate of adiabatic temperature rise as functions of concrete placement temperature and $t_{0,AT}$: age of starting of adiabatic temperature rise.

4.1.2 Strength development model

Time dependent compressive strength of concrete can be determined by Eq. (3) where concrete age, temperature dependence, cement type and water-to-cement ratio are considered. Splitting tensile strength and Young’s modulus of concrete are determined by Eqs. (4) and (5) respectively (JCI 2016a).

\[
f'_{ct}(t) = \frac{t - S_f}{a + b \cdot (t - S_f)} f'_{c}(t_0) \tag{3}
\]

\[
f'_{st}(t) = \frac{t - S_f}{a + b \cdot (t - S_f)} f'_{s}(t_0) \tag{4}
\]

\[
f'_{e}(t) = \frac{t - S_f}{a + b \cdot (t - S_f)} f'_{e}(t_0) \tag{5}
\]
where, \( t_e \): effective material age (day), \( t_c \): strength control age of concrete cured under water at 20°C (day): 28 days, \( f_c'(t_c) \): compressive strength of concrete at \( t_c \) (N/mm²), \( a \) and \( b \) are the calibrated parameters representing strength development, \( S_t \): temperature adjusted age corresponding to initiation of hardening (day), \( f_c(t_e) \): compressive strength of concrete at \( t_e \) (N/mm²). Moreover, \( f_a(t_e) \): splitting tensile strength of concrete at \( t_e \) (N/mm²), \( E_s(t_e) \): Young’s modulus of concrete at \( t_e \) (N/mm²) and \( C_1, C_2, C_3, C_4 \) = parameters.

Eqs. (3) and (5) showed approximately good agreement with the test results corresponding to both 20°C and 40°C curing temperature (Figs. 9(b) and 9(c)). Parameters in Eq. (3) were calibrated as \( a = 4.0 \) and \( b = 0.9 \). Parameters in Eq. (5) were kept as \( C_3 = 6500 \) and \( C_4 = 0.45 \) according to the guidelines (JCI 2016a). Further, Eq. (4) was considered as the tensile strength development model for the corresponding concrete where \( C_1 = 0.13 \) and \( C_2 = 0.85 \).

4.1.3 Autogenous shrinkage model

The autogenous shrinkage strain in concrete can be determined by Eq. (6a) considering age, temperature dependence, type of cement and water-to-cement ratio (JCI 2016a). According to the test results, the effect of temperature seemed insignificant compared to the effect of water-to-binder ratio on the maximum value of autogenous shrinkage strain of Shinkesen Ohashi RC slab concrete (Fig. 9(d)). Hence, the constants in autogenous shrinkage model were calibrated based on the measured autogenous shrinkage in 20°C and 40°C curing temperature (Fig. 9(d)) to obtain acceptable agreement with the test results (Eqs. (6)b and (6)c).

\[
e_{ag} = -\beta e_{aut} \times (1 - \exp(-a \times (t_e - t_c)^b)) 
\]

\[
e_{aut} = 2800 \times \exp(-5.8 \times (W/C)) + e_{aut} 
\]

\[
e_{aut} = 10 \times (1 - \exp(-1.2 \times 10^{-5} \times (T_{max} - 20)^b)) 
\]

where, \( \beta \): coefficient indicates the influence of cement and admixture (\( \beta = 1 \) for blast furnace slag cement), \( t_e \): effective material age, \( t_c \): initial setting time, \( e'_{aut} \): final value of autogenous shrinkage, \( a, b \): coefficients expressing the progressive characteristics of autogenous shrinkage, \( W/C \): water-to-cement ratio, \( e_{aut} \): autogenous shrinkage contributed by maximum temperature and \( T_{max} \): maximum concrete temperature.

4.1.4 Setting time

Setting time tests (JIS A 6204) for specimens cured at 20°C temperature confirmed initial and final setting time as 0.27 and 0.39 day, respectively. Corresponding initial and final setting time for specimens cured at 40°C temperature was 0.12 and 0.16 day. In the verification process, measurement data and simulation results for strain were compared from the initial setting time corresponding to curing temperature ignoring the unpredictable thermal expansion before hardening of concrete.

4.2 Level 2: Member level verification and calibration of parameters

4.2.1 JCI-S-009-2012 expansion strain specimen model

In Level 2, FEM thermal stress simulation were conducted to simulate several member level specimens such as JCI-S-009-2012 restrained expansion cylindrical specimen and small-scale RC slab specimen. Material models calibrated from the test results in Level 1 were given as inputs. Two important parameters dependent on structural restrained conditions such as total expansion energy (Tanabe and Ishikawa 2017) and the reduction factors calculating effective Young’s modulus of concrete considering early age creep were calibrated in Level 2.

(1) Laboratory investigation

Restained expansion strain was measured according to JCI-S-009-2012 using cylindrical thin walled tinplated steel molds φ100 × 200 mm (Figs. 10(a) and 10(b)). A strain gauge was attached horizontally with glue at the center of the outer surface of the tinplated steel mold to measure the circumferential strain. The specimens were kept in sealed condition.

(2) Finite element model

The 1/8th of the symmetric specimen was modeled with 3D linear isoparametric solid elements (Fig. 10(c)).

\[
f_s(t_e) = C_1 \times f_c'(t_e) 
\]

\[
E_s(t_e) = C_1 \times f_c'(t_e) 
\]
Since the concrete inside the steel mold produces radial compressive stress and circumferential tensile stress against the mold due to the expansion of concrete, perfect bond was considered between the concrete and the mold. 2D heat element was applied upon the outer surface of the steel mold and the top surface of the concrete. Moreover, mesh sensitivity analysis was performed to confirm the appropriate mesh sizes of the model.

(3) Thermal Analysis
Adiabatic temperature rise model (Eq. (2)) with calibrated parameters was applied in thermal analysis both for 20°C and 40°C curing conditions. Concrete placement temperatures were 20°C and 35°C for 20°C and 40°C curing conditions, respectively. Since the specimens were kept in the curing room with constant temperature, there was no influence of the natural wind flow. Accordingly, heat transfer coefficient was set as 6.0 W/m²°C for the steel mold and 8.0 W/m²°C for top surface of the concrete sealed by a cellophane plastic sheet. The input properties for thermal analysis i.e. heat conductivity, specific heat, density of concrete, steel mold and steel reinforcing bars are described in Table 2.

(4) Stress analysis and calibration of parameters
JCI 2016 time dependent models for compressive strength, tensile strength, and Young’s modulus (Eqs. (3), (4) and (5)) with calibrated parameters were applied in stress analysis. The free autogenous shrinkage obtained from laboratory investigations at 20°C and 40°C room temperature was given as inputs.

(a) Calibration of expansion strain energy model
In the present research expansion strain model based on the total energy conservation hypothesis (Eq. (7)) proposed by Tanabe and Ishikawa (2017) is applied to simulate the expansion strain of concrete based on the test investigation results.

\[
U(t_e) = U_\infty (1 - \exp(-a(t_e - t_0)^b))
\]

where, \(U(t_e):\) total energy at effective concrete age \(t_e\) (N/mm²), \(U_\infty:\) ultimate value of the total energy (N/mm²), \(a\) and \(b:\) coefficient indicating the influence of the type of cement on the progressive characteristics of total energy, \(t_0: \) effective material age at the beginning of expansion. Parameters for expansion strain energy model (Eq. (7)) were calibrated as \(U_\infty = 100 \times 10^{-6}\) N/mm², \(a = 1.5, b = 1\) based on the average of the experimental results.

(b) Confirmation of reduction factors for effective Young’s modulus
The creep effect is considered by calculating the effective Young’s modulus as a product of Young’s modulus of concrete and a reduction factor as shown in Eq. (8).

\[
E_e(t_e) = \varphi(t_e) \times E(t_e)
\]

where, \(E(t_e):\) effective Young’s modulus at \(t_e\) (N/mm²),
\( \phi(t_e) \): reduction factor, \( \phi(t_{max}) = 0.42 \) until the effective material age of maximum temperature and \( \phi(t_{max+1}) = 0.65 \) after one day of the effective material age of maximum temperature. \( \phi(t_e) \) is linearly interpolated between these two effective material ages as recommended in the guidelines (JCI 2016a). These reduction factors were confirmed to be applicable in the present simulation utilizing JCI-S-009-2012 FEM model.

(5) Verification of FEM Model
Simulated strains showed substantially good agreement with the measured average expansion strain both for 20°C and 40°C curing temperature (Figs. 10(d) and 10(e)) validating the applicability of the calibrated parameters in this range of concrete temperature.

### 4.2.2 Simulation of small RC slab specimen model
In another member level, the restrained expansion and shrinkage behaviour of small RC slab specimen (400 × 400 × 200 mm) was experimentally investigated (Fig.11(a)). The specimen was kept at 20°C constant room temperature until 28 days under sealed condition at bottom and side surfaces of the specimen. The top surface was kept under wet curing to reproduce the actual curing condition in practice. The 1/8th symmetric FEM model was configured with 3D isoparametric solid elements for concrete and 1D embedded truss elements for reinforcing bars (Fig.11(b)). Heat transfer coefficient was set as 6.0 W/m²°C applying 2D heat transfer surface elements for sealed and wet curing in 20°C temperature. Simulation of reinforcing steel bar strain exhibited good agreement with the corresponding measured strain (Fig.11(c)). Slight discrepancy between simulation and measurement of restrained concrete strain at the centre of the specimen is observed as because the applied maximum expansion energy parameter \( (U_e = 100 \times 10^6 N/mm^2) \) was calibrated based on average expansion strain obtained from the JCI-S-009-2012 specimen.

### 4.3 Level 3: Real structural level simulation and verification
The FEM systematic simulation procedure validated in member level restrained specimens was applied in real structural level simulation of seven continuous span steel box girder Shinkesen Ohashi Bridge model (Figs. 12(a) to 12(g)).

#### 4.3.1 Instrumented monitoring of Shinkesen Ohashi Bridge
(1) Locations of strain gauges and thermo-couples Embedded flexible strain gauges for concrete with temperature measuring function (gage length = 100 mm, stiffness = 40 N/mm², capacity = ±5000 × 10⁻⁶) were installed in longitudinal (X-axis), transverse (Y-axis) and vertical (Z-axis) directions to measure concrete strain and temperature in Lot-8 on Pier-3 at location A where RC slab is directly connected to the steel girder (Figs. 12(a), 12(b) and 12(d)). Temperatures and strains were also monitored in longitudinal and transverse direction at location B in Lot-8 (Zone-T on temporary plywood form) (Figs. 12(c) and 12(d)). Further, location C and D in Figs. 12(b) and 12(d) were additionally accounted for temperature, strain, and stress simulation. Location C is at the center of RC slab along Zone-P on permanent plywood form. Additionally, girder temperature and ambient temperature were measured using thermo-couples attached with the box girder underneath the bridge deck (location G in Fig. 12(d)).

(2) Duration of measurement
Measurement of concrete temperatures at A and B along with girder and ambient temperatures at G were initiated on 22nd June 2016 at 8:40 am with the start of Lot-8 concrete placement. The measurement of strain was started at 0.28 day after the initiation of concrete placement. Measured temperature and strain until 28 days after concrete placing were utilized in validating bridge model during the wet curing period.

(3) Significances of measured strains
The longitudinal strain along bridge axis is considered under the highest external restraints due to the continuous composite connections of the main girders and RC slab. The generation of tensile stress along the bridge axis is the key factor for generation of transverse cracks. Hence, the longitudinal strains were mainly utilized for validating the corresponding FEM bridge model and evaluating the cracking risk. Further, transverse strain in RC slab is governed by the transverse reinforcement, non-uniform shape of the deck, permanent plywood forms and the composite connections of girders. Thus,
the measured transverse strains are conveniently utilized to verify several modeling assumptions. Nevertheless, hardening point of concrete is determined based on the vertical strain history with respect to temperature rise under the least structural restraint as illustrated in Fig. 12(h). Accordingly, the initial setting time at 0.28 day is defined as the hardening point or the starting point of strength development in stress simulation.
4.3.2 Full-scale FEM modeling of Shinkesen Ohashi Bridge

(1) Modeling of bridge components and structural boundary conditions

Full scale Shinkesen Ohashi Bridge FEM model consists of RC deck slab, main box girders, secondary girders, transverse girders, permanent and temporary plywood forms, and rubber bearing (Figs. 12(b) to 12(g)). Approximately 1/4th of the bridge is modeled with symmetric structural and thermal boundary conditions considering multiple spans. The unique modeling approaches were adopted for lead plugged rubber bearings and plywood forms for accurate simulation of thermal and volumetric behaviour of the bridge model. Thermal and mechanical properties of different component materials are summarized in Table 3. The modeling aspects of different components of Shinkesen Ohashi Bridge are described as below.

(a) RC slab and mesh discretization

The newly placed Lot-8 slab concrete was modeled with 3D hexahedral isoparametric heat generating solid elements. Previously placed hardened Lot-1 concrete was modeled as the 3D non-heat generating solid elements. As shown in Fig. 12(e), the thickness of slab elements at top and bottom heat transfer and heat conduction surfaces were finer (5 mm) compared to those at the center of slab to ensure accurate thermal analysis. Temperature simulation was performed at nodal points. Conversely, normal stress is calculated as the average of the stresses at two integral points in each element. Hence, the aspect ratio of the elements at the central part of RC slab along thickness direction was kept 1.2 at location A and D. The maximum aspect ratio was 3.2 at locations B and C to obtain satisfactory results regarding stress analysis.

(b) Steel reinforcement

The embedded steel reinforcements in RC slab along longitudinal and transverse directions were modeled with 1D embedded truss elements. The minimum and maximum steel reinforcement ratio of the RC slab were 1.32% and 2.4% along Zone-P (on permanent form in Fig. 12(c)) in the mid-span and on the supports, respectively.

(c) Steel girders

Main box girders, transverse girders, and secondary girders in Figs. 12(b) and 12(d) are modeled with non-heat generating 3D solid elements. The connection between the steel girder and the RC slab is considered as perfect bond representing the effect of shear connectors and slab anchors.

(d) Permanent and temporary plywood forms

Permanent plywood forms between RC slab and main girders and temporary plywood forms underneath the RC slab are modeled as non-heat generating 3D solid elements considering perfect bond (Figs. 12(c), 12(d) and 12(f)) with appropriate thermal and mechanical properties. As the Young’s modulus of plywood is significantly smaller than that of concrete after hardening, it is supposed that the modeling of plywood forms considering perfect bond will not generate any substantial stress in RC slab.

(e) Rubber bearing

Multilayer rubber bearings with alternately piled up rubber and steel plates along with lead plugs installed at bridge supports were modeled with three consecutive sections (Fig. 12(g)). The top steel plate is connected to the bottom surface of the box girder. The bottom steel plate is restrained in longitudinal, transverse, and vertical axes at the bottom surface reproducing the real connections of the rubber bearing to the substructure (bridge pier). The middle section is modeled with several layers of elements incorporating the lower bound of equivalent composite properties based on the rule of mixture for composite materials (Alger 1997) as Eq. (9).

\[ M_c = \left( \frac{f_R}{M_R} + \frac{f_S}{M_S} + \frac{f_L}{M_L} \right) \]

where, \( M_c \) = equivalent composite material property, \( M_R \), \( M_S \), and \( M_L \) represent the thermal and mechanical properties of rubber sheet, steel plate and lead plug, respectively. Similarly, \( f_R \), \( f_S \), and \( f_L \) represent the volume fractions of rubber sheet, steel plate and lead plug, respectively.

(2) Thermal boundary conditions

(a) Heat transfer surfaces

Heat transfer surfaces were defined by applying 2D heat elements upon 3D solid elements of the FEM model. Heat transfer coefficients were determined based on the parametric studies considering site exposures, curing method, curing duration, formwork type and wind flow.
conditions. As shown in Fig. 13(a), heat transfer coefficient of the top surface of RC slab with three layered special curing mats is defined as 6.0 W/m²/°C. Exterior surface of the girders were considered to be exposed to wind flow, hence the corresponding heat transfer coefficient was defined as 14.0 W/m²/°C. As the wind flow was obstructed inside the box girder, heat transfer coefficient of the interior surface of the box girder was considered 12.0 W/m²/°C. The temporary plywood form underneath the RC slab was precisely modeled considering appropriate thermal properties. The corresponding heat transfer coefficient was defined as 12.0 W/m²/°C based on the parametric study.

(b) Input ambient temperature
The ambient and the girder temperatures were recorded inside the main girder at location G (Fig. 12(d)). The radiation effect caused by direct sunlight was observed in the measured RC slab concrete temperature in comparison to the girder temperature (Fig. 13(b)). The average peak temperature difference was 7°C between the girder temperature and the RC slab temperature. Considering the radiation effect in the daytime, measured ambient temperature was calibrated to apply on the top surface of RC slab ensuring realistic temperature simulation (Fig. 13(c)). However, the measured ambient temperature was applied on other heat transfer surfaces such as exterior and interior surfaces of the girders as well as the bottom surfaces of the temporary plywood forms replicating real site conditions (Fig. 13(d)).

(3) Thermal and Stress analysis inputs
Thermal analysis of the FEM bridge model was performed based on the adiabatic temperature rise model of concrete inputting parameters calibrated in Level 1. Thermal analysis is followed by the stress analysis. Hence, material models calibrated in Level 1 and Level 2 were given as inputs in stress module. Other input properties for thermal analysis (heat conductivity, specific heat, density and initial temperature) and for stress analysis (Young’s modulus, Poisson’s ratio and CTE corresponding to concrete, steel, plywood forms and composite rubber bearings) are described in Table 3.

(4) Starting point of simulation
The measurement data prior to hardening of concrete incorporate substantially large strains and comparatively smaller stresses because of the plastic nature and a very large coefficient of thermal expansion. Therefore, the starting point of the strain measurement was defined as the starting point of hardening that could be equivalent to the initial setting time observed in the vertical strain history (Fig. 12(h)). Although thermal and stress analyses are initiated considering the starting time of concrete placement, strength development, autogenous shrinkage and expansion strain in stress simulation was considered from 0.28 day of material age. The simulated thermal strain until the material age of 0.28 day is deducted from the simulated total strain in verification process.

(5) Validations of Shinkesen Ohashi Bridge FEM model
The full-scale FEM model was validated based on the instrumented monitoring and the simulation results at different locations affecting generation and propagation of cracks in the RC slab as discussed below:

![Heat Transfer Surfaces and Heat Transfer Coefficients](image-url)
(a) Temperature history
Temperature simulations exhibited satisfactory agreement with the measured temperature history of steel girder at location G and in concrete at locations A and B as illustrated in Figs. 14(a), 14(b) and 14(c). It is to be noted that temperature simulation at nodal points was carried out by calculating the average temperature between consecutive time steps allowing insignificant differences between measured and simulation results.

(b) Longitudinal strain in RC slab
Simulation of longitudinal strain in RC slab at locations A and B showed acceptable agreement with the corresponding measured strains (Figs. 14(d) and 14(g)). The simulated total strain exhibit inconsequential difference in comparison to the measured strain at the initial stage of hardening of concrete during the period of temperature rise. Possibly, it is because of the slight disagreement in simulation of adiabatic temperature rise and the restrained thermal strain applying the constant coefficient of thermal expansion of concrete. However, the simulation of longitudinal strains in RC slab satisfactorily validate the FEM bridge model reproducing the external restraints by the modeling of steel girders, longitudinal reinforcing bars, plywood forms and composite rubber bearing.

(c) Transverse strain in RC slab
The simulation of transverse strains in RC slab at locations A and B exhibit good agreement with the corresponding measured strains (Figs. 14(e) and 14(h)) validating the modeling aspects such as transverse reinforcement, non-uniform shape of RC slab, plywood forms and composite rubber bearings.

(d) Vertical strain in RC slab
Thermal and chemical expansion at the initial stage during temperature rise of concrete was the largest along the vertical axis of the RC slab because of the negligible structural restraints. However, the simulated total vertical strains at locations A and B were smaller than the corresponding measured strains (Fig. 14(f)). It is because of the limitation of the applied expansion strain model (Tanabe and Ishikawa 2016) ineffectual of simulating the free expansion of concrete in absence of external restraints. Furthermore, the input of calibrated autogenous shrinkage based on JCI 2016 guidelines (Eqs. (6)a, (6)b and (6)c) is predicted to be larger than the autogenous shrinkage in the real RC slab cured in wet condition. These are the reasons why simulated vertical strains in RC slab exhibited discrepancy compared to the measured strains.
(e) Location of maximum temperature and stress vulnerable to cracking of RC slab
Simulation of temperatures at A, B, C and D revealed that the temperature rise in concrete was maximum at location C in Zone P of RC slab due to the low heat conductivity of the plywood forms permanently placed between RC slab and steel girders (Figs. 15(a) and 15(b)). Simulation results of longitudinal and transverse strains in these locations of RC slab were also compared (Figs. 15(c) and 15(d)) which exhibited the similar trends. Further, simulation of stresses at locations A, B, C and D identified that longitudinal and transverse stresses were largest at location C. The occurrences of maximum tensile stress in longitudinal (2.3 MPa) and transverse (1.3 MPa) directions in the RC slab seemed to be influenced by the occurrence of maximum tem-

Fig.15(a) Cross-section of RC slab, temperature distribution and stress contour along X-axis.

Fig.15(b) Simulation of temperatures in different locations of RC slab.

Fig.15(c) Simulation of longitudinal concrete strains in RC slab.

Fig.15(d) Simulation of transverse concrete strains in RC slab.

Fig.15(e) Simulation of longitudinal stresses in RC slab along bridge axis.

Fig.15(f) Simulation of stresses in RC slab along transverse direction bridge.

Fig.15(g) Cracking Index of RC slab regarding different locations.
perature in concrete at location C in Zone P (Figs. 15(a), 15(e) and 15(f)). Again, the cracking index (CI) was considerably low at location C (lowest CI = 1.2 at 26th day of material age) increasing the risk of cracking compared to those at locations A, B, and D as illustrated in Fig. 15(g).

According to Fig. 4, maximum stepping construction stress was estimated as 1.0 MPa in Lot-1. Cracking index is supposed to be decreased significantly (CI < 1.0) when 1.0 MPa stepping construction stress is imposed to the RC slab. Therefore, early age thermal stress simulation of Shinkansen Ohashi Bridge FEM model confirmed that transverse cracks were initiated primarily along Zone P on permanent plywood form as observed in Fig. 7 owing to the large tensile stresses caused by thermal and volumetric changes of the RC slab where cumulative tensile stresses due to the stepping construction were large. However, the minimum reinforcement ratio (1.32%) used in RC slab is supposed to be adequate to prevent local yielding of reinforcement controlling excessive widening of cracks. Eventually, the multiple span steel girder full-scale FEM bridge model has been successfully verified in structural level regarding temperature, strains and occurrence of cracks through incorporating calibrated material models and parameters, appropriate structural and thermal boundary conditions as well as the precise modeling of the individual components of the bridge.

5. Influential factors affecting early age transverse cracking

The established bridge model was successfully utilized to identify the effect of expansive additive, coefficient of thermal expansion (CTE) of concrete, ambient temperature, concrete placement temperature and stepping construction stress on the occurrence of early age transverse cracking. The cracking risk was evaluated based on the simulated tensile stress and cracking index (CI) of the RC slab obtained from the parametric studies described in the following subsections.

5.1 Effect of expansive additive in reducing cracking risk

FEM simulation of the validated bridge model without expansive additive (expansion strain energy model is not applied) represent that tensile stress along bridge axis can be as large as 3.5 MPa and at location C and 2.25 MPa at location A, B and D in RC slab increasing the risk of cracking (Fig. 16(a)). Figures 16(b) and 16(c) represent that tensile stresses in RC slab can be reduced by around 1.0 MPa utilizing 20 kg/m³ expansive additive. In Fig. 16(d), cracking index less than or around 1.0 at different locations of RC slab without expansive additive indicate the high risk of thermal cracking even though the stepping construction stress is not added.
5.2 Effect of coefficient of thermal expansion of concrete

Parametric studies considering different coefficient of thermal expansion (CTE) of concrete proved that CTE larger than $6 \times 10^{-6}/\text{°C}$ significantly increase the risk of cracking (Figs. 17(a), 17(b) and 17(c)). Further, it is revealed that the effect of CTE in increasing the tensile stress is more prominent at location C (the location of maximum temperature) compared to location A.

5.3 Effect of ambient and concrete placement temperatures

Parametric studies are conducted to evaluate the influences of ambient temperature and initial concrete placement temperature considering varying CTE of concrete. Three ambient conditions such as 10°C, 20°C and 30°C constant temperatures are applied while concrete placing temperatures and CTEs are varied correspondingly (Figs. 18(a), 18(b) and 18(c)) regarding location C exhibit maximum temperature and lowest cracking index). It is revealed that multiple span steel box girder bridge RC slab is vulnerable to early age thermal cracks (low cracking indices) regardless of ambient condition, placing temperature when CTE is larger than $6 \times 10^{-6}/\text{°C}$ because of the higher structural restraints against volume changes even though expansive additive has been incorporated.

5.4 Effect of stepping construction of RC slab on transverse cracking

Simulation of temperature, strain and stress were verified with respect to the measurement data in concrete Lot 8. However, the maximum cumulative tensile stress was estimated as 1.0 MPa occurred in Lot 1 imposed by stepping construction (Fig. 4). Stepping construction stress can be altered depending on the concrete placement order arrangement. In this perspective, cracking indices are estimated assuming the maximum cumulative tensile stress as 1.0 MPa and added to the simulated thermal stress to visualize the severest condition. In Fig. 19, minimum cracking indices are shifted in vicinity to 1.0 due to the imposed construction stress, even if CTE is as small as $6 \times 10^{-6}/\text{°C}$ in different ambient and concrete placement temperatures. It indicates that the risk of cracking in multiple span continuous steel girder bridge increases due to the accumulated tensile stresses induced by the stepping construction.

6. Conclusions

The conclusions of the present research are summarized based on the three aspects described as below:

A. Three leveled thermal stress simulation validating full-scale FEM bridge model

The full-scale 3D FEM model of multiple span steel box
A girder bridge is successfully established and verified with respect to monitored temperature, volumetric strains and cracking scenario of the RC slab. The time dependent concrete strength and autogenous shrinkage models calibrated in material level, expansion energy parameters and reduction factors for creep established in member level and detailed modeling of the individual bridge components with appropriate structural and thermal boundary conditions in real structural level are the key factors for obtaining good agreements in FEM thermal stress simulation.

B. Evaluation of cracking mechanism in Shinkesen Ohashi durable RC slab
(1) Simulation of temperature utilizing the validated model identify that the temperature rise in concrete seems maximum in RC slab upon permanent forms owing to the low heat conductivity of plywood existing between RC slab and box girders.
(2) Simulation of stresses reveal that longitudinal and transverse stresses due to the restrained thermal and volumetric changes are the largest in RC slab on permanent forms along main girders. Accordingly, estimated cracking indices are considerably low (C.I. = 1.2) intensifying the risk of cracking in RC slab on permanent forms as compared to the other locations.
(3) Eventually, early age thermal stress simulation of the FEM bridge model confirms that transverse cracks were initiated primarily along main girders on permanent form induced by large tensile stresses in consequence of the restrained thermal and volumetric changes of RC slab where cumulative tensile stress due to the stepping construction was large. However, the applied minimum reinforcement ratio (1.32%) in the RC slab is considered adequate in controlling the crack width.

C. Parametric studies confirming significant influential factors for transverse cracking
(1) Cracking indices of the RC slab without using expansive additive are significantly low (CI ≤ 1.0) indicating the high vulnerability to early age thermal cracking. Simulation results imply that tensile stress in RC slab is reduced by around 1.0 MPa utilizing 20 kg/m³ expansive additive. Therefore, it can be conferred that the multiple protection durable concrete utilizing expansive additive has effectively reduced the extent and risk of cracking.
(2) Coefficient of thermal expansion larger than $6 \times 10^{-6}/^\circ C$ significantly increases the risk of cracking in RC slabs. It is revealed that the effect of CTE in increasing the tensile stress is more prominent in RC slab zone upon permanent plywood forms.
(3) Parametric studies based on different ambient temperatures, concrete placing temperatures and varying CTEs of concrete signify that multiple span steel box girder bridge RC slab especially along main girders upon permanent form is vulnerable to early age thermal cracks regardless of ambient conditions and placing temperatures when CTE is larger than $6 \times 10^{-6}/^\circ C$ even if expansive additive is incorporated.
(4) It is confirmed that the accumulated tensile stresses induced by the stepping construction of RC slab significantly increase the risk of cracking in case of multiple span continuous steel girder bridges regardless of ambient conditions, concrete placing temperature and CTE of concrete.

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