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Evaluation of critical crack width for reinforcement corrosion in RC member based on numerical simulation of transport of chloride ions in concrete

Nguyen Thi Hien* and Takumi Shimomura

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Abstract: Chloride-induced reinforcement corrosion in reinforced concrete structures exposed to marine environment is one of the most important factors affecting the durability of structures. To minimize the effect of cracks on the deterioration of RC structures, current design codes often limit the crack width. However, recent investigations indicated that condition of steel-concrete interface is a more essential criterion related to reinforcement corrosion. The purpose of this study is to clarify the effect of various conditions including cover thickness, water-cement ratio, environment action and interfacial void on limitation of crack width. The validity of crack width limitation in design codes is discussed based on analytical results.

Keywords: crack width limitation, chloride ingress, bleeding, interfacial void, flexural crack.

1. Introduction

Corrosion of reinforcing bars in concrete due to chloride penetration is one of the main causes of deterioration of reinforced concrete structures under marine environment. Reduction of chloride penetration is, therefore, crucial to the design and construction of concrete structures under load and environmental action. RC structures are in general allowed to have flexural cracks under the service load. The effect of flexural crack on chloride penetration into concrete has been studied by many researchers [1-5]. It has been confirmed that chloride ingress into concrete increases with increasing surface crack width.

The influence of crack width on corrosion initiation has been investigated by many researchers [6-9]. It has been clarified that the risk of corrosion initiation increases with increasing crack width. Therefore, to ensure design service life of RC structure, crack width should not exceed the threshold value to avoid corrosion risk of embedded steel due to chloride. In several studies [10-18], efforts were made to determine the threshold value of crack width. Table 1 summarizes literatures which proposed the critical crack width for reinforcement corrosion. It was found that critical crack width is not a unique value depending on the definition of initiation of corrosion. While a value of 0.1 mm was reported by Schiessl [11], O’neil [14] found a value of 0.4 mm and it was expressed as a function of cover thickness by Yachida [17].

Table 1 – Limitation of crack width for reinforcement corrosion

<table>
<thead>
<tr>
<th>Researcher</th>
<th>Limitation of crack width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shiessl., Rehm, G [10,11]</td>
<td>0.1</td>
</tr>
<tr>
<td>Shalon, R [12]</td>
<td>0.15</td>
</tr>
<tr>
<td>Okada.K and Miyagawa .T. [13]</td>
<td>0.1-0.2</td>
</tr>
<tr>
<td>O’neil.E.F [14]</td>
<td>0.4</td>
</tr>
<tr>
<td>Kamiyama [15]</td>
<td>0.01</td>
</tr>
<tr>
<td>Maruyama and Seki [16]</td>
<td>0.2</td>
</tr>
<tr>
<td>Yachida [17]</td>
<td>0.0065c-0.14c</td>
</tr>
<tr>
<td>Kamiyama [15]</td>
<td>0.1</td>
</tr>
</tbody>
</table>

NOTE: c – cover thickness (mm)

Steel-concrete interface in RC member such as girder bridge, slab bridge, and highway pavements, is generally damaged due to increased tensile stress or cyclic load. The defect of steel-concrete interface is also caused by concrete bleeding resulting in the formation of interfacial void around reinforcing bar. The influence of defect of steel-concrete interface on corrosion initiation has been confirmed in a few studies [19-26]. Mohammed [19] found that the presence of gaps/voids at the steel-concrete interface causes the complete loss of passivity with the
presence of chloride and, therefore, it is necessary to make concrete without interfacial void at steel-concrete interface to ensure long-term durability of RC structures. Savija [20] found that the defect of steel-concrete interface was an important factor in chloride ingress and subsequent reinforcement corrosion. A. Castel [21] showed that the quality of steel-concrete interface is greatly dominant in prediction of the initiation of corrosion in reinforced concrete members. T.A Soylev [22] concluded that damage of steel-concrete interface affected directly to the corrosion rate. Interaction between flexural crack and interfacial void has been investigated in the author’s previous studies [27, 28]. The authors found that chloride ingress into concrete with flexural crack was promoted with presence of interfacial void, therefore, defect around reinforcement due to bleeding should be avoided to make crack width control in RC member effective.

Several current structural codes [29-34] specify the allowable crack width with respect to environmental conditions and cover thickness. Threshold value of crack width varies among the codes due to different criteria. For example, in the standard specification (2002) for concrete structures by Japan Society of Civil Engineering (JSCE) [29], limitation of crack width is experimentally determined considering various influencing factors on corrosion as concrete cover, environmental condition, design life, type of reinforcement, in addition to chloride concentration. In CEB-FIP Model Code 1990 [30], crack width limitation is determined to ensure that steel bar will not be depassivated during anticipated service life. In the ACI 318 [32], maximum crack widths result from expected loads on the structure. Though this limitation of crack width has been proved adequate practically, its reasonableness has not been theoretically confirmed.

Based on aforementioned reasons above, the purpose of this study is to clarify the effect of various conditions including cover thickness, water-cement ratio, environment action and interfacial void on limitation of crack width. The validity of crack width limitation in various standards will be discussed based on analytical results.

2. Method of numerical simulation

2.1 Modeling of concrete cover with flexural crack

Numerical simulation of transport of water and chloride in concrete cover was carried out to investigate the influence of flexural crack on chloride ingress into RC members and its interaction between the interfacial void around reinforcement. The layout of a part of RC member with flexural crack and the interfacial void is shown in Fig. 1, in which $L$ is crack interval, $w$ is crack width, $v_w$ is width of interfacial void, $c$ is thickness of concrete cover. Variation of crack width along the crack depth, which may exist in actual flexural crack in reinforced concrete members, was not considered in the calculation in this study. In addition, though actual crack surface is rough and crack path is tortuous, crack is assumed straight in the calculation in this study. Water and chloride ions are assumed to penetrate into concrete from the exposed surface, the crack surface and the surface of the interfacial void around reinforcement.

![Fig. 1 - Layout of RC member with flexural crack and interfacial void](image1)

![Fig. 2 - Penetration of liquid water into crack and interfacial void around reinforcement](image2)
When concrete surface directly contacts with liquid water, liquid water penetrates into crack and interfacial void around reinforcement by capillary suction. Liquid water penetrates into concrete from three surfaces: the exposed surface, the crack surface and the surface of interfacial void around reinforcement. The amount of penetrated water into concrete from the crack surface and the surface of interfacial void around reinforcement does not exceed the amount of water in the crack and the void respectively. After the crack and the interfacial void have become empty, liquid water penetrates into concrete only from the exposed surface as shown in Fig. 2.

To solve governing equations numerically, a computer program developed by the authors was used [38]. Drying process was calculated by implicit FDM (Finite Difference Method). Wetting process was calculated using an integral solution of capillary suction model. Transport of chloride ions was calculated by explicit FDM.

2.2 Transport of water in concrete

Transport of water in nonsaturated concrete is calculated by the computational model based on pore size distribution function and microscopic thermodynamic behavior of water in pore structure [35].

Pore size distribution of concrete is modeled by following function:

\[ V(r) = V_0 \left[1 - \exp \left(-B r^C \right) \right] \quad (1) \]

where \( V(r) \) is accumulated pore volume whose radius is not greater than \( r \) in unit concrete volume (m\(^3\)/m\(^3\)), \( V_0 \) is the total volume of pores per unit concrete volume (m\(^3\)/m\(^3\)), \( r \) is pore radius (m), \( B \) is parameter for pore size distribution, \( C \) is parameter for pore size distribution.

The mass conservation equation of water in nonsaturated concrete is expressed as:

\[ \frac{\partial w}{\partial t} = -\text{div}(J_v + J_i) \quad (2) \]

where \( w \) is mass concentration of water per unit concrete volume (kg/m\(^3\)), \( t \) is time (s). The mass flux of vapour \( J_i \) and the mass flux of liquid water \( J_v \) in nonsaturated concrete are respectively calculated by the following equations.

\[ J_v = -K_v V_g D_{vo} \text{grad} \rho_v \quad (3) \]

\[ J_i = \int_0^r \frac{\rho_l}{\rho_f} \frac{dV(r)}{dr} \left(-K_i \frac{r^2}{8 \mu} \text{grad} \left( -\frac{2 \gamma}{r^3} \right) \right) dr \quad (4) \]

where \( K_v \) and \( K_i \) is non-dimensional material factor for transport of vapour and liquid water respectively, \( V_g \) is volume fraction of gas phase per unit concrete volume (m\(^3\)/m\(^3\)), \( D_{vo} \) is diffusivity of vapour in free space (m\(^2\)/s), \( \rho_v \) and \( \rho_l \) are density of vapour and liquid water respectively, \( r \) is pore radius where the liquid-gas interface is developed (m), \( \mu \) is viscosity of liquid water (Pa*s), \( \gamma \) is surface tension of liquid water (N/m).

2.3 Penetration of liquid water into concrete within crack and interfacial void

The mass flux of liquid water into concrete from the concrete surface \( (J_{lp}) \) by capillary suction is evaluated by following model [36]:

\[ J_{lp} = \int_{r_0}^r \frac{\rho_l}{\rho_f} \frac{dV(r)}{dr} \left( K_{lp} \frac{r}{8 \mu w} \right) dr \quad (5) \]

where \( K_{lp} \) is non-dimensional frictional coefficient depending on pore structure of concrete, \( r_0 \) is minimum radius of pore where capillary suction takes place (m), \( t_0 \) is time from the initiation of wetting process (s).

The mass flux of water in crack \( (J_{lw}^{cr}) \) and interfacial void \( (J_{lw}^{iv}) \) by capillary suction are evaluated respectively as:

\[ J_{lw}^{cr} = \rho_l \sqrt{\frac{w \gamma}{2 f w}} \quad (6) \]

\[ J_{lw}^{iv} = \rho_l \sqrt{\frac{v_w \gamma}{2 f w}} \quad (7) \]

where \( f \) is friction factor for transport of liquid water within crack and interfacial void (kg/m.s), \( w \) is crack width (m), \( v_w \) is width of interfacial void (m).

2.4 Drying from the crack surface and the surface of the interfacial void

The evaporation of water from concrete surface is evaluated by the following equation [35]:

\[ J_w = \frac{D(w_l)}{h} (w_l - w_{lw}) \quad (8) \]
where \( J_B \) is mass flux of water through the boundary surface (kg/m\(^2\)/s), \( w_i \) is water concentration of concrete at the surface (kg/m\(^3\)), \( w_{bg} \) is water concentration in equilibrium with atmosphere (kg/m\(^3\)), \( D(w) \) is equivalent moisture diffusivity at the boundary (m\(^2\)/s), \( h \) is thickness of boundary film representing the state of humidity distribution in the atmosphere near the surface, which is 0.00075 (m) at the ordinary exposed surface.

Since humidity in crack and the interfacial void around reinforcement in concrete is considered higher than the atmosphere, the evaporation of water from the crack surface and the surface of the interfacial void should be smaller than from the ordinary exposed surface. The evaporation of water from crack surface and surface of interfacial void are assumed respectively as:

\[
J_{Ber} = \beta_{cr} J_B
\]

\[
J_{Bv} = \beta_r J_B
\]

where \( J_{Ber} \) is mass flux of water through the boundary surface of crack (kg/m\(^2\)/s), \( J_{Bv} \) is mass flux of water through the boundary surface of interfacial void (kg/m\(^2\)/s), \( \beta_{cr} \) is non-dimensional factor which represents reduction ratio of evaporation from crack surface, \( \beta_r \) is non-dimensional factor which represents reduction ratio of evaporation from surface of interfacial void.

The value of \( \beta_{cr} \) and \( \beta_r \) will be determined in the next chapter of this paper based on comparison of numerical analysis and experiment.

### 2.5 Transport of chloride ions in concrete

Transport of chloride ions in concrete is calculated with considering molecular diffusion of free chloride ions within liquid water and mass flux of free chloride ions carried by liquid water:

\[
\frac{\partial C_{clh}}{\partial t} = -\text{div} \left( J_{Cldif} + C_{freeCl} \frac{J_l}{\rho_l} \right)
\]

where \( C_{clh} \) is total mass concentration of chloride per unit concrete volume, \( J_{Cldif} \) is mass flux of chloride by diffusion, \( J_l \) is mass flux of liquid water, \( C_{freeCl} \) is mass concentration of free chloride.

In this study, capillary suction from concrete surface is considered as transport mechanism of liquid water. The max flux of chloride ions by diffusion is calculated as:

\[
J_{Cldif} = -K_{Cl} D_{Cl} \nabla C_{freeCl}
\]

where \( K_{Cl} \) is non-dimensional material factor which presents the effect of narrowness and tortuosity of the pore structures of concrete, \( D_{Cl} \) is diffusivity of chloride ion in liquid water.

The transition between free and fixed chloride is calculated based on the equation proposed by Maruya et al [37]:

\[
C_{Clf} = \alpha C_{clh}
\]

where \( \alpha \) is fixing rate of chloride ions with cement hydrate in hardened concrete formulated as a function of \( C_{clh} \) and the type of cement.

### 3 Experimental investigation and its numerical simulation

#### 3.1 Specimen

Four reinforced concrete specimens, whose sizes are 100 mm x 200 mm x 900 mm, were prepared as shown in Fig. 3. Specimen names starting with H letter represent horizontal members and specimen names starting with V letter represent vertical member. One deformed steel bar of 13 mm in diameter was embedded in the longitudinal direction in the specimens, with 40-mm concrete cover from the top surface. Table 2 shows the mix proportion of concrete used. To emphasize the influence of bleeding, unit water of concrete was set as much as 185 kg/m\(^3\).

Specimens were cured being wrapped with wet mattress for 28 days in the laboratory. Thereafter, flexural cracks were induced by three-point-loading. Cracks are named C1, C2, C3, C4, and C5 as shown in Fig. 3. In specimen V1 and V2, two cracks were induced. However, since the two cracks had almost same widths with each other, only one crack in each specimen, which are named C4 and C5, were selected to measure chloride content. Crack widths measured by a crack scale are presented in Table 3. The measured crack widths are surface crack widths. In the analysis, these surface crack widths are considered as crack widths. In order to control the penetration of chloride ions into concrete, all surfaces except the exposed surface of the specimens were sealed with epoxy-type adhesive. Then, all specimens were placed in a chamber with controlled temperature and humidity and peri-
dical drying-wetting actions with mist of sodium chloride solution (5% NaCl). The length of one cycle was set one day consisting of 12 hours (1/2 day) mist containing 5% sodium chloride solution at 40°C temperature and 100% relative humidity and 12 hours (1/2 day) drying at 40°C temperature and 60% relative humidity. Though crack width may change actually by drying and wetting action in the experiment, crack width was not controlled during the exposure test.

After 65 days exposure test, specimens were taken out from the chamber and cut into 25 mm slices. Then, samples of concrete powder were taken from around the steel bar in the slices using an electric drill. Chloride concentration in concrete was measured with a chloride ion meter. The procedure of making concrete powder sample is shown in Fig. 4.

<table>
<thead>
<tr>
<th>Table 2 - Concrete mix proportion</th>
</tr>
</thead>
<tbody>
<tr>
<td>W/C (%)</td>
</tr>
<tr>
<td>---------</td>
</tr>
<tr>
<td>60</td>
</tr>
</tbody>
</table>
3.2 Test results

Figure 5 shows the experimental results of chloride concentration along reinforcing bar embedded in concrete after 65 days exposure test. It is regarded in all specimens that chloride concentration along reinforcing bar descends in accordance with the distance from crack. Chloride concentration around crack C1 whose width is 0.75 mm in specimen H1 is greater than around crack C2 whose width is 0.15 mm and around crack C3 whose width is 0.08 mm in specimen H2. This suggests that chloride ingress into concrete from crack surface is promoted by increasing of crack width. Chloride concentration around crack C1 is greater than in specimen H2. It is attributable to the existence of interfacial void around reinforcing bar due to bleeding, whose photographs are shown in Fig. 6. The influence of interfacial void on chloride ingress along reinforcing bar is greater in case of greater crack width.

Chloride concentration around crack C2 in specimen H2 in which concrete was cast in perpendicular direction to steel bar is almost same with chloride concentration around crack C4 in specimen V1 in which concrete was cast parallel to steel bar even though the crack width is same as 0.08 mm. The reason for this is that, although interfacial void due to bleeding occurred in specimen H2, liquid water did not reach reinforcing bar through crack because crack width was very small. Consequently, the influence of interfacial void is not so much when crack width is small.

Chloride concentration around crack C5 in specimen V2 is greater than around crack C4 in specimen V1 even though the crack width of both cracks are same as 0.08 mm. The reason of this is that, since concrete was cast parallel to steel bar in both specimens, concrete around crack C5 near the casting surface became more porous than around crack C4 far from casting surface due to bleeding effect.

3.3 Numerical simulation of experimental result

The values of \( \beta_{cr} \) and \( \beta_c \) are determined based on following parametric analysis, Figures 7 through 9 show distribution of calculated chloride concentration in concrete along the reinforcement near the crack, in which value of \( \beta_{cr} \) varied. Crack width in Figs. 7, 8, and 9 are 0.08 mm, 0.15 mm and 0.5 mm, respectively. No interfacial void is provided around reinforcement. Three values of \( \beta_{cr} \) shown in Figs. 7, 8, and 9 are examined for each crack width. It is clear that chloride concentration at the location of reinforcing bar increases with increasing of \( \beta_{cr} \). According to the results, analytical results agree well experimental results when the value of \( \beta_{cr} \) is 0.04 for \( w = 0.08 \) mm, 0.075 for \( w = 0.15 \) mm and 0.25 for \( w = 0.5 \) mm. The relationship between crack width and obtained \( \beta_{cr} \) is plotted in Fig. 10. From these results, \( \beta_{cr} \) is modeled as a function of crack width using following formulae:

\[
\begin{align*}
\beta_{cr} &= 0.5w \\
\beta_{cr} &= 1(w \geq 2\text{mm})
\end{align*}
\]

Figure 10 shows distribution of calculated chloride concentration in concrete along the reinforcement near the crack, in which value of \( \beta_c \) varied. Crack width is 0.75 mm and the width of interfacial void is assumed 0.02 mm. According to the results, analytical result agrees well experimental result when the value of \( \beta_c \) is 0.005. As there was only one experimental result with the interfacial void, the relationship between \( \beta_c \) and void width is assumed the same with the relationship between \( \beta_{cr} \) and crack width.

\[
\begin{align*}
\beta_v &= 0.5v_w \\
\beta_v &= 1(v_w \geq 2\text{mm})
\end{align*}
\]

where \( v_w \) is width of interfacial void (mm).

The material parameters for transport of moisture transport and chloride ions in concrete used in the analysis are listed in Table 4. These values are determined from mix proportion of concrete based on previous studies [35, 36, and 38]. Since the portion around crack C5 in Specimen V2 may be affected by bleeding, porosity \( V_o \) in this case is assumed greater than other cases. Conditions of specimens are presented in Table 5. The width of the void at steel-concrete interface in specimen H1 and H2 was assumed 0.02 mm according to the measurement results by microscope, while no interfacial void was provided in specimen V1, V2. Figure 12 shows the comparison of experimental chloride distribution and analytical ones along reinforcement in concrete. The tendency of chloride distribution around cracks: C1, C2, C3, C4, and C5 are all well simulated by the analysis. The un-smoothness of chloride distribution around C1 is attributable to the effect of penetration of chloride ions from the surface of the interfacial void into concrete. The analysis can express the influence of crack and interfacial void around reinforcement observed in the experiment by determining material parameters in the model adequately.
(a) C1 in Specimen H1 (w = 0.75 mm)

(b) C2 (w = 0.15 mm) and C3 (w = 0.08 mm) in Specimen H2

(c) C4 in Specimen V1 (w = 0.08 mm)

(d) C5 in Specimen V2 (w = 0.08 mm)

Fig. 5 - Experimental results of chloride concentration along reinforcing bar after 65 days of exposure
Fig. 6 - Interfacial void around reinforcement

(a) Specimen H1, H2
(b) Specimen V1, V2

Fig. 7 - Chloride concentration along of reinforcing bar in concrete with various value of $\beta_{cr}$ ($w = 0.08$ mm)

Fig. 8 - Chloride concentration along of reinforcing bar in concrete with various value of $\beta_{cr}$ ($w = 0.15$ mm)

Fig. 9 - Chloride concentration along of reinforcing bar in concrete with various value of $\beta_{cr}$ ($w = 0.5$ mm)

Fig. 10 - Chloride concentration along of reinforcing bar in concrete with various value of $\beta_{cr}$ ($w = 0.75$ mm, $v_w = 0.02$ mm)

Fig. 11 – Relationship between $\beta_{cr}$ and crack width
Table 4 - Material parameters

<table>
<thead>
<tr>
<th>No.</th>
<th>$V_0$ (m$^3$/m$^3$)</th>
<th>B</th>
<th>C</th>
<th>$K_v$</th>
<th>$K_l$</th>
<th>$K_{lp}$</th>
<th>$K_{cl}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>H1, H2, V1</td>
<td>0.15</td>
<td>14,000</td>
<td>0.5</td>
<td>0.05393</td>
<td>0.00108</td>
<td>0.03236</td>
<td>0.01079</td>
</tr>
<tr>
<td>V2</td>
<td>0.21</td>
<td>14,000</td>
<td>0.5</td>
<td>0.05393</td>
<td>0.00108</td>
<td>0.03236</td>
<td>0.01079</td>
</tr>
</tbody>
</table>

Table 5 - Conditions of specimens in numerical simulation

<table>
<thead>
<tr>
<th>No</th>
<th>c  (mm)</th>
<th>Crack width, w (mm)</th>
<th>Interfacial void, $v_w$ (mm)</th>
<th>Crack interval, L (mm)</th>
<th>Dry-wet (days)</th>
<th>Time (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>H1</td>
<td>40</td>
<td>0.75 (C1)</td>
<td></td>
<td>0.02</td>
<td>200</td>
<td>0.5-0.5</td>
</tr>
<tr>
<td>H2</td>
<td>0.15 (C2), 0.08 (C3)</td>
<td>0.02</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>V1</td>
<td>0.08 (C4)</td>
<td></td>
<td></td>
<td></td>
<td>200</td>
<td>0.5-0.5</td>
</tr>
<tr>
<td>V2</td>
<td>0.08 (C5)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>65</td>
</tr>
</tbody>
</table>

**Fig. 12 - Comparison of experimental and analytical chloride concentration along reinforcing bar after 65 days of exposure**
4. Evaluation of critical crack width based on numerical simulation

4.1 Method of evaluation

(1) Average chloride concentration at reinforcing bar

Chloride concentration along the reinforcement in concrete at each time $C(c, y)$ is obtained through numerical transport analysis. Average chloride content along reinforcement bar is considered as index of risk of reinforcement corrosion. Average chloride concentration at reinforcing steel bar $C_{av}$ is calculated by averaging $C(c, y)$ with respect to position $y$ for a crack interval:

$$C_{av} = \frac{1}{L} \int_{0}^{L} C(c, y)dy$$

(16)

Fig. 13 - Determination of $C_{av}$ by averaging method

(a) Calculated time-dependent chloride profile along the bar

(b) Calculated average chloride concentration at location of steel bar as a function of time

4.2 Critical chloride concentration for onset of corrosion

Corrosion of reinforcement embedded in concrete is promoted by chloride, supplement of water and supplemental oxygen. In case of structures under chloride prone conditions such as coastal area affected by airborne salt, splash zone, and tidal zone, onset of corrosion is mainly governed by chloride concentration at the location of reinforcing bar. In the standard specification for concrete structures by Japan Society of Civil Engineers (JSCE), critical chloride concentration for initiation of steel corrosion $C_{lim}$ ($kg/m^3$) is formulated as a function of type of cement and water-cement ratio of concrete. For ordinary Portland cement, it is expressed as:

$$C_{lim} = -3.0(W/C) + 3.4$$

(17)

where $W/C$ is water-cement ratio of concrete.

In this study, criteria of onset of corrosion is defined as

$$\frac{C_{av}}{C_{lim}} = 1$$

(18)

where $C_{av}$ is average chloride content at location of reinforcing bar.

4.3 Critical crack width to prevent corrosion during service life

Based on numerical analysis, $C_{av}/C_{lim}$ is evaluated as a function of time. It increases with increasing of time. Critical crack width is defined when $C_{av}/C_{lim}$ becomes one at the end of service life of structure. If crack width is greater than the critical width, reinforcement corrosion will start within service life. Therefore, critical crack width depends on the expected service life of the structure. The smaller crack width should be restricted to, the longer service life is expected. In this study, 50 years of service life is assumed.

4.2 Cases of parametric sensitivity analysis of influencing factors on critical crack width

Table 6 shows influencing factors and their variation examined in the conducted sensitivity analysis. Environmental condition represented by drying-wetting cycle is examined because limitation of crack width in the JSCE standard specification (2002) is regulated depending on severeness of environmental condition. The influence of water-
cement ratio of concrete is considered in terms of pore size distribution, material parameters for transport of water and chloride ions in Chapter 2 and critical chloride concentration for initiation of steel corrosion. 43 cases shown in Table 7 are calculated in total.

Table 7 - Calculated cases in parametric sensitivity analysis

<table>
<thead>
<tr>
<th>Case</th>
<th>W/C (%)</th>
<th>c (mm)</th>
<th>w (mm)</th>
<th>vw (mm)</th>
<th>L (mm)</th>
<th>L. W-Dry-Wet (day)</th>
<th>Cav/Clm at 50 years</th>
<th>W-Dry-Wet (day)</th>
<th>Cav/Clm at 50 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>45</td>
<td>80</td>
<td>0</td>
<td>0</td>
<td>400</td>
<td>0.15</td>
<td>0.48</td>
<td>2.85</td>
<td>0.7</td>
</tr>
<tr>
<td>2</td>
<td>45</td>
<td>80</td>
<td>0.05</td>
<td>0</td>
<td>400</td>
<td>0.15</td>
<td>0.57</td>
<td>2.85</td>
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<td>0.15</td>
<td>0.48</td>
<td>2.85</td>
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</tr>
</tbody>
</table>

Table 6 - Influencing factors examined and their variation

<table>
<thead>
<tr>
<th>Influencing factors</th>
<th>Variations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete cover, c</td>
<td>30, 50, 80</td>
</tr>
<tr>
<td>Water-cement ratio, W/C (%)</td>
<td>30, 45</td>
</tr>
<tr>
<td>Crack width, w (mm)</td>
<td>0, 0.05, 0.1, 0.2, 0.5, 1.0</td>
</tr>
<tr>
<td>Void width, vw (mm)</td>
<td>0, 0.15</td>
</tr>
<tr>
<td>Environment condition (Drying–wetting cycle)</td>
<td>Tidal zone (2.85d D-0.15d W), splash zone (1.5d D-1.5d W)</td>
</tr>
</tbody>
</table>

Fig. 15 – Chloride concentration at the location of reinforcing bar at 50 years (without interfacial void due to bleeding)

Fig. 16 - Chloride concentration at the location of reinforcing bar at 50 years (with interfacial void due to bleeding)
4.3 Analytical results of critical crack width

(1) Influence of cover thickness and interfacial void on critical crack width

Figures 15 and 16 show the relationship between crack width and calculated average chloride concentration at the location of reinforcing bar at 50 years with and without interfacial void around reinforcement due to bleeding respectively. Arrow indicates the point where average chloride concentration at the location of reinforcing bar reaches critical chloride concentration at 50 years. Crack width at this point is regarded as critical crack width to prevent corrosion during service life in terms of chloride ingress. In the series in Fig. 15, no interfacial void is provided around reinforcement. In series in Fig. 16, 0.15mm interfacial void due to bleeding is assumed. It is found in Fig. 15 that critical crack width is 0.05mm, 0.18mm, and 0.6mm in cases that cover thickness is 30mm, 50mm, and 80mm respectively. In Figure 16, critical crack width is 0.02mm, 0.05mm, and 0.13mm in cases that cover thickness is 30mm, 50mm, and 80mm respectively. The relationship between provided cover thickness and evaluated critical crack widths are plotted in Fig. 17. It can be seen in Fig. 17 that critical crack width increases with increasing of cover thickness in the series both with and without interfacial void. Critical crack width is decreased by the existence of interfacial void. This suggests that defect around steel bar due to bleeding should be avoided to make crack width control in RC member effective.

(2) Influence of environmental condition on critical crack width

Figure 18 shows the relationship between crack width and calculated average chloride concentration at the location of reinforcing bar at 50 years in tidal zone and splash zone. The tidal zones consist of 1.5 days of wetting and 1.5 days of drying, while the splash zones consist of 2.85 days of wetting and 0.15 day of drying. It is found that critical crack width is 0.05mm in tidal zone and 0.18mm in splash zone respectively. It means that tidal zone is severer than splash zone for corrosion of reinforcement in concrete. This is because water and chloride penetration into concrete through crack increases with increasing of wetting period.

(3) Influence of water-cement ratio on critical crack width

Figure 19 shows the relationship between crack width and calculated average chloride concentration at the location of reinforcing bar at 50 years for concrete of W/C = 0.3 and 0.45. It is found that critical crack width is 0.34mm and 0.18mm in cases that water-cement ratio is 30% and 45% respectively. It is suggested that, when concrete with higher water to cement ratio is used, crack width should be restricted more strictly to attain same level of durability.
5. Comparison of analytical critical crack width with those in design codes

Critical crack width or allowable crack width in several design codes is presented in Table 8.

Table 8 - Allowable crack width in design codes

<table>
<thead>
<tr>
<th>Name of Standard</th>
<th>Crack width limitation (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>JSCE (2012)</td>
<td>0.005c (&lt; 0.5 mm)</td>
</tr>
<tr>
<td>fib Model Code (2010)</td>
<td>0.3 (c/40)</td>
</tr>
<tr>
<td>BSI</td>
<td>0.3</td>
</tr>
</tbody>
</table>

In JSCE standard and fib Model Code, critical crack width is regulated as a function of cover thickness. Figure 20 shows the relationship between cover thickness and critical crack width by JSCE standard specification, fib Mode Code, British Standards Institution and obtained by numerical simulation in this study. Compared with fib Mode Code, analytically obtained critical crack width is close when cover thickness is 80mm, while the difference becomes greater when cover thickness is thinner. One of the reasons of this is that numerical simulation is carried out under drying-wetting condition, which is regarded severe condition for corrosion. Compared with JSCE standard, analytically obtained critical crack width is almost same when cover thickness is smaller than 60mm, the difference become greater when cover thickness is 80mm. In consequence, analytical critical crack width shows the tendency between JSCE standard and fib Model Code in both their value and dependency on cover thickness. Though critical crack width by BSI is constant value, it is almost same with average value of critical crack width by JSCE with respect to cover thickness. It was confirmed that the critical crack width that has been used in practical design is reasonable from the viewpoint of protection of reinforcement from ingress of aggressive agent.

6. Conclusions

Followings conclusions can be drawn from the conducted experiment and analysis:

(1) It was experimentally confirmed that chloride ingress along reinforcement is accelerated by the existence of the interfacial void around reinforcement. The influence of interfacial void on chloride ingress along reinforcing bar became greater when crack width was great.

(2) When concrete was cast parallel to steel bar, concrete near the casting surface became more porous than one far from casting surface due to bleeding effect.

(3) Analytical method for chloride ingress into RC member, in which transport of water and chloride through flexural crack and the interfacial void around reinforcing bar are considered, was developed and verified by laboratory test. Chloride concentration at the location of steel bar increases with increasing of either crack width or width of interfacial void.

(4) The results of the conducted parametric sensitivity analysis suggested that defect around reinforcement due to bleeding should be avoided in order to make the crack width control in RC member effective and crack width should be restricted more strictly to attain a same level of durability when concrete with higher water to cement ratio is used.

(5) The critical crack width for corrosion of reinforcement in concrete evaluated by the numerical simulation showed similar tendency with critical crack widths in regulated in the design codes including JSCE standard specification. It was confirmed that the critical crack width that has been used in practical design is reasonable from the viewpoint of protection of reinforcement from ingress of aggressive agent.

Acknowledgement

The authors wish to express their gratitude to members of Concrete Laboratory in Nagaoka University of Technology. Especially thanks to Mr. Yamaguchi, Mr. Ohara, Mr. Ino and Mr. Minowa for kind helping and good working in experimental work.

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Research on flexural behavior of the externally prestressed UHPC box girder

Jiazhan SU*; Renyuan DU; Baochun CHEN; and Qingwei HUANG

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Abstract: Experiment on an externally prestressed ultra high performance concrete (UHPC) box girder subjected to symmetrical concentric loads is carried out. The behavior of the test girder is investigated, including the load-deflection curve at mid-span, cracking pattern, strain distribution, and failure mode. The mid-span nominal moment capacity of the girder is analyzed. Test results show that the anti-cracking capacity of UHPC box girder is better than that of a normal prestressed concrete (PC) girder. The tensile property of UHPC should be taken into account in the process of calculating mid-span nominal moment capacity of the externally prestressed UHPC box girder. Finite element (FE) model analysis results agree well with the test results. The influencing parameter, concrete grades, was studied numerically to compare the flexural behavior of UHPC girder with PC girder. The cracking moment and ultimate load calculation method is proposed, which can meet precision for engineering practice and can be a reference method for design calculation of a prestressed UHPC box girder.

Keywords: flexural behaviour, UHPC, external prestressing, box girder, test, calculation method.

1. Introduction

Ultra High Performance Concrete (UHPC) is a relatively new type of concrete which has higher strength, greater stiffness, and better durability than normal concrete and high-performance concrete. It is mainly composed of cement, silica fume, sand, super-plasticizer, and steel fiber [1]. It has a broad application prospects in civil engineering, including bridges, buildings, nuclear engineering, municipal structures, ocean engineering, due to its different roles in reducing the structural self-weight, enhancing the bearing capacity, and improving the ductility [2,3]. At present, the UHPC box girder had been used for bridges in Japan [4], Austria [5,6], and Malaysia [7,8]. Compared to I-shaped or T-shaped girders, the box girders offer better resistance to torsion, which is particularly beneficial if the bridge deck is curved in plan. Additionally, larger girders can be constructed, because the presence of two webs allows wider and hence stronger flanges to be used. However, experimental investigation has focused on I-shaped and T-shaped girders [9,10], few researches have to do with prestressed UHPC box girders, and few test results are available on the UHPC girders. Therefore, the mechanical behavior of large-scale UHPC box girder is required by engineering and it is necessary to carry out an investigation.

2. Experimental programme

Taking a trial design UHPC foot-bridge as the prototype [11], the UHPC box girder model was a quarter of size of the original one. The total length of the test girder was 12.4 m (see Fig. 1). The test girder was simply supported at both ends with a span of 12.0 m. The girder was made up of five precast segments connected by wet joints cast-in-situ (200-mm-thick). Six shear keys were set on two webs at joint section of segment for improving the shear capacity at the connection of segments. The profile and reinforcement of a segment is shown in Fig. 2. The height of the segment is 400 mm. The width of the top and bottom flanges is 1,250 mm and 600 mm, respectively. The top and bottom flanges and two webs have the same thickness of 50 mm. A total of 15 and 7 steel bars with 8-mm diameter were used at the top flange and the bottom flange, respectively, while ten 6-mm diameter steel bars were used in the webs. Two 15.2-mm diameter

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Qingwei HUANG is an associate professor of College of Civil Engineering, Fuzhou University, China.
steel strands were set as external tendons and 200-mm width deviators were set for external tendons.

The mix proportion of UHPC used for the girder was as follows: 842.4 kg/m$^3$ ordinary Portland cement, 985.6 kg/m$^3$ sand taken from Minjiang River with particle size less than 0.63 mm, 252.7 kg/m$^3$ silica fume in which the content of SiO$_2$ was more than 90%, and 156 kg/m$^3$ micro steel fibers with length of 13 mm and diameter of 0.22 mm and tensile strength of 2,850 MPa. In order to match the material properties of the precast UHPC and the cast-in-situ UHPC, the steel fiber volume content of wet joint UHPC was 3%, higher than that of the precast concrete (2%). The water-to-binder (the cementitious materials include cement and silica fume) ratio was 0.18 and 21.1 kg/m$^3$ poly carboxylic high-performance super-plasticizer was used to improve the workability of the UHPC mixture. The test girder was cured in natural environment for 2 days, and then in steam environment (180ºC) for 8 hours, and then left in natural environment for 14 days. Wet joints were cured in 100ºC steam chamber for 3 days in-place.

The mechanical properties of the UHPC are given in Table 1, where $f_{cc}$ is the compressive strength measured on 150 mm $\times$ 150 mm $\times$ 150 mm cubes, $f_{ck}$ is the prism (150 mm $\times$ 150 mm $\times$ 300 mm) compressive strength, $f_t$ is the flexural tensile strength, $E_c$ is the Young modulus. The mechanical properties of reinforcing steel bars are listed in Table 2.

The loads were located in third span points symmetrically as shown in Fig. 1. The central region between the two loads was subjected to constant bending moment with zero shear. Strain gauges were positioned at $L/2$, $L/4$, $3L/4$, two loading points, and bearing sections to measure the strains of reinforcing steel and UHPC. Seven linear variable displacement transducers (LVDTs) were set at the same positions to measure the displacement of the girder. The test photograph is shown in Fig. 3.

![Fig. 1 – Test setup for the model (unit: mm)](image-url)

Table 1 – Material properties of UHPC

<table>
<thead>
<tr>
<th>Item</th>
<th>$f_{cc}$ (MPa)</th>
<th>$f_{ck}$ (MPa)</th>
<th>$f_t$ (MPa)</th>
<th>$E_c$ (MPa)</th>
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<tr>
<td>Precast segments</td>
<td>160.0</td>
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<td>8.01</td>
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<tr>
<td>Wet joints</td>
<td>150.4</td>
<td>140.4</td>
<td>9.18</td>
<td>4.05$\times$10$^4$</td>
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Table 2 – Material properties of reinforcing steel

<table>
<thead>
<tr>
<th>Item</th>
<th>Yield strength (MPa)</th>
<th>Ultimate tensile strength (MPa)</th>
<th>Modulus of elasticity (MPa)</th>
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<tr>
<td>Normal steel bars</td>
<td>359</td>
<td>460</td>
<td>$1.97 \times 10^3$</td>
</tr>
<tr>
<td>Steel strands</td>
<td>1,603</td>
<td>1,886</td>
<td>$1.94 \times 10^5$</td>
</tr>
</tbody>
</table>
3. FE model

For further studying of the flexural behaviour of UHPC girder, the finite element program, ANSYS, was used for 3-D analyses of UHPC girder. The objectives of the analysis were to explain the discrepancies of experimental results, and to enhance the understanding of the accuracy of the theoretical assumptions for the problems. Throughout the studies, the concrete was modeled using solid element, Solid65. The 8-node solid element with 24 DOF is capable of modeling cracking and crushing of concrete. Link element (Link8) was used for the tendons. Figure 4 shows the FE mesh of a girder. The mesh consisted of 8,452 elements and 11,620 nodes. The shear transfer coefficient for an open crack and a closed crack of Solid65 was 0.5 and 1.0, respectively. The boundary condition of the FE model was set as close as possible to the boundary condition of the test. The loads were applied to the top loading blocks as concentrated loads at the test load points. To model the boundary conditions associated with the bottom surface, the transversal displacement DOF associated with the nodes at the supports were restrained.

The compressive constitutive relationship proposed by Du (2014) [12] was adopted for concrete in the FE analyses, as shown in Eq. (1). The tensile constitutive relationship proposed by Du (2014) [12] was adopted, as shown in Eq. (2).
The structural behaviour of the girder can be divided into three phases: 1) the elastic phase, 2) the crack developing phase, and 3) the reinforcing steel yielding phase. The horizontal displacement of the girder was approximately symmetric.

4. Test results and FE analysis

4.1 Deflection curves

The maximum longitudinal displacement of the girder was only 6.85 mm at the sliding bearings in test. It was indicated that the requirement as a simply-supported girder is satisfied. Figure 5 shows the load-versus-vertical-displacement curves of the model obtained from test and FE model. It can be seen that the structural behaviour of the girder can be divided into elastic phase, crack developing phase, and reinforcing steel yielding phase.

4.2 Cracks

The first crack appeared at mid-span while the load was increased to 55 kN. More cracks were observed with the increasing load. These cracks got wider and deeper and propagated towards lateral sides with further loading. The biggest crack width was about 0.6 mm, located at the bottom at the mid-span. The crack zone was about 5,750 mm long when the load was reached to 100 kN, as shown in Fig. 7(a). The crack distribution in the pure bending zone was uniform, with an average spacing of 95 mm. The cracks in the shear-bending zone were distributed in the range of about 900 mm from the load point. It is indicated that the anti-crack capaci-
The tensile strength of UHPC should be taken into consideration in the process of calculating the cracking moment. The crack distribution of the FE model is within 6,447-mm length (see Fig. 7(b)), which is slightly larger than the test result. Therefore, the proposed numerical analysis approach can be employed to analyse well the mechanical behaviour of UHPC girders.

4.3 Strain distribution

The strain distribution of the test girder at mid-span is presented in Fig. 8(a). It can be seen that the average strain distribution of the girder agrees with plane section assumption. Figure 8(b) shows that the strains on concrete across the top flange are almost same: the difference between the maximum and the minimum is only 29 με. It shows that the strain distribution is relatively uniform on the flange and the shear lag effect is small in this kind of girder.

4.4 Failure mode

In the process of the experiment, there was no partial cracking and damage in wet joints. It is demonstrated that the construction method of the girder is feasible. The maximum compressive strain of UHPC is 744 με in the test girder, smaller than the ultimate strain 3,373 με. The external pre-stressed tendons have not reached its ultimate strength when the girder damaged. The test girder demonstrated a ductile failure. Therefore, it is reasonable that the cross-section of test UHPC girder is under-reinforced as the normal PC beam for avoiding the brittle failure or even explosive destruction of UHPC.

4.5 Parametric studies

It has been shown in Section 4.1 that the FE can be used to model the UHPC girders with excellent accuracy. In order to compare the flexural behavior of UHPC girder with PC girder, the influencing parameter, i.e. concrete grade of C40, C60, and C80, with compressive strength of 40 MPa, 60 MPa, and 80 MPa, respectively, were studied numerically. Other geometric factors were kept the same as those in the test girder. The compressive and tensile stress-strain curves were modified for concrete grade of C40, C60, and C80, as shown in Eqs. (5) and (6), respectively.
Failure load (kN)
Cracking load (kN)
Table 3

34.3%, and 35.1% higher than those of PC girders
and ultimate load of UHPC girder are 34.1%,
which is similar to that of test UHPC girder. How-
ever, the cracking load, reinforcement yield load,
and ultimate load of UHPC girder are 34.1%,
34.3%, and 35.1% higher than those of PC girders
(see Table 3). It is indicated that the compressive
strength has less effect on the cracking, yield,
and failure loads, while tensile strength of UHPC has
better effect on those loads. Therefore, the tensile
property of UHPC should be taken into account in
the process of calculating mid-span nominal capaci-
ties of the externally prestressed UHPC box girder.

The distribution and simplified assumption of
stress and strain of normal section when the girder
began to crack is shown in Fig. 10. It assumes the
stress of concrete in both of the compressive and
tensile zones is equivalent to a triangle. Equations
(7) and (8) can be obtained according to the equilib-
rium condition. Equations (7) and (8) can be obtained according to the equilib-
rum condition. Equations
\[ \sigma = \begin{cases} \sigma_0 \left( \frac{\varepsilon - \varepsilon_0}{\varepsilon_y - \varepsilon_0} \right)^2 & 0 \leq \varepsilon \leq \varepsilon_y \\ \sigma_0 \left( \frac{\varepsilon - \varepsilon_0}{\varepsilon_y - \varepsilon_0} \right)^2 & \varepsilon_y < \varepsilon < \varepsilon_{cu} \end{cases} \]

Fig. 9 – Load-deflection relationship of different girders at mid-span

Table 3 – Principal loads of different girders

<table>
<thead>
<tr>
<th>Concrete grade</th>
<th>PC girder</th>
<th>Test UHPC girder</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>C40</td>
<td>C60</td>
</tr>
<tr>
<td>Cracking load (kN)</td>
<td>40</td>
<td>41</td>
</tr>
<tr>
<td>Yield load (kN)</td>
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<td>67</td>
</tr>
<tr>
<td>Failure load (kN)</td>
<td>73</td>
<td>74</td>
</tr>
</tbody>
</table>

5. Simplified calculation method of UHPC girder

5.1 Cracking moments

The distribution and simplified assumption of
stress and strain of normal section when the girder
began to crack is shown in Fig. 10. It assumes the
stress of concrete in both of the compressive and
tensile zones is equivalent to a triangle. Equations
(7) and (8) can be obtained according to the equilib-
rum condition. Equations
\[ \sigma = \begin{cases} \sigma_0 \left( \frac{\varepsilon - \varepsilon_0}{\varepsilon_y - \varepsilon_0} \right)^2 & 0 \leq \varepsilon \leq \varepsilon_y \\ \sigma_0 \left( \frac{\varepsilon - \varepsilon_0}{\varepsilon_y - \varepsilon_0} \right)^2 & \varepsilon_y < \varepsilon < \varepsilon_{cu} \end{cases} \]

\[ \sigma = \begin{cases} \frac{\sigma}{\varepsilon} \varepsilon & 0 \leq \varepsilon \leq \varepsilon_t \\ \frac{\sigma_{tu} - \sigma}{\varepsilon_{tu}} \varepsilon - \varepsilon_t & \varepsilon_t < \varepsilon \leq \varepsilon_{tu} \end{cases} \]
in the compressive zone as a triangle, while the stress in the tensile zone is equivalent to a rectangle. It is taken as the ultimate tensile strength \( f_t \) multiplied by a reduction factor \( \beta \). Equations (9) and (10) can be obtained according to the equilibrium condition. Table 5 lists the mid-span nominal bending capacity under different reduction factor. It can be found that the value of \( M_{u2}/M_{u1} \) is 0.96 when \( \beta = 0.8 \). Therefore, the simplified method coincided with test ultimate load better and safer, which was suggested for further use in practice.

\[
\int_{A_c} \sigma_{c} dA = \int_{A} f_y A_y + f_y A_y + \sigma_{pu} A_p
\]

\[
M_u = \int_{A_c} f_y A_y dA + f_y A_y + \sigma_{pu} A_p y_p
\]

in which, \( \sigma_{pu} \) is ultimate tensile strength of prestressed tendons.

### Table 4 – Comparison between calculation and test results

<table>
<thead>
<tr>
<th>Item</th>
<th>Calculation ( M_{cr} ) (kN.m)</th>
<th>Test results ( M'_{cr} ) (kN.m)</th>
<th>( M_{cr}/M'_{cr} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test UHPC girder</td>
<td>125</td>
<td>130</td>
<td>0.96</td>
</tr>
<tr>
<td>UHPC girder (T600S) in Ref. [13]</td>
<td>212</td>
<td>224</td>
<td>0.95</td>
</tr>
<tr>
<td>UHPC girder (T1300S) in Ref. [13]</td>
<td>923</td>
<td>1011</td>
<td>0.91</td>
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</table>

### Table 5 – Mid-span nominal bending capacity under different \( \beta \)

<table>
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<tr>
<th>Reduction factor ( \beta )</th>
<th>0.7</th>
<th>0.8</th>
<th>0.9</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test result ( M_{u1} ) (kN.m)</td>
<td>220</td>
<td>220</td>
<td>220</td>
<td>220</td>
</tr>
<tr>
<td>Calculation ( M_{u2} ) (kN.m)</td>
<td>202</td>
<td>211</td>
<td>221</td>
<td>230</td>
</tr>
<tr>
<td>( M_{u2}/M_{u1} )</td>
<td>0.92</td>
<td>0.96</td>
<td>1.00</td>
<td>1.05</td>
</tr>
</tbody>
</table>

![Fig. 10 – Schematic diagram of cracking moments of UHPC box girder](image)

![Fig. 11 – Simplified model at ultimate](image)

### 6. Conclusions

In the process of the experiment, there is not partial cracking and damage in wet joints, which demonstrated that the construction method of the girder is feasible. The girder presented ductile failure like the normal under-reinforced beam. The tensile strength of UHPC should be taken into account
in the process of calculating cracking moment and ultimate loads. Same as PC girder, the plane-section assumption is satisfied and can be used for sectional analysis in the test girder. The shear lag effect is small at the elastic stage in the girder, for it has just a little difference among the strains on the top flange at the mid-span. FE model analysis results agree well with the test results. The simplified cracking moment and ultimate load calculation method is proposed for the prestressed UHPC girder. It is demonstrated that the method can meet precision for practice engineering and can be a reference method for design calculation of a prestressed UHPC box girder.

Acknowledgment
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References
Technical Paper

Sandwich panels of ultra-high performance concrete composite with expanded polystyrene

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Abstract: The performance of sandwich panels is upgraded by increasing thermal resistance using UHPC (ultra-high performance concrete) with EPS (expanded polystyrene) beads composite. The core of the sandwich panels is made of lightweight UHPC composite with EPS and the face sheet by thin UHPC plates. The core provides the thermal resistance and the outer face sheets provide the flexural strength of the sandwich panels. Fresh UHPC is prepared to mix with EPS beads to produce new composite material with improved thermal resistance. The weak bond at interfaces between UHPC and EPS beads can be improved by pre-wetting the beads for one day. The ratio of UHPC to EPS beads should be proportioned to balance between strength and thermal resistance. Various thermal and mechanical properties of UHPC composite core material and the flexural strength of sandwich panels for architectural components are investigated in this paper. The performance of sandwich panels of UHPC composite core with UHPC face sheets shows one of potential applications of UHPC.

Keywords: sandwich panel; lightweight aggregate concrete; ultra-high performance concrete (UHPC); expanded polystyrene.

1. Introduction

The higher-rise and larger and longer spanning structures are being constructed rapidly in various ways with more diversity of buildings and civil engineering structures. Better structural and durability performance of construction materials with higher strength, lower density, higher energy efficiency and others is required. Especially, the demand for lightweight concrete in many applications of modern construction is increasing. Owing to the advantage of lower density and load-bearing elements of smaller cross sections, a corresponding reduction in the size and a significant reduction in the self-weight have a positive impact on the economics of construction projects. Lightweight concrete can be applied in a variety of ways. One of them is the application as a core material of the composite sandwich structure. The composite sandwich panels have been widely used for weight-sensitive structures that require high flexural strength for several decades.

The composite sandwich panels have emulated a typical structure comprising a relatively thin, stiff, and strong face sheet with a relatively thicker and lighter core. Sandwich structures can be combined in a variety of face sheets and core materials to create an optimal design. The main advantages of composite sandwich panels are high strength and stiffness, lightness, high insulation, and the possibility of creating versatile functions.

The main purpose of this paper is to investigate the thermal and mechanical properties of ultra-high-performance concrete composite sandwich panels by combining various core materials and face sheets. The possible panel configurations of sandwich panels were selected and ultra-high-performance concrete with expanded polystyrene composite (UHPEPC) was used as the core material. In addition to UHPEPC, the mechanical properties of the sandwich panels were also investigated. The actual panel behavior was observed by bending load tests on seven types of composite sandwich panels.

2. Background

The material for sandwich panel selection is based on its mechanical properties, low cost, low density, resistance to fluctuations in temperature,
moistures and chemicals, and good formability. Expanded polystyrene is used because its strength properties are well matched to the needs of particular structures and a wide range of concrete densities and strength can be achieved by incorporating the EPS beads in the concrete or mortar at different volume ratio [1]. Furthermore, EPS possesses low moisture absorption characteristics. It should be noted that the moisture absorption rates decrease as density increases, but not significantly. It has quite uniform and reliable density of 32 kg/m³.

The core usually is the weakest portion of sandwich panels and therefore in many ways control the capacity and lifetime of the whole composite sandwich structure. Earlier researchers reported that EPS beads have extremely low density and are hydrophobic. It can result in a poor mix distribution and segregation, necessitating admixtures or treatment on EPS beads’ surface. In that context, bonding additives such as water-emulsified epoxies and aqueous dispersions of polyvinyl propionate were added [2] or chemically treated EPS beads which are capable of preventing the segregation in the concrete mixture were used [3].

Previous research reported that the compressive strength of EPS concrete increases with a reduction in EPS bead size for the same concrete density [4, 5]. This scaling phenomenon was first observed by Parant and Le Roy based on an experimental study aiming to formulate and optimize an EPS concrete with a density ranging from 600 to 1,400 kg/m³ and having structural strength more than 20 MPa [6].

Sandwich panels, comprising of a core covered by face sheets, are frequently used as an alternative of solid plates because of their high bending stiffness-to-weight ratio. The high bending stiffness is the result of the distance between the face sheets, which carry the load, and the light weight is owing to the light weight of the core [7]. The separation of the face sheets by the core increases the moment of inertia of the panel with little increase in weight, producing an efficient structure for resisting bending and buckling loads. The face sheet materials are typically aluminum or fiber-reinforced composites such as glass fiber reinforced polymer (GFRP); the cores are rigid polyurethane, expanded polystyrene (EPS) or paper-resin honeycombs, or balsa wood, aluminum [8]. Despite their very competitive costs, the structural capacity of these conventional sandwich panels is hardly compatible with their use for floors, walls in buildings or bridge decks. The main weaknesses of these panels originate from the low stiffness and strength of the core, and the top face sheet vulnerability to delamination and buckling, due to the local incongruity stiffness and the absence of reinforcements connecting the core and the face sheets [9].

The contribution of core material that has high strength and shear stiffness is significant. It should be used to determine the overall behavior of the composite sandwich beams. Correia et al. [9] fulfilled the experimental investigations that included material characterization and flexural tests on composite sandwich panels. The panels are constituted by a rigid plastic polyurethane (PU) foam and polypropylene (PP) honeycomb – combined with glass fiber reinforced polymer (GFRP) face sheets. Characteristics of the core material – a PU rigid foam and PP honeycomb core were compared. The panels made of PP honeycomb core were stiffer than those made of PU foam core, fundamentally due to the higher shear modulus of the PP honeycomb core. The panels collapsed attributable to core shear failure.

Considering their possible structural use in real applications, the structural capacity of panels should be studied with experiments. Manalo et al. [10] studied the flexural behavior and failure mechanisms of composite sandwich beams in flatwise and edgewise positions. In the flatwise position, the composite sandwich beams failed with sudden brittle failure under flexural loading. In the edgewise position, the introduction of fiber composite face sheets increased the ultimate strength of the composite sandwich beams. When tensile cracks occurred in the core, the non-horizontal face sheets prohibited it from widening and prevented the sudden failure of the beam.

Typical concrete composite sandwich panels comprise of concrete and insulation. Various types of composite sandwich panels have been developed to increase the thermal efficiency. These panels have been applied to various building structures, such as residential and office buildings, cold storages, and industrial buildings. They have been more commonly used for the exterior wall, but they have also been used for the interior wall. There are various insulation materials, including fiberglass, mineral wool, and polystyrene. The extruded polystyrene (XPS) and expanded polystyrene (EPS) are most commonly used for the insulation due to high thermal performance and workability. Their construction cost is lower than that of other materials when the same thermal performance is secured.

3. Mechanical properties of UHPEPC

To facilitate the evaluation of the varying thermal and mechanical characteristics per the quantity of EPS lightweight aggregate, the method
of volumetric substitution for UHPC was investigated in this paper. The basic approach of material design is to replace the UHPC contained in the unit volume with EPS. As the volume of EPS beads increases, the UHPC of the same volume decreases, the strength decreases, and the lightness and heat insulation characteristics are improved.

### 3.1 Preparation of materials

The mixing proportion of UHPC is presented in Table 1. The specimens were cast and wet cured for 24 hours. After demolding, they were steam cured for 48 hours. Type I Portland cement meeting the requirements of ASTM C150, and silica fume made in Norway were used in this research. A commercial silica powder with particle-size distribution of 45–800 µm was used as aggregate. This silica powder contained 97% of SiO$_2$ and the hardness and density were 7 and 2.65 g/cm$^3$, respectively. The silica powder filler which was of medium size between cement and silica fume and improves the compressive strength of concrete. It also activates hydration reaction by supplying additional SiO$_2$ component. Super plasticizer which has 1.01 g/cm$^3$ density and the steel fiber with 0.2 mm of diameter and 13 mm of length were used as shown in Table 1.

Expanded polystyrene (EPS) beads were utilized as artificial lightweight aggregates for decreasing the weight and producing different grades of EPS concrete. The size of 85% of EPS particles was about 3.5 mm and their true density was evaluated to be 50.58 kg/m$^3$.

The strength of high performance expanded polystyrene concrete was varied by changing the steel fiber addition rate from 0% to 2% by volume to improve the flexural strength. Curing temperatures were set to 20, 60, and 90 °C, respectively, to investigate the effects of different curing temperatures on high performance expanded polystyrene concrete. The total 17 high performance expanded polystyrene concrete specimens were tested under compression and, in addition, flexural strength was also examined for 7 specimens among them. Table 2 presents the parameter of test specimens. The mixtures include substituting 0%, 30%, 40%, 50%, 55%, 60%, and 70% of aggregate volume by EPS beads as partial replacement of UHPC. The mixing was done in a specific sequence. EPS beads were prepared initially and mixing with UHPC was continued until a uniform and well flowing mixture was obtained. To prevent segregation of fresh UHPEPC, EPS beads were soaked in a super plasticizer for one day before mixing with the other material. The weak bond at interfaces between UHPC and EPS beads was improved by pre-wetting the beads.

Cubes (50 x 50 x 50 mm) were used to measure the compressive strengths at 7 and 28 days. Beam specimens of 160 x 40 x 40 mm size were used to conduct flexural strength test. To evaluate modulus of elasticity and Poisson’s ratio, cylindrical concrete specimens with 100-mm diameter and 200-mm height were used. The replacement ratio of UHPC with EPS beads was 30%, 40%, 50%, 60%, and 70% by volume as shown in Fig. 1.

### Table 1 – Mixing proportion of UHPC

<table>
<thead>
<tr>
<th>Materials</th>
<th>Cement</th>
<th>Silica Fume</th>
<th>Sand</th>
<th>Filler</th>
<th>Super plasticizer</th>
<th>Water</th>
<th>Steel fiber</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wt. % of cement</td>
<td>1.0</td>
<td>0.25</td>
<td>1.1</td>
<td>0.35</td>
<td>0.025 ~ 0.04</td>
<td>0.185 ~ 0.225</td>
<td>2.0 (vol.% )</td>
</tr>
</tbody>
</table>

![Fig. 1 – Section of UHPEPC specimens depending on the EPS by volume ratio](image_url)
The average values of density measured by three specimens are presented in Table 2. The density is in the range of 801.5 ~ 1,695.6 kg/m³ and decreases as the EPS replacement ratio increases. The test specimens cured at 90 °C showed that the density of UHPEPC for every 10% increase in EPS content by volume decreases by an average of about 223.5 kg/m³. For the specimens with EPS aggregates, it shows a wide compressive strength range of 4.81 ~ 65.1 MPa when the density is 801.5–1,695.6 kg/m³. The compressive strength of UHPEPC varies depending on the content of EPS beads. Compressive strength decreases by 15.09 MPa on average as EPS content increases from 30% to 70% by 10% in Fig. 2. Test results show that the strength of the concrete is greatly influenced by the curing method. The most important factors affecting the strength of concrete are curing temperature and curing time. Particularly, in the case of UHPC, it is effective to perform high-temperature curing in early ages to promote the hydration reaction resulting in the strength gain. Therefore, the strength of high performance expanded polystyrene concrete highly depends on temperature in early stage of curing. Lightweight aggregate concrete has a low density because it uses porous aggregates to lighten the concrete. However, the lightweight aggregate weakens the compressive strength of the concrete due to the weak strength of the aggregates.

![Fig. 2 - Relationship between compressive strength and EPS content by volume ratio](image)

When the flexural strength of UHPEPC was determined on specimens with the EPS content of 50%, 55%, and 60% respectively, it was found to be distributed in the range of 5.0 ~ 12.5 MPa. In Table 2, as the EPS content increases for specimens without steel fibers, the flexural strength is decreased and the higher strength is exhibited at 90 °C curing than at 60 °C curing. The flexural strength of UHPC is strongly influenced by the amount of steel fibers. In case of high performance expanded polystyrene concrete, because the strength is governed by UHPC, the incorporation of steel fibers is a very important parameter in measuring the flexural strength. When comparing the specimens that contain 50%, 55%, and 60% EPS with steel fibers of 2% volume ratio and the specimens without steel fibers, the flexural strength of specimens with steel fibers are 1.74, 1.5 and 1.24 times larger than those without fibers, respectively.

As shown in Table 2, the modulus of elasticity decreases by an average of about 0.05 GPa for every 10% increase of EPS content volume. It is increased by an average of about 0.05 GPa for an average 223.5 kg/m³ increase of the density and every 15.09 MPa increase of the compressive strength.

The Poisson’s ratio increases from 0.55 to 0.63 with increasing compressive strength as shown in Table 2. The shear modulus of elasticity also increased from 36.9 MPa to 202.6 MPa. The Poisson’s ratio of UHPEPC exceed the theoretical maximum value of 0.5 probably because of the volume changes due to the presence of voids inside and because the material is not homogeneous.

4. Thermal insulation performance of new materials

Thermal properties of four types of new materials including UHPEPC were investigated. EPS mortar was compared as an alternative core material, and two types of UHPC panels with different reinforcement - steel fiber of 2% volume fraction and glass fiber reinforced polymer (GFRP) mesh - were tested to be used for face sheets. It is common to incorporate steel fiber as a method to improve the flexural performance of UHPC. However, steel is a heavy material and has high thermal conductivity, making it an inefficient material for structures requiring heat performance or light weight. GFRP mesh belongs to textile reinforcement, and is expected to play a role of increasing the tensile strength of UHPC instead of steel fiber as a representative material with high thermal capacity and light weight. In this study, orthogonally netting mesh type of reinforcement was used to maximize the tensile strength of GFRP by securing the smoothness of the shell and the convenience of installation [11].

Three thermal properties were measured in this study. Firstly, k value (thermal conductivity) was measured. The ASTM Standard C168 [12] defines the term as follows: Thermal conductivity is the time rate of steady state heat flow through a unit area of a material induced by a unit temperature gradient in a direction perpendicular to that unit.
area. Secondly, $R$ value, thermal resistance is the quantity determined by the temperature difference, at steady state, between two defined surfaces of a material that induces a unit heat flow through a unit area. Finally, there is $U$ value, known officially as thermal transmittance. This is more of an engineering term used to designate the thermal performance of a system. Thermal transmittance is the heat transmission in unit time through unit area of a material and the boundary air films, induced by unit temperature difference between the environments on each side.

The thermal conductivity $k$, thermal transmittance $U$, and thermal resistance $R$ values are presented in Table 3. For the core material, the $U$ value and $k$ value of UHPEPC is about 3.34 times lower than that of EPS mortar. It means UHPEPC is a better material for insulation. For the material of face sheets, UHPC with GFRP face sheet has 1.43 times higher $U$ value and lower $R$ value than that of UHPC with steel fibers. The greater the performance of a piece of insulation, the greater its $R$ value. Figure 3 shows the surface temperature of specimens. It can be seen that the temperature difference between the top and bottom of UHPC with GFRP mesh specimen. It appears to be a phenomenon caused by peeling between UHPC and GFRP. On the other hand, the steel fibers are perfectly integrated with UHPC and maintains a tight structure resulting in better heat shielding effect.

Table 2 – Density and compressive strength of UHPEC

<table>
<thead>
<tr>
<th>ID</th>
<th>Bulk of EPS (%)</th>
<th>Volume fraction of Steel fibers (%)</th>
<th>Curing temp. (°C)</th>
<th>Density (kg/m$^3$)</th>
<th>Compressive strength (MPa)</th>
<th>Young’s modulus (GPa)</th>
<th>Flexural strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>7 days</td>
<td>28 days</td>
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<td></td>
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<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>90</td>
<td>2311.07</td>
<td>196.83</td>
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<td>-</td>
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<tr>
<td>2</td>
<td>30</td>
<td>0</td>
<td></td>
<td>1695.65</td>
<td>65.15</td>
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<td>-</td>
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<tr>
<td>3</td>
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<td>1551.71</td>
<td>60.07</td>
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<td>4</td>
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<td>43.65</td>
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<tr>
<td>5</td>
<td>60</td>
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<td></td>
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<td>21.23</td>
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<td>6</td>
<td>70</td>
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<td>801.55</td>
<td>4.81</td>
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<td>0</td>
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<td>17</td>
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<td>1545.95</td>
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<td>30.89</td>
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### Table 3 – Thermal properties of new materials

<table>
<thead>
<tr>
<th>T-specimen ID</th>
<th>Target area</th>
<th>Material type</th>
<th>Density (kg/m$^3$)</th>
<th>$k$ (W/mK)</th>
<th>$U$ (W/m$^2$K)</th>
<th>$R$ (m$^2$K/W)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Core</td>
<td>EPS mortar</td>
<td>1384.07</td>
<td>1.649</td>
<td>32.987</td>
<td>0.030</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>UHPEPC</td>
<td>1382.33</td>
<td>0.493</td>
<td>9.864</td>
<td>0.101</td>
</tr>
<tr>
<td>3</td>
<td>Face sheet</td>
<td>UHPC with GFRP mesh</td>
<td>2244.96</td>
<td>1.203</td>
<td>24.053</td>
<td>0.042</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>UHPC with steel fiber</td>
<td>2311.07</td>
<td>0.837</td>
<td>16.743</td>
<td>0.060</td>
</tr>
</tbody>
</table>

(a) EPS mortar  
(b) UHPEPC  
(c) UHPC with GFRP mesh  
(d) UHPC with steel fibers

Fig. 3 – Surface temperature of different new materials
Table 4 – Specimen details and flexural test results

<table>
<thead>
<tr>
<th>S-specimen ID</th>
<th>Core</th>
<th>Adhesive</th>
<th>Face sheet</th>
<th>Failure mode</th>
<th>Delamination</th>
<th>$P_{\text{max}}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M-U1</td>
<td>EPS mortar</td>
<td>Mortar</td>
<td>UHPC with steel fiber</td>
<td>Face sheet failure</td>
<td>n/a</td>
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<tr>
<td>M-G</td>
<td>GFRP</td>
<td></td>
<td></td>
<td>Core, bond failure</td>
<td>Observed</td>
<td>3.4</td>
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<tr>
<td>U-U1</td>
<td>Mortar</td>
<td></td>
<td>UHPC with steel fiber</td>
<td>Core failure</td>
<td>n/a</td>
<td>21.1</td>
</tr>
<tr>
<td>U-U2</td>
<td>UHPEPC</td>
<td>Mortar</td>
<td>UHPC with GFRP mesh</td>
<td>Face sheet failure</td>
<td>Partially observed</td>
<td>18.02</td>
</tr>
<tr>
<td>U-G</td>
<td>GFRP</td>
<td></td>
<td></td>
<td>Core, bond failure</td>
<td>Observed</td>
<td>15.29</td>
</tr>
<tr>
<td>U-U1-E</td>
<td>Epoxy bond</td>
<td></td>
<td>UHPC with steel fiber</td>
<td>Core failure</td>
<td>n/a</td>
<td>6.18</td>
</tr>
<tr>
<td>U-G-E</td>
<td>GFRP</td>
<td></td>
<td></td>
<td>Core failure</td>
<td>n/a</td>
<td>8.99</td>
</tr>
</tbody>
</table>

(a) U-U1 specimen

(b) U-G specimen

5. Flexural structural behavior of composite sandwich panels

To investigate the mechanical behavior of composite sandwich panels, the panels studied are constituted by core and face sheets and the influence of the three components – the mechanical properties of the core material, the strength of the face sheet material, and the bond strength adhesive material – was evaluated. The combination of the tested sandwich panel is shown in Table 4. The first character of S-specimen ID indicates core material and the second one indicates the face sheet material. The GFRP face sheet was manufactured using three different types of mats, embedded in a polyester resin matrix. The core thickness is 55 mm and the thickness of each face sheet is 5 mm. Flexural tests were conducted on each type of panels (one specimen for each type) according to ASTM C393 [13] standard in a four-point bending configuration. The sandwich panels which were 650-mm long, 320-mm wide and 65-mm thick, were tested in a 600-mm span and the loaded sections were distanced 200 mm apart. The supports were materialized by steel rollers. Composite sandwich panels were monotonically loaded up to failure. Test results are indicated in Table 4. Figure 4 shows the core failure of specimens with and without delamination.

All panels exhibited an approximately linear behavior up to failure of the core material. The EPS mortar core of specimen M-U1 cracked at the load of 6.41 kN, and then the sheet yielded subsequently at the load of 9.26 kN. M-G specimen collapsed because of the bond failure of core-to-facing interface, followed by core failure instantly. The flexural strength of specimens with EPS mortar core strongly depends on the face sheet capacity because the core capacity is relatively weaker than the flexural capacity of face sheets. The flexural capacity of the specimens with UHPEPC core showed high strength in a stable linear behavior.

Fig. 4 – Typical failure mode of flexural test results
before core crack. The maximum strength also depends on types of face sheet material. The core cracking load of U-U2 and U-G specimen recorded at 14.48 kN and 15.29 kN respectively. However, the maximum strength of U-U2 was 18.02 kN with a considerable deformation but U-G specimen failure right after core crack occurred. The U-U1 failed due to core cracking, but the stiffness and the maximum strength was greater than other specimens. The specimens bonded by epoxy failed by core cracking with low capacity although the core material was used as UHPEPC.

6. Conclusions

This study investigated the mechanical properties of ultra-high performance concrete with expanded polystyrene composite (UHPEPC) and the structural behavior of composite sandwich panels containing UHPEPC core experimentally. The conclusion from the research is as follows:

(1) The compressive strength, flexural strength, and modulus of elasticity of UHPEPC increases with increasing density. Material can be designed depending on the EPS content in large range of strengths and densities for various applications. The UHPEPC has superior mechanical properties when the density ranges between 1,200 ~ 1,500 kg/m³.

(2) The thermal resistance of UHPEPC is about 3.34 times lower than that of EPS mortar, which shows that UHPEPC can perform as a better insulation as a core material.

(3) From the flexural test results of sandwich panels, it can be concluded that elastic behavior of the composite sandwich panels depends on the core capacity and the post-core cracking behavior is governed by the types of face sheet material. The sandwich panel specimens with UHPEPC core shows outstanding flexural capacity except for applying epoxy as an adhesive material. UHPC reinforced by steel fiber and GFRP mesh enhanced flexural capacity with respect to the ultimate load and ductility, respectively.

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Technical Paper

Blast-resistance of ultra-high strength concrete and reactive powder concrete

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Abstract: Recent advances in nanotechnology research have been applied to improve the durability, serviceability, and safety of ultra-high performance concrete (UHPC). Furthermore, improvements in the compressive strength of concrete have allowed concrete structural members size and self-weight to be significantly reduced, which has in turn resulted in cost reduction and structural aesthetic enhancement. Among many UHPcs currently available on the market, the most representative ones are ultra-high strength concrete (UHSC) and reactive powder concrete (RPC). Even though UHSC and RPC have compressive strengths of over 100 MPa, their safety has been questioned due to possible ultra- brittle failure behavior and unfavorable cost-to-performance efficiency. The blast-resistant capacities of UHSC and RPC were experimentally evaluated to determine the possibility of using UHSC and RPC in concrete structures susceptible to terrorist attacks or accidental impacts. In addition, ANFO blast tests were performed on reinforced UHSC and RPC panels. Incidental and reflected pressures, as well as maximum and residual displacements and the strains of rebar and concrete were measured. Blast damage and failure modes of the reinforced panel specimens were recorded. The maximum displacement ratio of UHSC-NSC and RPC-NSC are 0.57, showing that UHSC and RPC have better blast resistance than NSC.

Keywords: ultra-high performance concrete (UHPC), ultra-high strength concrete (UHSC), reactive powder concrete (RPC), blast-resistant capacity, ANFO blast charge.

1. Introduction

The recent construction trends of building super-span bridges and mega-height high-rises mandate the use of ultra-high performance concrete (UHPC) due to its outstanding safety, serviceability, durability, and economical advantages [1,2]. Among many UHPcs available in the market, the most representative ones are Ultra High Strength Concrete (UHSC) using reinforcing steel with additives and Reactive Powder Concrete (RPC) using steel fibers with reactive mineral addition [1,3]. This study was performed to evaluate the blast resistance capacities of UHSC and RPC to determine whether these materials are suitable for use in structures susceptible to terrorist attacks or accidental impacts. In 2009, the Korean Building Code was modified to require terrorist-resistant designs for any high-rises located within the city limits of Seoul with an above-ground height of over 200 m or 50 or more floors above ground [4,5]. This code regulation reflects the public concern regarding possible terror attacks on buildings and structures in Korea. Because of the ultra-high strengths and energy absorption capacities of UHSC and RPC, they seem to be optimal materials for use in structures that are potential targets of terror attacks or accidental impacts. Therefore, in this study, the evaluation of the blast-resistant capacities of UHSC and RPC are carried out.

2. Specimen details

The panel dimensions were 1,000 × 1,000 × 150 mm. Two layers of D10 mesh reinforcements with 82-mm spacing in both directions were placed in the NSC and UHSC panel specimens. The yield and ultimate strength of the D10 reinforcement was 400 and 600 MPa, respectively, with a nominal cross-sectional area of 71.33 mm² and a unit weight of 0.56 kg/m. The reinforcement ratios of NSC and UHSC specimens were the same, whereas 2% volume of special short steel fibers were used in the RPC specimens. The selected mix proportions of
NSC, UHSC, and RPC are tabulated in Tables 1, 2, and 3, respectively. In Table 1, S1 is regular sand and S2 is micro-silica sand. Due to the patent copyright of the developer of the materials, the mix proportions of RPC and UHSC are listed as range values. The specific mixture contents are reported in the Korean patent. UHSC and RPC were steam-cured for 3 days at 90°C. Average compressive strength of NSC, UHSC, and RPC are 25.6, 202.1, and 202.9 MPa, respectively, as shown in Fig. 1 [9]. Compressive strength of UHSC and RPC is approximately 7.9 folds greater than that of NSC. Average elastic modulus of NSC, UHSC, and RPC are 16,300, 53,143, and 50,511 MPa, respectively, as shown in Fig. 1 [10]. Elastic modulus of UHSC and RPC is approximately 3.09–3.26 folds greater than that of NSC. As shown in Fig. 2, average split tensile strength of NSC, UHSC, and RPC are 2.2, 9.2, and 21.4 MPa, respectively. RPC have a higher resistance to split tensile strength than that of UHSC [11].

<table>
<thead>
<tr>
<th>Max. size of coarse aggregate (mm)</th>
<th>Target strength (MPa)</th>
<th>Slump (mm)</th>
<th>W/B (%)</th>
<th>S/a (%)</th>
<th>Unit water (kg)</th>
<th>Unit binder (kg)</th>
<th>Unit fine aggregate (kg)</th>
<th>Unit coarse aggregate (kg)</th>
<th>AE admixture (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>25</td>
<td>24</td>
<td>100</td>
<td>49.8</td>
<td>47.7</td>
<td>163</td>
<td>294</td>
<td>33</td>
<td>616</td>
<td>264</td>
</tr>
</tbody>
</table>

Table 1 – Mix proportion design of normal strength concrete (NSC) [7]

<table>
<thead>
<tr>
<th>W/B (%) less than</th>
<th>S/a (%) less than</th>
<th>Unit water (kg) less than</th>
<th>Unit binder (kg) less than</th>
<th>Unit fine aggregate (kg) less than</th>
<th>Unit coarse aggregate (kg) less than</th>
<th>AE admixture (%) range of</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>39.1</td>
<td>140</td>
<td>1300</td>
<td>450</td>
<td>700</td>
<td>1 to 3</td>
</tr>
</tbody>
</table>

Table 2 – Mix proportion design of ultra-high strength concrete (UHSC) [7]

<table>
<thead>
<tr>
<th>W/B(%) less than</th>
<th>Cement (kg) less than</th>
<th>Unit water (kg) greater than</th>
<th>Silica fume (%) range of</th>
<th>Unit fine aggregate (kg) range of</th>
<th>Filler (2.2–200 μm) (kg) greater than</th>
<th>Admixture (%) range of</th>
<th>Steel fiber (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>800</td>
<td>200</td>
<td>10 to 30</td>
<td>800 to 1000</td>
<td>200</td>
<td>1 to 3</td>
<td>2</td>
</tr>
</tbody>
</table>

3. Blast-resistant capacity

The blast-resistant capacity of reinforced UHSC and RPC panels under ammonium nitrate/fuel oil (ANFO) blast loading is evaluated [12]. The experiments were carried out at the test site of the Agency for Defense Development of Korea located near the Military Demarcation Line (MDL).

Fig. 1 – Average compressive strength and elastic modulus of NSC, UHSC, and RPC

Fig. 2 – Split tensile strength of NSC, UHSC, and RPC
Fig. 3 – Configuration photo of buried supporting frame

(a)

Fig. 4 – Measurement sensor locations: (a) pressure-meter placement setup photo, (b) strain gauge locations [7]
15.88 kg of ANFO and a standoff distance of 1.5 m were selected from the trial test before main test.

3.1 Blast test details

As shown in Fig. 3, a steel frame is buried in the ground for the placement of the specimen to eliminate the ground reflection effect [4-7]. The supporting steel frame made using SM520 with 7-mm thickness were attached with stiffeners at 250 mm spacing to prevent the frame distortion during blast loading. The clamp was provided to prevent uplifting of the test specimen. Free-field incident pressure and reflected pressure were measured at distances of 5 m and 1.5 m away from the center of the blast charge, respectively, as shown in Fig. 4(a). The reflected pressure transducers were placed on the top surface of the specimens, at the center and at 230 mm from the center, 1/3 of the diagonal distance from the center to the corner as shown in Fig. 4(b). To measure wave impact acceleration, an accelerometer was attached on the top center of the specimens and linear variable differential transformers (LVDTs) were placed on the bottom surface to measure maximum and residual vertical displacements.

3.2 Blast test results

3.2.1 Surface examination and crack patterns

Schematic drawings of the bottom surface crack patterns of NSC, UHSC, and RPC panel specimens are shown in Fig. 5. One-directional multiple medium length macro-cracks bisected the middle of the RPC specimens. This crack pattern was expected for RPC, because RPC is a cement mortar reinforced with short steel fibers; crack control by the fibers prevented catastrophic macro-crack propagations, resulting in the formation of medium length macro-cracks only in the direction perpendicular to the principle tensile strain direction as shown in Fig. 6(c). The macro-crack means visually observable crack.
Because both UHSC and RPC specimens failed due to macro-cracks, it is safe to assume that they failed in a quasi-brittle manner even under the flexure mode because of their ultra-high compressive strengths. The lack of shear cracks on the specimens led to a conclusion that the shear capacities of UHSC and RPC are sufficient to withstand blasts. In summary, the failure patterns of UHSC and RPC indicate that they are much more resistant to blast loading than NSC and have superior blast-resistant capacities. Furthermore, because relatively fewer cracks were found in these specimens than in the NSC specimens, it would require less effort and cost to repair blast-damaged UHSC and RPC members than NSC members.

3.2.2 Blast pressure measurements

The pressure comparison results are shown in Fig. 7. The second peak pressure obtained from the experiment of UHSC and RPC were approximately 18% and 30% less than the first peak pressure predicted by ConWEP, respectively. ConWEP software is an analytical program used to calculate the blast loadings of blast pressure, fragmentation, surface impact, etc. based on Unified Facilities Criteria (UFC) 3-340-01 [13]. These results indicated that reflected pressure is highly dependent on experimental variabilities and environmental conditions, validating the implementation of a magnification factor in the ConWEP calculation [5,8]. The experimental data were inconsistent due to experimental variations and environmental conditions (i.e., charge shape, charge angle, wind velocity, humidity, etc.). A second peak pressure followed the first peak overpressure at the center of the specimen for both reflected and free field pressures. This could be ascribed to the finite time duration of the explosion of an ANFO charge, resulting in a relatively slower detonation speed. Due to the continuous explosion characteristics of an ANFO charge, the reflected and re-reflected pressures are combined, creating different applied pressures and several peak overpressures as shown in Fig. 7.

3.2.3 Deflection measurements

The center point deflection histories of NSC specimens with a 15.88 kg ANFO charge are shown in Fig. 8(a), while the center point deflection histories of the UHSC and RPC specimens with an ANFO charge of 15.88 kg are shown in Fig. 8(b) and Fig. 8(c), respectively. The maximum and residual deflections of the NSC, UHSC, and RPC specimens from the 15.88 kg ANFO charge were 18.57 and 5.79 mm, 15.238 and 5.65 mm, 10.73 mm and 3.202 mm, respectively, as shown in Fig. 8. These results indicate that RPC has the best blast-resistant capacity followed by UHSC and then RPC. This is a reasonable result, because the blast resistance of RPC is significantly enhanced by the presence of short steel fibers, which provide improved crack-bridging characteristics and energy absorption capacity.

3.2.4 Strain measurements

The strains measured in this study are shown in Fig. 9. Because RPC specimens do not have reinforcing bars, steel strain measurements were only obtained from the NSC and UHSC specimens. The strain data indicate bottom reinforcement yielding in all specimens, with higher strains occurring in the reinforcements towards the center of the specimen. The maximum strains measured from bottom reinforcement of the NSC and UHSC specimens were approximately 28,000 and 6,500 με, respectively, as shown in Fig. 9. These results indicated that smaller displacements occurred in the UHSC specimens than in the NSC specimens, confirming that UHSC has better blast-resistant capacity than NSC.

4. Conclusions

In this study, the blast-resistant capacities of ultra-high strength concrete and reactive powder concrete were experimentally evaluated. The results showed that they have outstanding blast-resistant capacities. The conclusions of this study are summarized as follows:
Fig. 8 – Center displacement versus time measurements from 15.88 Kg ANFO blast loading: (a) NSC, (b) UHSC, (c) RPC

Fig. 9 – Reinforcement bar strain versus time measurements from blast loading: (a) NSC, (b) UHSC
The blast-resistant capacities of UHSC and RPC were verified by blast tests using a 15.88 kg ANFO charge with a 1.5 m standoff distance, applying a blast load with strain rate of 278~457 s$^{-1}$. Pressure, deflection, and strain from the blast tests revealed that UHSC and RPC panel specimens have higher blast-resistant capacities than NSC specimens.

Rebar and short steel fibers used in the UHSC and RPC specimens, respectively, negate the brittle material characteristics of UHSC and RPC members, provide sufficient ductility, and confer outstanding energy absorption and crack controlling capacities to these materials.

Acknowledgements

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References

Technical Paper

Effects of different fine aggregates on the properties of ultra-high strength concrete

Taku Matsuda* and Takafumi Noguchi

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Abstract: When a coarse pore-structured fine aggregate with a large water absorption was used for concrete with a water-binder ratio of 0.12, unit water content of 135 kg/m$^3$, and sand-aggregate ratio of 55%, the mobility, strength, and elastic modulus of concrete increased, with the autogenous shrinkage becoming extremely low. The authors attributed the reason for this phenomenon to the internal curing effect of fine aggregate based on tests using mortar samples and the properties of fine aggregate. The authors also considered that one reason for the increased strength of concrete is the reduction in the autogenous shrinkage of the mortar phase, which reduces the risk of failure under local stress, based on the stress-strain relationship of concrete and mortar under compressive deformation.

Keywords: ultra-high-strength concrete, fine aggregate, internal curing, pore structure.

1. Introduction

In their earlier study [1,2], the authors investigated the effect of using silica fume and fly ash in combination as supplementary cementitious materials (SCMs) for ultrahigh strength concrete with a water-binder ratio (W/B) of 0.20 or less. As a result, concrete made using both SCMs, silica fume and fly ash, led to a lower viscosity than with silica fume alone. Therefore, sufficient mobility was achieved even by reducing the unit water content (W) from 150 kg/m$^3$ to 135 kg/m$^3$ and increasing the sand-aggregate ratio (s/a) from 27% to 55%. The strength and elastic modulus thus increased and the autogenous shrinkage significantly decreased. (Figure 1 shows part of the results of the study [2]). However, these are results of tests conducted using specific materials (low-heat portland cement, silica fume, fly ash, ferronickel slag fine aggregate, and crushed hard sandstone). The purpose of the present study is to examine the applicability of the above-mentioned results to ultrahigh strength concrete made using different materials, while considering the reason why different materials lead to different properties of concrete (mobility, mechanical properties, and autogenous shrinkage), thereby acquiring knowledge that contributes to the development of proportioning techniques.

Fig. 1 – Part of the results of the early study [2]

Table 1 gives the materials used in the present tests, which were conducted in two series. The binders are premixed cement (SFPC) made by replacing a part of low-heat-type cement with silica fume, and fly ash (FA) conforming to Type I specified in JIS A 6201; the fine aggregates are andesite (S1), hard sandstone (S2), and two types of ferro-nickel slags (S3 and S4); the coarse aggregates are andesite (G1) and two types of hard sandstone (G2 and G3). G2 and G3 from different sources are
2. Series 1

2.1 Test procedure and measurement items

Tables 2 and 3 give the mixture proportions of concrete and the types and curing conditions of specimens, respectively. The W/B of all mixtures is fixed to 0.12. Con-1 and Con-2 in Table 2 represents mixtures containing silica fume but no fly ash, whereas Con-3 to Con-9 represent mixtures containing both silica fume and fly ash. Con-1 to Con-4 are proportioned with W = 155 kg/m³ and s/a = 29%; Con-5 to Con-7, with W = 135 kg/m³ and s/a = 55%; and Con-8 and Con-9 contain W = 125 and 95 kg/m³, respectively. In series 1, mixtures are distinguished by the type of fine aggregate.
Mixtures containing S1 as fine aggregate, for instance, are referred to as S1 mixtures for convenience sake, where appropriate. Wet-screened mortar was taken from each concrete mixture and cured under the same conditions as concrete. A twin-shaft forced-action mixer was used for mixing concrete, with the chemical admixture (SP) being added at the same dosage for all mixtures (B x 2.30%). The air content and temperature of fresh concrete ranged from 2.0 to 3.0% and 28.5 to 32.0°C, respectively. The autogenous shrinkage was measured by strain transducers embedded in beams measuring 100 x 100 x 400 mm.

2.2 Test results and discussion

Figures 2 through 4, and Figure 6 show the results of slump flow tests, compression tests, elastic modulus tests, and autogenous shrinkage tests, respectively. Before going into the details, the results of Con-4 and Con-5 with fine aggregate S2 and coarse aggregate G2 are compared in these figures. A reduction in W from 155 to 135 kg/m^3 and an increase in s/a from 29 to 55% result in a reduction in the mobility but no marked changes in the strength, elastic modulus, and autogenous shrinkage. This is totally different from the tendencies of the earlier reports (Fig. 1) [2]. In other words, the properties of ultrahigh strength concrete can widely vary depending on the materials.

2.2.1 Mobility

As to the slump flow test results (see Fig. 2), the results of Con-1 to Con-4 demonstrate that the mobility of specimens containing both silica fume and fly ash is higher than that of specimens containing silica fume but no fly ash. No marked difference in mobility is observed between the aggregate combinations of S1 + G1 and S2 + G2 (between Con-1 and -2 and between Con-3 and -4), as well as between coarse aggregates G2 and G3 (between Con-6 and -7). On the other hand, replacing S2 with S3 significantly reduces the viscosity (Con-5 to Con-6), resulting in concretes with viscosities lower than Con-1 and Con-2 even with a W of as low as 95 kg/m^3. This is presumably because the mobility of the mortar phase of the S3 mixtures is higher than that of the other mixtures (This is to be verified using mortar samples later in Section 3 of this paper).

2.2.2 Mechanical properties

In the compression test results (see Fig. 3), as for S1 mixtures, the strength of Con-3 with combined SCMs of silica fume and fly ash is equivalent to those of specimens with a single SCM of silica fume (Con-1) after heating to 90°C, though the strength of the former is lower than the latter when kept at 20°C, similarly to the results of the earlier report [1]. Also, for S2 mixtures, roughly same tendency also holds for the relationship between Con-2 and Con-4.

The mortar strength of all mixtures is higher than the concrete strength. In regard to Con-3 to Con-9 made using the same binder, both the concrete and mortar strengths are S1 ≈ S2 < S3 mixtures when these are kept at 20°C. After heating to 90°C, the mortar strength of all mixtures exceeds 240 N/mm^2, with the comparison being S1 ≈ S2 < S3 mixtures. However, the concrete strength is S1 < S2 < S3 mixtures, with the gap between the mortar and concrete strengths of S1 mixtures being particularly wide (The reason for this is discussed at the end of this section and in Section 3). Accordingly, one reason for the high concrete strength of S3 mixtures compared with those of S1 and S2 mixtures is the high strength of its mortar phase.
As seen from Fig. 4, the elastic moduli of concrete and mortar specimens Con-3 to Con-9, which are made using the same binder components, are both in the order of S1 < S2 < S3 mixtures. The gradients of the stress-strain curves of concrete and mortar are nearly the same for S1 and S3 mixtures, whereas the gradient of the concrete curve for S2 is steeper than that of the mortar curve. Also, the compressive strain at break of each mortar specimen is greater than that of the corresponding concrete specimen.

Figure 5 shows the relationship between the gap between the strains of mortar and concrete at failure in compression, $\Delta \varepsilon$ $[\times 10^{-6}]$, and the elastic modulus ratio ($E_m/E_c$). The stiffnesses of hard sandstone G2 and G3 are assumed to be the same here, since no marked difference is observed between the results of Con-6...
and Con-7 in which the proportioning conditions excepting coarse aggregate are the same. Based on this assumption, the mechanical properties of these mixtures are discussed. To begin with, S2 and S3 mixtures containing coarse aggregates with the same stiffnesses are examined. The stiffness of concrete falls between those of mortar and coarse aggregate, and the closer the stiffnesses of mortar and coarse aggregate, the closer the stiffnesses of concrete and mortar becomes. This suggests that the stiffness of the mortar phase of S3 mixtures is closer to those of coarse aggregate (G2 and G3) than that of S2 mixtures. According to the literature, the elastic modulus of hard sandstone (G2 and G3) falls in the range of 56.5 to 71.0 kN/mm² [3,4]. Based on this and the values in Fig. 4, the elastic moduli are roughly ranked as [the mortar phase of S2 mixtures < the mortar phase of S3 mixtures ≤ coarse aggregate (hard sandstone)]. The present discussion is thus considered to be valid. The stress distribution in the concretes of S3 mixtures under compressive loading is therefore more uniform than that in S2 mixtures. It follows that the former’s concretes are less prone to failure under stress concentration due to the gap between the stiffnesses of coarse aggregate and mortar [5,6]. In other words, $E_{\text{m}2}/E_c$ of S3 mixtures is closer to 1.0 than that of S2 mixtures, with $\Delta \varepsilon$ being smaller (see Fig. 5). This explains the fracture strength of S3 mixtures being higher than that of S2 mixtures (see Fig. 3).

As for S1 mixtures, the stress-strain curves of concrete are shaped similarly to those of mortar (Fig. 4), with their $E_{\text{m}1}/E_c$ distributing closer to 1.0 than those of S3 mixtures (Fig. 5). However, $\Delta \varepsilon$ of S1 mixtures is evidently greater than that of S3 mixtures. According to the literature, the strength of andesite, the coarse aggregate of S1 mixtures, falls in the range of 182 to 218 N/mm² [3,4,7]. The concrete strengths of S1 mixtures heated to 90°C and those kept at 20°C are 190 to 196 N/mm² and 146 to 157 N/mm², respectively. Therefore, there is a possibility that the large $\Delta \varepsilon$ of concrete heated to 90°C is due to the coarse aggregate failure before mortar failure. However, $\Delta \varepsilon$ of concrete kept at 20°C is greater than that of S3 mixtures, despite the fact that the strength of concrete kept at 20°C is lower than that of concrete heated to 90°C and therefore less prone to aggregate failure. This demonstrates the presence of a factor that determines the compressive strength of concrete other than the strength and stiffness of coarse aggregate and the mortar phase (This factor is discussed in section 3).

### 2.2.3 Autogenous shrinkage

According to the results of autogenous shrinkage tests (Fig. 6), the autogenous shrinkage of mixtures containing SCMs of silica fume and fly ash (Con-3 and Con-4) is smaller than that of mixtures solely containing silica fume as a SCM (Con-1 and Con-2). Also, when comparing among mixtures in which the proportioning conditions are the same excepting fine aggregate, the autogenous shrinkages are found to be S2 mixtures < S1 mixtures from Con-1 to Con-2, and from Con-3 to Con-4. It is also found from Con-5 to Con-6 that the autogenous shrinkages are S3 mixtures < S2 mixtures. Since the autogenous shrinkage of Con-6 is significantly smaller than that of Con-5, the autogenous shrinkage of the mortar phase of S3 mixtures is considered to be extremely small (The reason for this is discussed in section 3).

Based on the discussion regarding series 1, it can be said that the strongest impact on the properties of concrete is brought about by the difference of fine aggregate.

### 3. Series 2

Series 2 ascertains the differences in the mortar properties due to differences of fine aggregate by testing and discusses (1) the effect of the differences in the mortar properties on the concrete properties and (2) the reason why the differences of fine aggregate cause differences in the mortar properties.

#### 3.1 Test procedure and measurement items

In addition to fine aggregates S1 to S3, ferro-nickel slag S4 was included in the investigation. The saturated surface dry (SSD) density of S1 and S2 is the same, but the water absorption of S1 is greater than that of S2. As for ferro-nickel slags S3 and S4, their SSD densities are the same, but the
water absorption of S3 is greater than that of S4. As for andesite S1 and ferronickel slag S3, their water absorptions are similar, but the SSD density of S3 is greater than that of S1. Tables 4 and 5 give the mixture proportions and fresh properties of mortar and the types and curing conditions of specimens, respectively. The W/B and Vs/Vmor are 0.18 and 0.44, respectively, to increase the W/B from the value for Series 1 (W/B = 0.12, Vs/Vmor = 0.17 to 0.43) to reduce the viscosity of the paste, while increasing the fine aggregate content, in expectation of emphasizing the effect of fine aggregate on the properties of mortar. A paddle mixer was used for mixing, with the dosage of SP being fixed (B = 1.40%) for all mixtures. In the autogenous shrinkage testing, uniaxial restraining tests using D10 deformed bars were conducted similarly to an earlier report \[8\]. Samples cured under the same conditions as strength test specimens were pulverized at the time of strength testing and dried at 105°C to measure the amount of evaporating water. The pore size distribution was also measured by mercury intrusion.

### 3.2 Test results and discussion

#### 3.2.1 Mobility

The mobility of mortar made using S3 or S4 (S-3 and S-4) is found to be higher than that of mortar made using S1 or S2 (S-1 and S-2), as the 0-impact flow of the former is larger and the JP funnel flow through time of the former is shorter (The mobility of concrete made using this mortar is therefore greater as shown in Table 4).

![Fig. 7 – Compressive strength and elastic modulus (series 2)](image_url)

#### 3.2.2 Mechanical properties

The compressive strengths are S-1 ≈ S-2 ≈ S-4 < S-3, regardless of the curing conditions. The elastic moduli of S-3 and S-4 are higher, regardless of the curing conditions (see Fig. 7). The high strength and elastic modulus tendencies of S-3 are analogous to mortars in series 1.

#### 3.2.3 Autogenous shrinkage

Figure 8 shows the results of autogenous shrinkage tests. The results until an age of 2 days,
in Fig. 8(a), (b), and (d), shows the difference in their early behavior. The exotherm and shrinkage begin at the earliest time in S-1, followed by S-2, and then simultaneously by S-3 and S-4. Both the autogenous shrinkage strain and restraining stress are ranked as $S_3 < S_4 < S_2 < S_1$, with the autogenous shrinkage of $S_3$ being found to be extremely small. This is considered to be the “factor that determines the compressive strength of concrete other than the strength and stiffness of coarse aggregate and the mortar phase” mentioned in Section 2.2.

Concretes of $S_3$ mixtures are less prone to the risk of local stress fracture under compressive loading because of the low internal stress, which is generated by coarse aggregate restraining the autogenous shrinkage of the mortar phase. One reason for the large autogenous shrinkage of $S_1$ is presumed to be the low elastic modulus of mortar, which is caused by lower stiffness of fine aggregate $S_1$ than the others, leading to a lower degree of autogenous shrinkage restraint in the pastes. Another possibility is the effect of the shrinkage of fine aggregate itself (the shrinkage of $S_1$ is greater than those of the other fine aggregates). These subjects remain to be tackled in the future. Meanwhile, the autogenous shrinkage of $S_3$ is extremely small, though the elastic moduli of the mortars of $S_3$ and $S_4$ are similarly high. The differences between fine aggregates $S_3$ and $S_4$ include the large water absorption of $S_3$ ($S_3$: 2.91%, $S_4$: 0.95%).
It is therefore possible that the autogenous shrinkage of S-3 is reduced by the internal curing effect whereby the paste is supplied with water from fine aggregate [9].

3.2.4 Effects of fine aggregates on the mortar properties

Figure 9 shows the pore size distribution of fine aggregate. The peak pore size varies depending on the fine aggregate type, being $S_1 < S_2 \approx S_3 < S_4$. While the water absorptions of $S_1$ and $S_3$ are similarly large, the pore structure of $S_3$ is coarser than that of $S_1$, being easier to releasing water, bringing about a greater internal curing effect. Although, $S_2$ and $S_4$ are similarly or more prone to releasing water in terms of the peak pore size, but their internal curing effect is weaker presumably due to the low water absorption. To verify this hypothesis, a SSD sample of approximately 100 g of each fine aggregate was placed on a dish for monitoring the mass changes (changes in the moisture content) in an environment of 20°C and 60% R.H. (see Fig. 10 (a)). The moisture content of $S_3$ decreases at a higher ratio than that of $S_1$. The reduction nearly leveled off by 24 h later, but the ultimate value of $S_1$ remains high at around 2% in contrast to $S_3$ at nearly 0%. Figure 10 (b) shows the amount of evaporated water converted to the volumetric ratio to the SSD volume of fine aggregate (on the vertical axis on the left) and the amount per fine aggregate content of the test mortar (on the right). This figure reveals that the water supply from $S_3$ in the paste is evidently larger than those from the other aggregates.

Also, Figure 8 (d) shows the restraining stress of $S_3$ turning to the negative side, demonstrating that its autogenous shrinkage behavior is on the expansion side at early ages. This can be a phenomenon similar to the case reported in the literature [10] in which artificial lightweight fine aggregate used for ultrahigh strength concrete was found to expand.

Figure 11 shows the unevaporated water ratios of the fine aggregates, which is here defined as $\frac{[(\text{unit water content} + \text{amount of water absorption of fine aggregate}) \text{converted to the mass of sample}}{-\text{amount of evaporated water}} \text{per unit mass of binder (B)} \text{[wt%]}$ based on the results of the measurement of evaporated water in mortar samples. The values of $S_3$ are prominently large, presumably because water contained in $S_3$ is supplied to the paste, contributing to hydration. This indirectly explains the above-mentioned discussion. Accordingly, the internal curing effect of $S_3$ not only reduces autogenous shrinkage but also increases strength and elastic modulus.

Figure 12 shows the relationship between the 0-impact flow and the age at which the shrinkage stress developed. The 0-impact flows of $S_1$ and $S_2$ are smaller than those of $S_3$ and $S_4$. To explain this, it is necessary to consider their physical aspects including the shape and grading of fine aggregate. On the other hand, the reason for the extremely early exotherm and shrinkage initiation can be attributed to the effect of the adsorption of SP by fine aggregate, as pointed out in a past study [11]. It can be considered that fine aggregate $S_1$ in $S_1$ adsorbs a large amount of SP, and this weakens the cement-dispersing effect in mortar, reducing the mobility, while accelerating hydration. $S_1$ and $S_4$ with coarse pore structures, which are easier to release water, are less likely to adsorb SP.

Accordingly, it is presumed that, under proportioning conditions with a low unit water content and high fine aggregate content, the effects of competitive adsorption of water and a chemical admixture by the binder, hydrates, and fine aggregate on the mobility, mechanical properties, and shrinkage properties are significant. The results of our earlier report [2] shown in Fig. 1 explain the phenomenon in which an increase in the content of fine aggregate with a coarse and water-absorptive pore structure increases the internal curing effect. It is necessary from now on to confirm the behavior of water and a chemical admixture in fine aggregate.
4. Conclusions

The following conclusions can be drawn in this study:

(1) When a coarse pore-structured fine aggregate with a large water absorption was used for concrete with a water-binder ratio of 0.12, unit water content of 135 kg/m³, and sand-aggregate ratio of 55%, the mobility, strength, and elastic modulus of concrete increased, with its autogenous shrinkage becoming extremely low. This is considered to be due to the internal curing effect of fine aggregate.

(2) One possible reason for the increased strength of concrete mentioned in (1) above is the reduced autogenous shrinkage of the mortar phase, which reduces the possibility of failure induced by local stress.

(3) The internal curing effect of fine aggregate mentioned in (1) above presumably increases as the unit water content decreases and as the fine aggregate content increases.

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