Proposal of a fatigue life prediction method for RC slabs failed under traveling wheel-type load test  1-11
Kyoko TAKEDA, Natsuko HAMADA and Yasuhiko SATO
Quality improvement of recycled concrete aggregate by a large-scale tube mill with steel rod  12-21
Lapyote Prasittisopin, Chawis Thongyothee, Phattarakamon Chaiyapoom and Chalermwut Snguanyat
Effect of pre-loading on chloride diffusion in concrete  22-28
Truyen T. Tran, Quyet V. Truong, H. Ranaivomanana and A. Khelidj
Performance evaluation of basalt fiber reinforced mortar under freeze-thaw and chloride-rich environments  29-34
Yiming Guo and Hiroshi Yokota
Comparison of concrete strength from cube and core records by bootstrap  35-46
Saha Dauji and Kapilesh Bhargava
Investigation on quality of thin concrete cover using mercury intrusion porosimetry and non-destructive tests  47-66
Liyanto Eddy, Koji Matsumoto, Kohei Nagai, Piyaphat Chaemchuen, Michael Henry and Kota Horiuchi

Tensile behavior of UHPFRC under uniaxial and biaxial stress conditions  67-78
Xiujiang Shen and Eugen Brühwiler
Deformation mechanism of hardened cement paste under high stress and application of flow law  79-88
Yuya Sakai
Strength, shrinkage and creep of concrete including CO2 treated recycled coarse aggregate  89-102
Gomboasuren Chinzorigt, Donguk Choi, Odontuya Enkhbold, Batzaya Baasankhuu, Myung Kwan Lim and Hee Seob Lim
Visual investigation method and structural performance evaluation for DEF induced damaged Indian Railway PC sleepers  103-115
Rajamurugan Sundaram, Koji Matsumoto, Kohei Nagai and Anupam Awasthi
Technical Paper

Tensile behavior of UHPFRC under uniaxial and biaxial stress conditions

Xiujiang Shen* and Eugen Brühwiler

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Abstract: Representative and accurate characterization of the tensile behavior of strain hardening Ultra High Performance Fiber Reinforced Cementitious Composite (UHPFRC) remain a challenge. Currently, the uniaxial methods, like direct tensile test (DTT) and 4-point bending test (4PBT), are commonly applied, although the biaxial tensile condition has been widely recognized in the UHPFRC applications, e.g. thin UHPFRC layers as external reinforcement for RC slabs. In this paper, results from ring-on-ring testing of circular slab-like specimens are presented to determine the equi-biaxial tensile response by means of inverse analysis using 3D finite element method (FEM). In addition, DTT, using structural specimens cut from large square plates, and 4PBT, using standard specimens cast in mould individually, were carried out. The tensile response from 4PBT was derived through inverse analysis using 2D FEM. Finally, the corresponding results from the three different testing methods under either uniaxial or biaxial stress condition were analyzed and compared in terms of tensile characteristic parameters, tensile material law, fracture process, and energy absorption capacity. While the three testing methods did not show significant difference in tensile strength, significantly higher strain hardening deformation was identified in the case of biaxial stress conditions.

Keywords: biaxial stress condition, FEM, inverse analysis, ring-on-ring test, UHPFRC, uniaxial stress condition, tensile behavior.

1. Introduction

The tensile response is a fundamental constitutive property of strain hardening Ultra High Performance Fiber Reinforced Cementitious Composite (UHPFRC), so the accurate and representative characterization of this response is necessary for the design of a given UHPFRC application. In general, this characterization is achieved by means of uniaxial test methods, especially direct tensile test (DTT) and 4-point bending test (4PBT) using small-scale laboratory specimens casting in moulds individually. Unfortunately, these tests exhibit considerable scatter and the results are often considered as an upper bound in case of small-scale laboratory specimens, hardly reproducing real design situations. Most infrastructures, in particular bridge decks and floors, are principally under biaxial stress condition, far from uniaxial stress state [1]. In this context, the actual tensile performance of UHPFRC under biaxial stress condition should be investigated and compared with that from uniaxial stress condition carefully.

The DTT using dumbbell-shaped specimen is commonly applied to determine directly the uniaxial tensile properties of UHPFRC for given preparation conditions (moulds, casting, and curing). For reliable results, wise design and preparation are required when conducting DTT. In order to avoid largely the initial eccentricity with bending effects, the specimen was built-in the testing machine by applying the principle “gluing without adherence”, developed by Helbling & Brühwiler [2]. This method was also applied in ref. [3,4]. Proposed by Graybeal et al. [5,6], the tapered aluminum plates were fixed to both sides of each specimen end to ensure final fracture occurs out of the central constant part. Otherwise, one or two layers of steel wire mesh were used to strengthen each end of specimen, as applied in ref. [7,8]. Additionally, boundary conditions also have important influence on test results, and the fixed conditions is
recommend for reliable estimation of tensile response of UHPFRC after elastic limit, as confirmed by Kanakubo [9]. The 4PBT, alternatively, was used successfully to identify the tensile property of UHPFRC indirectly by means of inverse analysis, including analytical method and FEM [10–12].

Regarding direct biaxial tests, four actuators and a big frame are generally necessary, and inherently, many challenges pertaining to uniform load distribution, frictional effect, accurate boundary condition and load control need to be addressed carefully. Therefore, only few studies have been conducted on the biaxial behavior of concrete [13,14], especially no experimental study on biaxial behavior of UHPFRC has been recorded through direct biaxial test. Recently, the ring-on-ring test, as a 3D version of 4PBT, has been developed to investigate the biaxial flexural strength of concrete [15–17]. This method was extended to UHPFRC by several researchers [1]. The limited test results show that the ring-on-ring test allowed the actual development of fiber bridging effects between cracks, thus accurately representing the behavior of UHPFRC members subjected to biaxial flexural loading.

In this paper, the ring-on-ring test on circular slab-like specimens has been developed to determine the equi-biaxial tensile response. In addition, direct tensile tests using dumbbell specimens cut from large square plates and 4PBT using small plates cast in molds were carried out. The corresponding tensile response from five DTT, six 4PBT and four ring-on-ring tests were analyzed and compared (See Fig. 1). The main objective was to examine the differences and relationships of the tensile response of UHPFRC under uniaxial and biaxial stress conditions, and to propose the most appropriate test method to determine the tensile property for a given UHPFRC application.

2. Experimental Program

2.1 Ring-on-ring test

The ring-on-ring test method was applied for indirect characterization of the tensile behavior under biaxial stress condition, using circular slab-like specimens with a diameter R = 600 mm and a thickness h = 50 mm. This method has been extensively adopted and even standardized by ASTM [18] in the ceramics and glass domain. Recently, this method was modified and validated to measure the biaxial flexural strength of concrete and UHPFRC [15–17,19,20]. The updated ring-on-ring test yielded stable test results with small scatter, and it is promising to be a reliable and rational means to investigate biaxial flexural behavior of UHPFRC.

Figure 2 shows the full test set-up and devices applied in this experimental campaign. The slab was simply supported on a steel support ring with R = 500 mm. Loading was imposed by a hydraulic jack acting on the center of slab through a steel force transmitting ring with r = 150 mm. All the slabs were subjected to three loading–unloading cycles to 20 kN with an actuator displacement rate of 1.0 mm/min. Afterwards, monotonic loading with the same displacement rate was applied up to the peak force, followed by a rate of 4.0 mm/min until the actuator displacement reached 80 mm. Under loading, the uniform stress is introduced on the bottom surface within the force transmitting ring area, where biaxial stress condition is assumed.

![Fig. 1 – Approach for the comparison of the tensile response of UHPFRC under uniaxial and biaxial stress conditions](image-url)
The slabs were tested with the casting surface facing upwards, allowing the observation of tensile crack propagation on the smooth sheathed surface. Before testing, the casting surface was polished and a mortar layer was placed between support ring and bottom surface to level out both surfaces. Two rubber pads (thickness of 10 mm, $E = 500$ MPa) were positioned between the slab surfaces and the two rings to distribute the force evenly.

As illustrated in Fig. 2, Digital Image Correlation (DIC) technique was applied to observe the deflection development, strain field, and micro-cracking during the whole testing process. Two digital cameras were placed underneath the slab at a distance of 0.5 m and an angle of 23 degrees to the vertical. The tensile surface of the slab was painted with matte white paint, and then spayed black speckle pattern with size less than 1 mm. The targeted area, which was visible to the DIC, was about Ø500 mm on the center of the slab. In such case, the DIC measurement accuracy can reach around 5 $\mu$e. In addition, several LVDTs were installed on the top surface to measure the deflection. All deflection measurements were performed with respect to the strong floor. The measurement frequency was 5 Hz. Further details about the ring-on-ring test applied in this study can be found in [21].

2.2 Direct tensile test (DTT)

The dumbbell shaped specimens, with a constant cross section of 80 mm × 50 mm at the central part, were adopted for uniaxial DTT. The geometry of specimen was designed based on the equation of Neuber’s spline [22,23]. In total five specimens were extracted from a large square plate (1,100 mm × 1,100 mm × 50 mm) with the same thickness and casting procedure as that for the circular slab-like specimen (See Fig. 3). This allowed to assess the variability of tensile behavior in the plate.

The tensile tests for all specimens were performed on a universal servo-hydraulic testing machine with a capacity of 1,000 kN, according to SIA 2052 [24]. The Digital Image Correlation (DIC) technique and three different series of sensors were adopted to measure the deformation and crack opening of the UHPFRC, as shown in Fig. 4. Further details about the developed DTT in this study can be found in ref. [3].

2.3 Four-point bending test (4PBT)

In total six small plate specimens with dimension of 500 mm × 100 mm × 3 mm were cast individually in molds. The 4PBT for all specimens was performed on a universal servo-hydraulic testing machine with a capacity of 200 kN, according to SIA 2052 [10,24]. The total span of the four-point bending test set up was 420 mm (See Fig. 5), and the supports allowed free displacement of the specimen along its longitudinal axis. Two transducers placed on a measuring frame on each side of the specimen measured the net deflection in the center of the span. The measurements were taken at a frequency of 5 Hz during the test.
2.4 Fabrication and curing

The chosen UHPFRC is an industrial premix containing 3.8\% by volume of straight steel fibers with length of 13 mm and diameter of 0.175 mm, and its water/cement ratio is 0.15. The UHPFRC was mixed to obtain a batch of 180 liters. The large square plate and circular slab-like specimens were cast in one step: the fresh UHPFRC mixture was
poured in the center of the formworks, and let flow without any pulling or vibration. Regarding the small plate specimens for 4PBT, the fresh mixture was poured from one side and let flow. Once the casting was completed, a plastic sheet was pulled over the specimens to allow for auto-curing of the material. The specimens were demolded after 24 hours, then kept under moist curing conditions (20°C, 100% humidity) for the following seven days; and subsequently, stored inside the laboratory until testing. The test age was more than 60 days, given that more than 90% of the UHPFRC material properties is attained after 60 days [4,25].

3. Test Results

3.1 Uniaxial tensile response from DTT

In Figure 6, the DTT results of five specimens are presented in terms of stress-strain (σ-ε) curves, in which the thick black line represents the average response. The stress is defined as the measured force divided by the constant cross-sectional area of dumbbell specimen, while the strain is based on the average value measured from two short LVDTs with measuring length of 160 mm. The main characteristic tensile parameters for each specimen are summarized in Table 1, including elastic modulus $E_{Ut}$, elastic limit point (stress $f_{Ue}$ and corresponding strain $ε_{Ue}$), and ultimate point ($f_{Uu}$ and $ε_{Uu}$). Here, the end of the linear relationship in σ-ε curve is regarded as elastic limit point, and the beginning of the tensile softening response is defined as ultimate point. The average curve is obtained through averaging 5 normalized curves, where the stress and strain are divided by the corresponding values at peak point ($f_{Uu}$ and $ε_{Uu}$), respectively. As shown in Fig. 6 and Table 1, a considerable variation of tensile response, strain-hardening behavior in particular, is observed. This is attributed to the variability of fiber distribution in different specimens depending on the distance from the pouring point [3]. In this case, due to the high fluidity and workability of the UHPFRC material, the fresh mixture flowed freely from the center to the border in radial direction. The flow exerted forces on the fibers, pushing fibers to align more perpendicularly to the flow direction, as illustrated in Fig. 7. Additionally, based on a previous study [3], random fiber distribution and orientation can be assumed for specimen T2–T4 around the pouring point.

Based on DIC analysis using VIC-3D, the whole microcracking and fracture process of each specimen is captured effectively during the loading process. The initiation and propagation of fine micro-cracks in the strain-hardening domain, in particular, can be detected visually in DIC full-field strain maps. The representative fracture process shown by T3 is illustrated in Fig. 8. Generally, point A (elastic limit) refers to the start of strain-hardening response in UHPFRC, symbolized by the activation of first fine micro-cracks; while point B (ultimate limit) stands for the end of this response, characterized by the formation of one single localized fictitious crack by grouping of several fine micro-cracks. Afterwards (beyond point B), the fictitious crack shows significant stress transfer through the fibers bridging the two crack sides and develops with increasing crack opening until no more stress is transferred when a crack opening of half the fiber length is reached, i.e., in present case, 6.5 mm (= $L_f/2$). It is important to note that the microcracks are mostly concentrated in two local zones with random distribution of several micro-cracks over a large extent (> $L_f$).

---

![Fig. 6 – Tensile response from DTT](image-url)
Table 1 – Tensile parameters from DTT

<table>
<thead>
<tr>
<th>N°</th>
<th>$E_u$ [GPa]</th>
<th>$f_{Ute}$ [MPa]</th>
<th>$f_{Utut}$ [MPa]</th>
<th>$f_{Utut}/f_{Ute}$</th>
<th>$\varepsilon_{Ute}$ [%]</th>
<th>$\varepsilon_{Utut}$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>45.60</td>
<td>9.70</td>
<td>12.23</td>
<td>1.26</td>
<td>0.22</td>
<td>3.48</td>
</tr>
<tr>
<td>T2</td>
<td>49.09</td>
<td>9.41</td>
<td>11.21</td>
<td>1.19</td>
<td>0.20</td>
<td>2.33</td>
</tr>
<tr>
<td>T3</td>
<td>47.39</td>
<td>7.74</td>
<td>9.62</td>
<td>1.24</td>
<td>0.17</td>
<td>2.68</td>
</tr>
<tr>
<td>T4</td>
<td>49.64</td>
<td>7.25</td>
<td>9.48</td>
<td>1.31</td>
<td>0.17</td>
<td>2.15</td>
</tr>
<tr>
<td>T5</td>
<td>48.38</td>
<td>7.20</td>
<td>9.10</td>
<td>1.26</td>
<td>0.16</td>
<td>0.68</td>
</tr>
<tr>
<td>Average</td>
<td>48.02</td>
<td>8.26</td>
<td>10.33</td>
<td>1.25</td>
<td>0.18</td>
<td>2.26</td>
</tr>
<tr>
<td>Std. dev.</td>
<td>1.59</td>
<td>1.21</td>
<td>1.33</td>
<td>0.04</td>
<td>0.03</td>
<td>1.02</td>
</tr>
<tr>
<td>COV</td>
<td>0.03</td>
<td>0.15</td>
<td>0.13</td>
<td>0.03</td>
<td>0.14</td>
<td>0.45</td>
</tr>
</tbody>
</table>

Fig. 7 – Final crack positions of dumbbell specimens and schematic view of fiber distribution in the plate

Fig. 8 – Representative microcracking and fracture process of UHPFRC specimen under DTT
3.2 Uniaxial tensile response based on inverse analysis from 4PBT

Figure 9 presents the bending behavior from 4PBT in terms of force-deflection curves, in which the thick black curve is the average response. All curves agree well with each other, showing comparable bending behavior. This results from the similar fiber distribution and orientation in the small plate specimens, given that they were fabricated individually in moulds following the same casting procedure.

The uniaxial tensile response of UHPFRC is evaluated indirectly by means of inverse analysis of 4PBT results using non-linear Finite Element Analysis (FEA). A 2D FE model was built using the non-linear FE analysis software DIANA (smear crack model), targeting at simulating the bending behavior in terms of force-deflection response and cracking pattern of small plate specimens under 4PBT. The best results of FE model fitting with the average experimental curve is shown in Fig. 9, where a close fit is achieved, as indicated by the thick red curve. The corresponding uniaxial tensile parameters are summarized in Fig. 13 and Table 2.

3.3 Biaxial tensile response based on inverse analysis from ring-on-ring test

The biaxial flexural responses of four UHPFRC circular slabs from ring-on-ring tests are presented in terms of force-deflection curve \((F - \delta)\) of the center point, as shown in Fig. 10. The recorded force value was adjusted considering a geometry factor that accounts for the precise thickness of each slab. The deflection was measured by DIC on the bottom surface, excluding the deformation of the rubber pad measured from three LVDTs on the top surface. It is obvious that all slabs show a consistent flexural response with little scatter. Up to a force value of about 40kN, the flexural behavior of the UHPFRC slabs in terms of \(F-\delta\) curve is almost linear, and the end of this linearity is herein defined as elastic limit (point A). Afterwards, a quasi-linear response (II, A-B) with a slight decrease of stiffness is noticed, in which the formation and propagation of multiple microcracks are expected. And sequentially, significant deflection hardening behaviour is identified until the peak point (C) is reached. Afterwards, the slabs exhibit important ductility with high residual resistance in the softening phase (IV C-D). Further details about the test results are described in [21].

Furthermore, the representative microcracking and fracture process from S1-3, as observed by DIC on the visualized central portion \((400\,\text{mm} \times 400\,\text{mm})\), is shown in Fig. 11. The selected DIC images in Fig. 11 represent the crack patterns in different characteristic phases following the \(F-\delta\) curve, and the white dash circle marks the position of the force transmitting ring.

Accordingly, several phenomena characterizing microcracking process of UHPFRC slabs are identified. Once the elastic limit (A) is reached, the first microcracks with random distribution initiate within the force transmitting ring area, where the tensile stress is uniform and maximal in all directions. Afterwards, these microcracks start to propagate irregularly, and more new microcracks are generated until point B is reached. In this phase II, the microcrack pattern changes continuously with increasing deflection and has a complex distribution, microcracks initiate randomly with irregular propagation paths; some of them produce multiple branches, and some
cross each other or combine together during propagation. At point B, several fictitious cracks ($w \geq 0.05 \text{ mm}$) are detected by DIC, and afterwards, more microcracking appears with increasing deflection. Fine fictitious cracks initiate from some of previous microcracks in phase II, and largely concentrate within the force transmitting ring area. Some fine fictitious cracks are localized at peak force (point C), and then propagate radially from the center to the edge with increasing crack opening in the softening phase (C-D). No additional cracks form beyond point C.

Similarly, in order to determine the biaxial tensile response of UHPFRC, the inverse analysis of the ring-on-ring test results was conducted by means of a 3D FE analysis using DIANA software. Considering the random fiber distribution in the slab, the full scale of the slab element was modeled, and the smeared crack concept was adopted. The boundary conditions in simulation of ring-on-ring test have a strong influence on the stability of the numerical procedure and on the fracture pattern. During testing, the specimens were not prevented from sliding and lifting from the supporting ring along fracture growth. Therefore, the two adjacent ¼ points of the support ring were only constrained in tangential directions, and the top surface central point was constrained in both X and Y directions. Additionally, the interface elements were built between the specimen and the rubber pads, avoiding tensile reaction force from the supporting ring. The base and verification of FE model are described in ref. [21]. The modeling results with best fitting of $F-\delta$ curve with respect to the average response is illustrated in Fig. 10 (red line), where a close fit is achieved. The results from the inverse analysis are summarized in Fig. 13 and Table 2.

4. Discussion

Finally, all the tensile responses under uniaxial or biaxial stress condition based on different methods, are summarized in Fig. 13 and Table 2. Regarding DTT, the average response from specimen T2–T4 is used for further comparison, since their locations in the large plate correspond to the area under biaxial stress condition in ring-on-ring test. The uniaxial tensile response from 4PBT cannot be applied directly, due to favorable fiber alignment in the small plates along the loading direction. This phenomenon is attributed to the small geometry of the mould and specific casting process as described in section 2.4. Thus, fiber distribution and orientation effect should be considered for better comparison.

Based on a previous study on an UHPFRC layer with thickness of 50 mm [12], the average fiber orientation factor ($\mu_0$) was identified in the range of 0.53–0.60. In this study, the mean value is applied, namely $\mu_0 = 0.57$, for the DTT specimens, given that random fiber distribution may be assumed in the central part of the large plate. For a UHPFRC layer or small specimen with thickness of 30 mm, $\mu_0$ was determined in the range of 0.61–0.70 in ref. [12]. Thus, the upper limit ($\mu_0 = 0.70$) is chosen for 4PBT specimens, respecting preferential fiber alignment in small plates. The corresponding efficiency factor ($\mu_1$) is obtained for both cases (0.94 and 0.96, respectively) based on Fig. 10. Accordingly, the uniaxial tensile response from 4PBT is modified to be representative for the large plate in the case of random fiber distribution, the results are also summarized in Fig. 13 and Table 2.

As observed in Fig. 13 and Table 2, in general, the tensile performance in terms of strength from both uniaxial and biaxial stress states are quantitatively similar. This phenomenon can be attributed to the fact that all the test methods provide the specimens with a certain area of uniform stress, allowing for initiation of microcracks and localization of fictitious cracks at local weaker zones with respect to the main stress direction. Thus, the tensile performance largely depends on the distribution and size of local weaker zones. In the case of random fiber distribution, the local weaker zones can be assumed to distribute randomly without any considerable preference in all directions.

Additionally, it should be noted that a significant increase in hardening strain $\varepsilon_{U_h}$ is found under biaxial stress state. This effect can be due to the fact that many more fibers in different directions contributed to the bridging and debonding effects under bi-axial stress state, offering considerably higher ductility and toughness including larger deformation, compared with the results from specimens with uniaxial stress state, where fibers perpendicular to the loading direction have no contribution. This difference can also be explained by the different cracking patterns under different stress states, as illustrated in Figs. 8 and 11. The circular slab subjected to biaxial stress state shows a large amount of microcracks distributed densely and randomly on the tensile surface in strain-hardening domain (phase II, AB), and most microcracks developed along irregular paths are interlocked. Sequentially, several fictitious cracks appeared. In the case of the DTT specimens under uniaxial stress state, the microcracks concentrated locally at weaker zones and propagated from one edge to the other following a relatively linear path, and only one or two fictitious cracks localized finally.
Fig. 10 – Biaxial flexural response from the ring-on-ring tests

Fig. 11 – Representative microcracking and fracture process of UHPFRC slab of a ring-on-ring test specimen.

Table 2 – Tensile parameters from the three different test methods

<table>
<thead>
<tr>
<th>Test Method</th>
<th>$E_U$ [GPa]</th>
<th>$f_{Ute}$ [MPa]</th>
<th>$f_{Utu}$ [MPa]</th>
<th>$f_{Utu}/f_{Ute}$</th>
<th>$\varepsilon_{Ute}$ [%]</th>
<th>$\varepsilon_{Utu}$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>DTT</td>
<td>49</td>
<td>8.13</td>
<td>10.10</td>
<td>1.24</td>
<td>0.18</td>
<td>2.39</td>
</tr>
<tr>
<td>4PBT</td>
<td>51</td>
<td>10.00</td>
<td>14.00</td>
<td>1.40</td>
<td>0.20</td>
<td>3.92</td>
</tr>
<tr>
<td>4PBT (modified)</td>
<td>51</td>
<td>8.00</td>
<td>11.20</td>
<td>1.40</td>
<td>0.16</td>
<td>3.14</td>
</tr>
<tr>
<td>Ring-on-ring test</td>
<td>50</td>
<td>9.60</td>
<td>11.35</td>
<td>1.18</td>
<td>0.19</td>
<td>5.54</td>
</tr>
</tbody>
</table>
5. Conclusions

This study investigated the tensile behavior of UHPFRC under uniaxial and biaxial stress conditions by means of DTT, inverse analysis based on 4PBT, and ring-on-ring test results using FEM. The results suggest that the tensile response of UHPFRC is not an intrinsic property and depends on several factors, including the specimen geometry, flow regime of fresh mixture during casting, and the stress condition imposed on the specimen.

In the case of random fiber distribution, there is no significant difference of tensile performance in terms of strength between uniaxial and biaxial stress conditions. Furthermore, since more fibers are activated, the multiple microcracking behavior is more
pronounced and complex under biaxial stress condition, resulting in a significantly higher strain hardening deformation $\varepsilon_{\text{thm}}$ compared with uniaxial stress condition. Consequently, when the fibers are distributed randomly, it is conservative to use uniaxial tensile parameters as obtained from DTT or 4PBT, to design UHPFRC structural elements or UHPFRC strengthening layers on concrete substrates subjected to biaxial stress condition.

References


Deformation mechanism of hardened cement paste under high stress and application of flow law

Yuya Sakai*

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Abstract: In this study, a creep test was performed on hardened cement paste (HCP) with stepwise load increase at different confining pressures and saturation degrees. The strain rate–stress relationship, obtained under a stress of >75% of the maximum strength and plotted on a log–log chart, showed a slope of six. From previous studies on crystalline materials, such as rock, metal, and ice, it can be inferred that this slope indicates a deformation governed by dislocation creep. If dislocation creep occurs in HCP, the deformation may be governed by crystalline hydrates other than calcium silicate hydrate (C-S-H) because dislocation creep is generally not defined for gel materials. Further study and careful discussion are required because a slope of six is a necessary condition for the dislocation creep. The activation volume was evaluated, and the flow law was applied to calculate the strain rate of HCP. The obtained activation volume gives a better fit for the measured results than the previously reported values.

Keywords: cement paste, triaxial test, flow law, dislocation creep.

1. Introduction

Concrete is one of the most important construction materials in the world and has been used for a long time. However, the mechanism of concrete deformation has not yet been fully understood. For example, creep deformation was reported more than 100 years ago [1], but its mechanism is still a subject of discussion. The main mechanisms of creep deformation proposed so far are based on microcrack formation [2], water seepage from hardened cement paste (HCP) [3], occurrence of slip between the globules of calcium silicate hydrate (C-S-H) [4], occurrence of microprestress [5], etc. Gaining a clear understanding of the concrete deformation mechanism is important for the safe and rational design and maintenance of concrete structures. With regard to creep deformation, it is known that the stress higher than the threshold stress causes creep fracture, and this threshold stress is called the sustained load strength [6–8]. The sustained load strength usually ranges from 70% to 80% of the maximum strength in normal concrete [6,9,10], although higher fractions (e.g. > 85%) have been reported for high-strength concrete [11,12]. Hsu et al. [13] studied the development of cracks in concrete and reported that matrix cracks formed continuous crack patterns under a stress of more than 70% of the maximum strength. Because this fraction (percentage of the sustained load strength in relation to the maximum strength) agrees with the ratio of the applied stress to the maximum strength, crack development may be related to the sustained load strength; however, the origin of the sustained load strength has not yet been clearly explained. The ratio between the sustained load strength and maximum strength for various types of concrete is almost consistent, which indicates that they have a common mechanism; gaining an understanding of the deformation mechanism under high stress (stress higher than the sustained load strength) may lead to a better understanding of the failure mechanism of concrete.

In this study, the deformation mechanism of HCP was studied by performing triaxial tests. The results showed that a shear plane was not formed in HCP that was subjected to a certain confining pressure, even though the HCP deformed upon the application of up to 10% strain [14]. Based on this result, it was inferred that the HCP shows plastic flow when subjected to a certain confining pressure. The plastic flow of HCP was clearly observed during the compaction of crushed HCP [15]. The mechanism of the flow was studied by performing a creep test with a stepwise increase in the load. Furthermore, the strain rate was calculated assuming that the HCP reached a static state within 20 min after the load increase [14].

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However, this assumption was not appropriate because HCP requires a longer time to reach a static state under sustained load [6]. The obtained results were not consistent and not easy to analyze or interpret. Moreover, a model to describe the deformation of HCP was not proposed. The strain rate due to plastic deformation is described by the flow law that is often applied to inorganic materials, such as rocks, ceramics and metals [16,17]. However, to the best of our knowledge, the application of the flow law to HCP has not been studied yet.

Therefore, in this study, another analysis method was applied to the results of the stepwise creep test, additional experiments were performed to apply the flow law, and a quantitative discussion was included on the deformation mechanism of HCP. First, a loading test was performed at a constant strain rate to obtain the maximum strength. The samples were cut after the test and the cross-sectional surfaces were observed. Subsequently, a creep test was performed with a stepwise increase in the load corresponding to 30–95% of the maximum stress. The deformation mechanism of the HCP was discussed based on the obtained strain rate–stress relationship. The activation volume was evaluated, and the flow law was applied to describe the deformation of the HCP under high stress.

2. Methodology

2.1 Sample preparation

In this research, cement paste (water-to-cement ratio = 0.4) made from ordinary Portland cement was used. The properties of the cement are presented in Tables 1 and 2. The mixing procedure was based on JIS R 5201. The paste was first mixed for 60 seconds in a mixer operating at a low speed (orbital rotation: 62 ± 5 rpm, planetary rotation: 140 ± 5 rpm). The mixer was stopped for 30–62 ± 5 rpm, planetary rotation: 140 ± 5 rpm). The mixed paste was cast in a plastic mould (250 × 150 × 100 mm) and sealed. It was demoulded 24 hours after casting and then kept under water for two months. The temperature of the room and water was 24 °C. After curing, cylinders of φ10 mm were cored from the HCP mass. Only the part deeper than 2 cm from the surface was used. Both ends of the cylinder were ground to achieve flat and parallel surfaces. The prepared cylinders (φ10 × 24 mm) were immersed into acetone for 24 hours to stop the hydration reaction and reduce capillary suction in the subsequent drying period. After immersion, the cylinders were dried in a desiccator at 24 °C and 20% RH until the weight change over 24 hours because of moisture loss was less than 0.1% of the specimen weight. Saturated samples were prepared by immersing the cylinders into tap water after drying at 24 °C and 20% RH until the weight change over 24 hours was less than 1% of the specimen weight. The specimen names are composed of the alphabets D or W followed by numbers (e.g. D0, W50); D and W indicate dry and saturated samples, respectively, and the number indicates the confining pressure $P_c$. Assuming that the saturation degrees of the saturated and oven-dried (at 105 °C) samples were unity and 0, respectively, the calculated saturation degree of a sample dried at 24 °C and 20% RH was 0.31. Considering that HCP was immersed in acetone, this saturation degree (0.31) was a mixture of water and acetone. The testing age varied in the range of 5–6 months. The carbonation depth of the sample, which was kept in the desiccator (24 °C and 20% RH) after all tests, was measured using a 1% solution of phenolphthalein in ethyl alcohol. The sample was split horizontally at the middle using a chisel, and the solution applied on the fracture surface indicated that the sample had a carbonation depth of 0.5 mm. The porosity of the sample, which was calculated using the following equation, was found to be 0.39.

$$\phi = 1 - (W_{oven} - W_{water})/(W_{sat} - W_{water}) \quad (1)$$

where $\phi$ is the porosity, $W_{oven}$ is the oven-dried weight, $W_{water}$ is the saturated weight under water, and $W_{sat}$ is the saturated weight in the air.

<table>
<thead>
<tr>
<th>Chemical composition of cement</th>
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<td>Chemical property (%)</td>
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<th>Physical properties of cement</th>
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<tr>
<td>Density (g/cm$^3$)</td>
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<td>3.12</td>
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2.2 Triaxial test

2.2.1 Test apparatus and sample assembly

The Paterson-type [18] triaxial test apparatus was used with argon gas as the pressure medium to generate confining pressure on the sample. The schematic view of the apparatus is shown in Fig. 1. A sample was placed between alumina and zirconia pistons enclosed in a heat-shrinkable (polyolefin) tube that sealed the sample from the confining medium. The end face of the HCP sample was exposed to the laboratory atmosphere through small center holes in the spacers and pistons to avoid pore pressure being generated during experiments. To achieve shrinkage, the tube was heated from the outside using a heat gun. To check the temperature of the HCP specimen during the heat treatment on the tube, a 2-mm-diameter hole was created on the HCP sample and the temperature in the hole was measured using a needle probe thermometer [19]. The distance from the sample surface to the hole was 2 mm. A heat-shrinkable tube was placed over the HCP sample and heated until the tube touched the sample completely. The maximum temperature reached was 46 °C; therefore, the heat might have some effect near the surface of the sample during this heating process. A saturated sample was wrapped with a plastic film before being covered with a heat-shrinkable tube. Steel anvils were then attached to the top and bottom of the zirconia pistons and fixed with steel wires over the heat-shrinkable tube. Figure 2 shows a photograph of the sample assembly. The prepared sample assembly was inserted into the pressure vessel of the test apparatus. Load was applied by moving the lower piston upward. The stress was calculated by dividing the load measured using the internal load cell by the cross-sectional area of the sample. The apparatus was designed such that the confining pressure did not affect the stress measured by the internal load cell; therefore, the calculated stress corresponded to the differential stress. The axial strain was calculated by dividing the displacement measured using the transducer located below the sample assembly by the initial length of the sample. The load, displacement, and confining pressure were measured at intervals of 1 second. All the tests were performed at room temperature (24 °C).

2.2.2 Constant strain rate loading test

Triaxial tests with constant strain rate loading were performed at $1.4 \times 10^{-4}$ s$^{-1}$. Two dry samples were tested at each value of $P_c$; one of the two samples was loaded until the strain reached 10%, and the
other one was loaded until the stress started decreasing after reaching the maximum stress ($\sigma_{\text{max}}$). Samples tested at $P_c = 0$ MPa were loaded until the stress decrease stagnated.

After the triaxial tests with constant strain rate loading, the sample covered with a heat-shrinkable tube was cut out from the assembly and impregnated into a two-component epoxy resin (room-temperature curing). The sample was then cut parallel to the long axis when the epoxy resin hardened. The cutting surface was polished using alumina powder (average diameter = 55 and 15 μm) and observed by the naked eye.

2.2.3 Stepwise creep test

A stepwise creep test was performed to obtain strain rate–stress relationship. In this test, the applied stress was increased stepwise from 30% to 95% of $\sigma_{\text{max}}$. The stress was increased every 30–60 minutes. The strain rate–stress relationship is known to show more internal consistency in the stepwise creep test than in the conventional creep test [20, 21]. In a previous study, the strain rate was calculated when the slope of the strain–time curve was regarded to be in a static state, but the slope actually kept decreasing and did not reach a static state [14]. Thus, as in the study by Zhang and Spiers [22], who studied the compaction mechanism of calcite powder, the strain rates were calculated from the slope of the strain increment-time relationship at certain strain values. The strain rate of D0 was calculated using data obtained 1000–2000 seconds after each value of stress was attained, and that of W0 was obtained by fitting the exponential approximate function for the entire data set because the strain in the samples without $P_c$ was small. The strain rate at steady-state creep is described by the following flow law [23]:

$$\dot{\varepsilon} = A\sigma^n d^{-m} \exp\left\{-(Q + PV)/RT\right\}$$  \hspace{1cm} (2)

where $\dot{\varepsilon}$ is the strain rate (s$^{-1}$), $A$ is a constant (m$^n$/Pa$^n$), $\sigma$ is the differential stress (Pa), $n$ is the stress exponent, $d$ is the grain size (m), $m$ is the grain size exponent, $Q$ is the activation energy (J/mol), $P$ is the pressure (Pa), $V$ is the activation volume (m$^3$/mol), $R$ is the gas constant (= 8.3 J/K/mol), and $T$ is the absolute temperature (K). The slope of the strain rate-stress relationship corresponds to $n$.

2.3 Hydrostatic pressure test

The value of the activation volume is necessary for applying the flow law to the deformation of HCP. The equation to calculate the activation volume is derived from Eq. (2) by taking the natural logarithm and differentiating partially with respect to $P$ (assuming $\sigma$ to be a constant) as follows:

$$\ln \dot{\varepsilon} = \ln A + \ln \sigma - m \ln d - Q/RT - PV/RT$$  \hspace{1cm} (3)

$$\partial \ln \dot{\varepsilon}/\partial P = -V/RT \quad (\sigma: \text{constant})$$  \hspace{1cm} (4)

$$V = -RT \partial \ln \dot{\varepsilon}/\partial P$$  \hspace{1cm} (5)

To calculate $V$ using Eq. (5), it is necessary to determine $\dot{\varepsilon}$ for different $P$ ($P_c$) values with $\sigma$ being constant. However, as shown later in Section 3.2, the strain rate–stress relationship follows the flow law only when $\sigma$ is more than 75% of $\sigma_{\text{max}}$. Because $\sigma_{\text{max}}$ of dry samples increases with an increase in $P_c$, the constant $\sigma$ can be less than 75% of $\sigma_{\text{max}}$ with increasing $P_c$ and $V$ would be calculated in the region where the data do not follow the flow law. Therefore, $V$ was calculated by measuring the strain rate of a saturated sample for different $P_c$ values with a constant $\sigma$ (92 MPa, 80% of $\sigma_{\text{max}}$ in W20) because $\sigma_{\text{max}}$ of the saturated sample hardly depends on $P_c$, as shown later in Section 3.1. For the saturated sample, $P_c$ was first set to 20 MPa, $\sigma$ corresponding to 80% of $\sigma_{\text{max}}$ was applied, and $P_c$ was then increased by 10 MPa up to 50 MPa to obtain the $\ln \dot{\varepsilon} - P_c$ relationship.

3. Results

3.1 Constant strain rate loading test

The differential stress–strain curves of the dry samples are shown in Fig. 3(a). The results for the samples loaded until $\varepsilon = 10\%$ and those loaded until the stress started decreasing are represented by broken lines and solid lines, respectively. The two samples with the same $P_c$ showed similar results. D0 showed a sudden stress decrease after $\sigma_{\text{max}}$, $\sigma_{\text{max}}$ increased with an increase in $P_c$. The stress kept increasing until $\varepsilon = 10\%$ for $P_c = 100$ MPa, but the stiffness decreased in the low-strain region. Figure 3(b) shows the results for the saturated samples. Compared to the dry samples tested at the same $P_c$, $\sigma_{\text{max}}$ for the saturated samples decreased. W20 and W50 showed similar curves.

Figure 4 shows the cut and polished surfaces of the samples after the triaxial tests. Except for D0, the left image shows the sample loaded up to 10% strain and the right image shows the sample loaded until the stress started decreasing. D30 shows only the sample loaded up to 10% strain. The black lines in the samples are the epoxy resin that penetrated through the cracks. Vertical cracks developed in D0, and shear planes were formed in D10, D30, and D50. A shear plane was not formed in D100.

3.2 Stepwise creep test

The obtained strain increment–time curves are shown in Fig. 5. The values in the legends indicate stress (the values in parentheses indicates the ratio of
the applied stress to $\sigma_{\text{max}}$). The values in parentheses at the upper right corner are the strain values at which $\dot{\varepsilon}$ was calculated. Figure 6 shows the relationship between $\dot{\varepsilon}$ and $\sigma$, with the lines indicating slopes of three and six. The data for stress values less than 0.75 $\times \sigma_{\text{max}}$ are shown as plots represented by short lines, and the data for stress values equal to or greater than 0.75 $\times \sigma_{\text{max}}$ are shown as plots represented by circles. The plots represented by the short lines are scattered, whereas the plots represented by the circles are distributed linearly, except for D0, and have slopes close to six.

3.3 Hydrostatic pressure test

Figure 7 shows the strain–time relationship and $\ln \dot{\varepsilon} - P_c$ relationship of a saturated sample in the hydrostatic pressure test. According to Eq. (5), the slope of the plots in Fig. 7(b), $-3 \times 10^{-8}$, multiplied by $-RT$ is equal to $V$, and the calculated $V$ for the saturated samples was $7.4 \times 10^{-5} \text{ m}^3/\text{mol}$.

4. Discussion

4.1 Deformation mechanism of HCP under high stress

The dry samples showed stress decrements, and vertical cracks or shear planes were formed when $P_c$ was 0–50 MPa. When $P_c$ was 100 MPa, the stress increased up to 10% strain and no shear plane was formed. In rock mechanics, fractures at lower strains with vertical cracks or shear planes are considered brittle, whereas those at higher strains without shear planes are considered ductile [18]. Following this classification, the HCP deformation changed from brittle to ductile with an increase in $P_c$. These results are consistent with those obtained in a previous study [14]. The horizontal crack in the sample shown on the right side of Fig. 4(d) might have been formed during unloading.

As seen in Fig. 3(a), the stress did not decrease when $P_c$ was 100 MPa; a similar tendency has been reported for concrete [24]. However, HCP showed a decrease in stress in the low-strain region, whereas concrete did not. A possible reason for this difference is that the aggregate and concrete were in contact with each other. Consequently, the aggregate generated resistance that prevented a decrease in stress. In porous rocks, stress decreases due to microscopic damage has been reported [25,26], and a similar phenomenon might have occurred in HCP. In Figure 6, the strain rate–stress relationship at stress values greater than 0.75 $\times \sigma_{\text{max}}$ is seen to be distributed linearly and the slope is approximately six. This slope corresponds to $n$ in Eq. (2).

![Fig. 3](image1.png)

Fig. 3 – Differential stress–axial strain relationship with constant strain rate loading

![Fig. 4](image2.png)

Fig. 4 – Cross section of dry specimens after constant strain rate loading test
The creep mechanisms of crystalline materials are classified according to the value of $n$: $n = 1$ is considered to represent diffusion creep (creep deformation governed by atomic diffusion), and $n > 1$ is considered to represent dislocation creep (creep deformation governed by dislocation movement) [26,27]. Bürgmann and Dresen [28] classified $n = 3–6$ as creep governed by dislocation climb. Various crystalline materials such as metal [29] and ice [30] have similar equations and classifications. Thus, the slope of six seen in Fig. 6 indicates that the deformation of HCP for stress values larger than $0.75 \times \sigma_{\text{max}}$ is governed by dislocation climb. The same slopes for the dry and saturated samples indicate that the deformation mechanism does not change according to the saturation degree. Concrete generally fails when it is subjected to a stress larger than a sustained load strength that is approximately 75% of the maximum strength [6,9,10]. This fraction corresponds to the threshold stress above which data are distributed with the slope of six in Fig. 6. This correspondence indicates that the creep failure in concrete may be attributed to deformation due to dislocation creep. Many materials show plastic deformation, which is sometimes followed by brittle fracture. In addition, the interaction between cracks and dislocation plays an important role in plastic deformation [31]. Therefore, the propagation of cracks in HCP at stress values of more than 70% of the maximum strength may be affected by dislocations. However, it should be noted that the slope of six is a necessary, but not sufficient, condition for dislocation creep.

Fig. 5 – Strain increment–time relationship in stepwise creep test
4.2 Application of flow law

The flow law (Eq. (2)) was applied to the obtained results. Figure 6 shows the results for $n = 6$ and $m = 0$ for the dislocation creep. The value of $Q$ was set as 35,000 J/mol [39,32] for the dry sample and 17,500 J/mol for the saturated sample [31–33] based on the results for rocks. The obtained $V$ for the saturated sample was $7.4 \times 10^{-5} \text{m}^3/$mol. Sammis et al. [36] reported the $Q$ and $V$ values for various materials; the values obtained in this study were close to those of the ionic crystals. The value of $V$ for the dry sample was also assumed to be $7.4 \times 10^{-5} \text{m}^3/$mol because a similar value was previously reported for dry and wet rocks [33]. The value of $V$ reported by Klug and Wittmann [37] was $1 \times 10^{-20} \text{cm}^3 (6 \times 10^{-3} \text{m}^3$/mol), which was approximately 100 times larger than the one obtained in this study. For fitting the calculation results to the measured data, $A$ was set to $1 \times 10^{-23}$ for the dry samples and $5 \times 10^{-15}$ for the saturated samples. The calculated curves using these parameters are shown in Fig. 8(a) along with the measured data for stress values larger than 0.75 $\sigma_{\text{max}}$. The calculated and measured data are in good agreement. Figure 8(b) shows the calculated curves obtained using the value of $V$ reported by Klug and Wittmann [37]. The value of $A$ was adjusted to obtain the best possible fit for the data ($A = 1 \times 10^{15}$ for dry and saturated samples) but the calculated curves varied significantly depending on $P_c$ and deviated from the measured data. In Equation (2), $V$ expresses the dependency of $\dot{\varepsilon}$ on $P_c$; therefore, the value of $V$ reported by Klug and Wittmann [37] is likely to be too large to describe the deformation of HCP under high stress based on the flow law.

4.3 Contribution of C-S-H to deformation of HCP

The strain rate–stress relationship of HCP in Fig. 6 indicates that the deformation of HCP was governed by dislocation similar to the case of crystalline materials such as rock [27,28], metal [29], and ice [30]. In concrete, dislocation was reported to occur in calcium hydroxide because of the stress caused by drying shrinkage [38]. However, 50–60% of the HCP volume is composed of C-S-H, which is generally regarded as a gel. Klug [39] and Klug and Wittmann [37] discussed the creep deformation of HCP, assuming the deformation of the amorphous solid skeleton. However, the plastic deformation mechanism of an amorphous material, such as metallic glass, is explained by the shear band formation or local atomic jump [41,42]. In both these mechanisms, the strain rate is a hyperbolic function of stress, and the value of $n$ changes from 1 to a very large number. The consistent slope of six obtained in this study indicates that the deformation mechanism of HCP is closer to that of a crystalline material than an amorphous material. The time-dependent response of C-S-H has been studied using various approaches, and recently, atomistic simulation has appeared as a powerful tool for its investigation. Morshedifard et al. [40] carried out a molecular dynamics simulation to study the time-dependent response of C-S-H, and they reported a behavior often seen in glassy systems. In addition, these authors reported that the amount of interlayer water changes the time-dependent response of C-S-H. If the response of C-S-H under high stress is similar to metal glass and if dislocation creep occurs in HCP, crystalline hydrates such as calcium hydrates or ettringite may govern the deformation under high stress. As these crystalline hydrates are not dominant in volume, they likely form a skeleton to resist the applied load, and the deformation of this structure may be governed by the dislocation creep [43]. The calcium hydroxide crystal is large, and its aspect ratio is high. Generally, to form a skeleton, the element volume fraction needs to be more than about 16% [44,45]. However, when the aspect ratio is high, the required volume fraction decreases significantly [46,47], and a skeleton is formed more easily. The volume fraction of the hydrates other than C-S-H is around 40%, which is sufficiently large for the hydrates to form a structure. Further study is necessary to conclude the deformation mechanism.

5. Conclusion

In this study, a stepwise creep test was performed to understand the deformation mechanism of hardened cement paste. Subsequently, the flow law was applied to the obtained test results. The following conclusions were derived:

1. The maximum strength of the saturated samples was lower than that of the dry samples under any given confining pressure.
2. The strain rate and differential stress relationship obtained in the creep test with a stepwise increase in the load showed a slope of six on a log–log chart. This slope indicates that the deformation is governed by dislocation creep.
3. Dislocation creep might have possibly caused the fracture of concrete when the stress was larger than the sustained load strength.
4. The calculated strain rate based on the flow law showed that the obtained activation volume was reasonable whereas the one reported by a previous study was too large.
Because 50–60% of the cement paste volume is composed of the gel hydrate C-S-H, dislocation cannot be defined in a gel. Other crystalline hydrates could likely govern the deformation, similar to the deformation governed by dislocation creep.

The results of this study indicate that dislocation might play an important role in the deformation of hardened cement pastes under high stress. If this is true, dislocation may govern various failure patterns of concrete, such as fatigue. Further study and careful discussion in this regard are necessary because the slope of six in the strain rate–stress relationship is a necessary but not a sufficient condition for dislocation creep.
Acknowledgements
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References


Technical Paper

Strength, shrinkage and creep of concrete including CO$_2$ treated recycled coarse aggregate

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Abstract: Strength development and long-term behaviour of recycled aggregate concrete (RAC) were investigated. Recycled coarse aggregate (RA) replaced natural coarse aggregate (NA) by 30%, 50%, and 100% by vol. in this study while natural fine aggregate was always used. For the investigation of strength development, 3, 7, 28, 56, 90, and 180 day compressive strengths and 28 day split tensile strength and flexural strength have been determined. Shrinkage and creep behaviours of RAC with 0%, 50%, and 100% RA replacing NA have been studied up to 6 months. The compressive strength of RAC was not affected by 30% replacement, but it was reduced by 50% and 100% replacement. Split tensile strength was not affected significantly while flexural strength was reduced with increasing amount of replacement. Shrinkage strain of RAC with 50% RA was similar to that of natural aggregate concrete, but shrinkage increased with 100% replacement. Specific creep of RAC with 100% RA increased by 38% over that of natural aggregate concrete. The strength of concrete with CO$_2$ treated RA was lower than that of concrete with RA without CO$_2$ treatment. Shrinkage of RAC with and without CO$_2$ treatment was similar, while the creep of RAC with CO$_2$ treated RA was smaller than that of RAC including RA without CO$_2$ treatment.

Keywords: recycled coarse aggregate, recycled aggregate concrete, strength, creep, shrinkage, carbonation.

1. Introduction

In South Korea, an economic boom has started in late 1960s about 50 years ago and many building and civil engineering infrastructures have been built starting from the late 1960s. The amount of construction and demolition waste (C&DW) generation is huge and takes about 50% of national waste generation, primarily due to demolition of old structures constructed during the economic boom period. Annual C&DW generation was 68 million tonnes in 2011, where the waste concrete took about 65% followed by waste asphalt concrete (19%), mixed waste (10%), and others [1].

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The effective reutilization of waste concrete typically includes use as road subbase material, secondary product such as bricks and blocks as well as recycled aggregates. Although the environmental impact for the reutilization varies depending on the end product, the most effective way to reutilize waste concrete is in the form of recycled aggregate especially from the view point of resource conservation. It should be noted that the use of structural quality recycled aggregate is not wide spread practice yet: Only 1-2% of waste concrete is used as structural grade recycled aggregate in South Korea for example.

There are many different standards that regulate the quality of recycled aggregate in different countries. For example, in Europe, only recycled coarse aggregate is accepted [2]. In South Korea and Japan, both recycled fine aggregate and recycled coarse aggregate are used [3–6]. Despite complicated modern day production technology of recycled aggregate including multiple-stage crushing, it is not possible to completely remove old mortar adhered to original natural aggregates. The adhered mortar makes the mechanical properties of the recycled aggregate inferior to those of natural aggregate, especially density and water absorption [7].
Two different approaches can be employed to improve the mechanical properties of recycled aggregate: i.e. strengthen the adhered mortar or remove the adhered mortar. One of the strengthening approach of the adhered mortar is accelerated carbonation [8-12].

There have been many studies on the long-term behaviour of recycled aggregate concrete, but very few on the shrinkage and creep behaviour of recycled aggregate concrete using carbonated recycled aggregate [13-20]. This study aims to investigate the following:

1. Improvement of mechanical properties of recycled coarse aggregate (RA) by accelerated carbonation to lower absorption and to increase density;
2. Monitor strength development of recycled aggregate concrete (RAC) with RA replacing NA by 30%, 50%, and 100% by vol. compared to that of natural aggregate concrete (NAC); and
3. Investigate the long-term properties such as shrinkage and creep of RAC with RA replacing NA by 50% and 100% by vol. compared to those of NAC.

2. Materials properties and preparation for long-term test

2.1 Aggregates and accelerated carbonation of recycled coarse aggregate

Crushed natural coarse aggregate (NA) and recycled coarse aggregate (RA) of 25-mm nominal size were used. Crushed natural fine aggregate (FA) was always used. RA was supplied by a local commercial waste concrete treatment company. Figure 1 shows sieve analysis results of NA, RA, and FA [21]. RA did not satisfy the density and absorption requirements by KS F 2573, which is density of 2,500 kg/m³ or greater (O.D.) and water absorption of 3% or smaller. Since RA did not satisfy the requirements in terms of density and water absorption, it was attempted to improve the quality of RA by accelerated carbonation following KS F 2584 [22] as the carbonation of concrete would result in increased strength and reduced permeability [23]. The carbonation of adhered mortar can be expressed by Eq. (1).

\[
\text{Ca(OH)}_2 + \text{CO}_2 = \text{CaCO}_3 + \text{H}_2\text{O}
\]

RA was carbonated in a carbonation chamber for three days (72 hours). The rate of carbonation depends on the moisture content of the adhered mortar and the relative humidity of the ambient medium [23,24]. To achieve the moisture content ideal for carbonation, the following procedure was adopted in this study.

RA was soaked in water for 10 minutes. Dry cloth was then used to clean up surface moisture of the aggregates. In the next step, RA was exposed to room environmental condition for five hours where the temperature was 21°C typ. and R.H. was 40%-45% typ. Moisture content of RA was measured every one hour during the five-hour period. As a result, the moisture content at entry to the carbonation chamber ranged between 63% and 67%.

During the three-day-long accelerated carbonation, the temperature inside the chamber was maintained at 20 ± 2 °C, R.H. was 60 ± 5%, and carbon dioxide (CO₂) concentration was 5 ± 0.2%, respectively, while the pressure inside the chamber was the same as the atmospheric pressure. The carbonated recycled coarse aggregates thus produced are called CRA in this study.

2.2 Adhered mortar amount of RA

RA mechanical properties are dependent on amount of adhered mortar. Pre-soaking in acid was the method adopted in this study to determine adhered mortar amount [27]. After RA was oven dried for 24 hours, RA was soaked in 20% hydrochloric acid (HCL) at 20°C for 24 hours and then soaked in distilled water. Difference in weights before and after soaking was used to determine adhered mortar amount as shown in Eq. (2):

\[
\text{Adhered mortar amount} = \frac{(W_1 - W_2)}{W_1} \times 100, \%
\]

where \(W_1\) is bulk weight of aggregate before soaking (O.D.) and \(W_2\) = bulk weight of aggregate after soaking (O.D.).

Table 1 summarizes the mechanical properties of all aggregates determined in this study in terms of water absorption, density, adhered mortar amount, crushing value, and fineness modulus (F.M.). Figure 2 shows RA with adhered mortar (before soaking) and RA without adhered mortar (after soaking).

2.3 Mix design

Volumetric concrete mix design of 100% natural aggregate concrete (NAC) was for a target strength of 30 MPa. Table 2 shows the mix design and properties of fresh concrete. All aggregates were prepared and mixed in SSD condition. Recycled aggregate concrete (RAC) was produced by substituting 30%, 50%, and 100% of NA with RA by volume. Multiple test cylinders (Φ100 x 200 mm) were made for compressive strength test and split tensile strength test, while prismatic specimens (100 x 100 x 400 mm) were used for flexural strength test.

Figure 1 shows sieve analysis results of NA, RA, and FA for 24 hours, while the pressure inside the chamber was the same as the atmospheric pressure. The carbonated recycled coarse aggregates thus produced are called CRA in this study.

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All cylindrical and prismatic specimens were demolded one day after casting and then cured under water until the test age. Compressive strength was tested at 3, 7, 28, 56, 90, and 180 days. Elastic modulus of concrete and Poisson’s Ratio were also determined at 28 days during compressive strength test while three replicate specimens were tested. Split tensile strength and flexural strength were determined only at 28 days while two replicate specimens were tested.

2.4 Preparation for shrinkage measurement

Shrinkage tests were conducted using a prismatic specimen (100 x 100 x 400 mm) for five different concretes: NAC, RAC-50, RAC-100, CRAC-50, and CRAC-100. One day after casting, beam mold was removed and the specimens were brought to inside an environmental chamber where temperature was set at 20°C and R.H. was set at 60%. A thermocouple was used to measure temperature inside the chamber while a portable hygrometer was used to measure R.H. As the specimens were exposed to air, shrinkage measurement started immediately using an embedded strain gauge (60-mm length) and additional two strain gauges (60-mm length) mounted on two side faces of the beam. Teflon sheet was used at bottom surface of the beam to eliminate friction between the steel base plate and the concrete beam. Shrinkage data were taken using a data logger connected to a computer at every 1 hour. The shrinkage measurement continued for 180 days.

2.5 Preparation for creep test

Creep test started 35 days after casting using a 150 x 300 mm cylinder for five different concretes: NAC, RAC-50, RAC-100, CRAC-50, and CRAC-100. The creep test specimens were cured under water for 35 days after which they were placed in the same environmental chamber for the shrinkage measurement until the end of the creep test which continued for 150 days. Two cylinder specimens were placed in a loading frame typ. on a 50,000-lb- (220-kN) capacity load cell, while the applied sustained load was about 30% of 28-day compressive strength. From a companion cylinder stored right next to the creep test specimen, shrinkage data were also retrieved. The creep measurement was made by means of three strain gauges per cylinder: one embedded strain gauge and two strain gauges mounted on two side faces symmetrically (same as shrinkage measurement). The second data logger connected to a computer was used while the data acquisition rate was one data set at every ten minutes.

<table>
<thead>
<tr>
<th>Aggregate type</th>
<th>Water absorption (%)</th>
<th>Density, SSD (kg/m³)</th>
<th>Crushing value (%)</th>
<th>Adhered mortar amount (%)</th>
<th>F.M.</th>
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</thead>
<tbody>
<tr>
<td>Coarse aggregate</td>
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<td>2.430</td>
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<td>24.2</td>
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<tr>
<td>CRA</td>
<td>3.14</td>
<td>2.490</td>
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<tr>
<td>Fine aggregate (FA)</td>
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<td>2.56</td>
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Table 2 Mix design for 1 m³ concrete and slump and air content of fresh mix

<table>
<thead>
<tr>
<th>Index</th>
<th>W/C</th>
<th>S/A</th>
<th>C (kg)</th>
<th>W (kg)</th>
<th>Sand (kg)</th>
<th>NA (kg)</th>
<th>RA (kg)</th>
<th>CRA (kg)</th>
<th>Ad. (kg)</th>
<th>Slump (mm)</th>
<th>Air content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NAC</td>
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<td>182</td>
<td>806</td>
<td>909</td>
<td>--</td>
<td>--</td>
<td>2.73</td>
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<td>RAC-30</td>
<td></td>
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<td></td>
<td></td>
<td></td>
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<td>248</td>
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<td>--</td>
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<tr>
<td>RAC-100</td>
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<td></td>
<td></td>
<td>--</td>
<td>821</td>
<td>--</td>
<td>--</td>
<td>150</td>
<td>4.3</td>
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<tr>
<td>CRAC-30</td>
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<td></td>
<td></td>
<td></td>
<td>640</td>
<td>--</td>
<td>254</td>
<td>--</td>
<td>155</td>
<td>5.7</td>
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<td>457</td>
<td>--</td>
<td>423</td>
<td>--</td>
<td>155</td>
<td>5.6</td>
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<td>CRAC-100</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>--</td>
<td>--</td>
<td>847</td>
<td>--</td>
<td>150</td>
<td>5.6</td>
</tr>
</tbody>
</table>

NOTE: W/C was 0.5 by wt.; S/A was 0.48 by vol.; super plasticizer was used at 2.73 kg; RAC-30 recycled aggregate concrete with 30% replacement of NA by RA; CRAC-50 carbonated recycled aggregate concrete with 50% replacement of NA by RA; natural fine aggregate was used for all mixes.
3. **Test Results**

### 3.1 Mechanical properties of NA, RA and CRA

In Table 1, the mechanical properties of NA, RA, and CRA are summarized as well as those of fine aggregates. It is seen that the water absorption of RA (3.84%) is significantly larger than that of NA (0.48%). SSD density of RA (2,430 kg/m³) is 90% of NA (2,690 kg/m³) while the crushing value of RA (21.2%) is larger than that of NA (17.4%). The adhered mortar amount which plays a major role for the high absorption and low density is 24.2% for RA in
Table 1. It is also shown in Table 1 that, after carbonation treatment, the density of RA increases from 2,430 kg/m$^3$ (SSD) to 2,490 kg/m$^3$ (SSD) and the water absorption decreases from 3.84% to 3.14%. Therefore the three-day accelerated carbonation scheme adopted in this study was effective and it can be safely assumed that all or part of the adhered mortar got carbonated which resulted in decreased absorption and increased density.

3.2 Strength, elastic modulus and Poisson’s Ratio of hardened concrete

Table 3 and Figures 4 and 5 show the compressive strength development. Compressive strengths of NAC can be compared with those of RAC-30, RAC-50, and RAC-100 in Table 3 and Figs. 4 and 5. Figure 5(a) shows that the compressive strength is similar for NAC and RAC-30, which indicates that the compressive strength is not affected by 30% replacement of NA with RA. Compressive strengths of RAC-50 and RAC-100 are lower than that of NAC. The results indicate that the strength is not influenced with 30% replacement of NA with RA but it is influenced with increasing replacement ratio of RA by 50% and higher. The strength reduction is not large even with 50% and 100% replacement. For example, at 28 days, the compressive strength of NAC is 34.4 MPa while it is 33.1 MPa (96% of NA) and 34.4 MPa (100%), respectively, for RAC-50 and RAC-100. After 180 days, the compressive strength is 39.4 MPa for NAC while it is 37.5 MPa for RAC-50 (95%), and 36.6 MPa for RAC-100 (93%).

Table 3 and Figure 5(a) also show that the strength development after 28 days is similar between NAC and RAC: i.e. the strength increases slowly and steadily even after 28 days up to 180 days as shown. KS F 2573 currently allows maximum 30% RA replacement [3]. Test results confirm that current 30% limit is valid. It is noted that the strengths of RAC-50 and RAC-100 are not much lower than that of NAC. There is a possibility that the current maximum replacement limit of 30% can be raised in case of good quality RA that meets the requirements of KS F 2573.

Figure 5(b) compares the strength development of NAC and CRAC-30, CRAC-50, and CRAC-100. Again the compressive strength of CRAC-30 with 30% replacement of NA with CRA is similar to that of NAC. However, the compressive strengths of CRAC-50 and CRAC-100 are significantly lower than that of NAC at all test ages. Test results suggest that, although the compressive strength is not affected by 30% replacement of CRA, it is reduced by replacement of CRA by 50% or higher. It must be noted in Table 3 that the compressive strengths of CRAC-50 and CRAC-100 are lower than those of RAC-50 and RAC-100, respectively, at all test ages.

This unexpected results need explanation because, after accelerated carbonation, the mechanical properties of CRA such as density and absorption improved over that of RA. At present authors do not have a clear explanation to this phenomenon. A hypothesis is suggested that, with three-day accelerated carbonation scheme adopted in this study, only the part of adhered mortar gets carbonated, which may result in poor bond with new cement paste at surface of carbonated adhered mortar.

Current test results agree well with that in the existing literature [8-20]. Andal et al. [18] tested strength and shrinkage of concrete using 20-mm recycled coarse aggregates of preserved quality and commercial quality (density = 2,310-2,320 kg/m$^3$, absorption = 4.88-5.32%, w/c = 0.45). They have suggested that the use of 100% RA resulted in some reduction in the strength, but the use of 30% RA as partial replacement of coarse aggregate produced compressive strength similar to that of concrete with 100% virgin coarse aggregate. Doming et al. [16] used 20-mm nominal size RA which replaced NA at 20%, 50%, and 100% (density = 2,460 kg/m$^3$, absorption = 5.19-6.08%, w/c = 0.5). They have observed that when the effective w/c was maintained constant, the compressive strength was the same so the substitution of NA by RA did not have a significant effect. Geng et al. [14] used 25-mm nominal size RA which replaced NA by 100% (density = 2,713 kg/m$^3$, absorption = 5.07%, w/c = 0.45). They have observed some reduction of compressive strength from 100% RA concrete than NA concrete, but the reduction was less than 10%.

Figure 6 shows the tensile strength test results in terms of both split tensile strength (black bar) and flexural strength (white bar) at 28 days. The split tensile strength ($f_{tu}$) test data show that the split tensile strength is not much affected by replacement of NA by RA (or CRA) with exception of RAC-100. On the other hand, the flexural strength ($f_f$) tends to decrease with increasing replacement ratio of NA with RA (or CRA). However, the flexural strengths are above the flexural strength predicted by the KCI Structural Concrete Design Code [28]. It can be concluded that the split tensile strength is not significantly affected by replacing NA with RA up to 100%. The flexural strength is negatively affected with increasing replacement ratio of NA with RA, but it still satisfies the code required flexural strength.

Table 3 and Figure 7 show elastic modulus measured at 28 days. The 30% replacement of NA by RA (or CRA) does not influence the elastic modulus. For 50% and 100% RA replacement, the elastic modulus reduces a little but are at least 90% that of NAC and are within 10% margin from the predicted value by KCI Structural Concrete Design Code [28]. It needs to be noted that due to relatively soft adhered
mortar, the elastic modulus of RA is typically lower than that of NA which in turn negatively affect the elastic modulus of RAC. Such negative effect is not clearly shown in Table 3 and Fig. 7. Table 3 also shows Poisson’s Ratio for NAC, RAC, and CRAC. Despite some scatter in test data, the Poisson’s Ratios range between 0.18 and 0.22 and there is no clear indication that the Poisson’s Ratio is influenced by replacement of NA by RA even up to 100% replacement.

3.3 Shrinkage of recycled aggregate concrete

Figure 8 shows temperature and R.H. in the environmental chamber during the test period. It can be seen that the temperature is relatively constant at 19.5°C and varies between 19°C and 21°C. R.H. ranges between 48% and 85% in the chamber reflecting seasonal variation of outside weather with a mean of 59.7%.

Figure 9 shows measured shrinkage strain versus time. Shrinkage strains show steady and fast increase during the first two weeks followed by steady increase during the next three months. After about 3-1/2 months, the shrinkage strain development somewhat levels out (or it still keeps increasing but at a much lower rate). Therefore, in Table 4, the shrinkage strain values are summarized at each time mark, i.e. after 2 weeks, 3-12/ months, and 6 months (180 days).

| Table 3 – Summary of strength, elastic modulus and Poisson’s Ratio |
|---------------------------------|-----------------|-----------------|-----------------|-----------------|
| Index                          | Compressive strength, $f_{cu}$ (MPa) | Tensile strength (MPa) | Elastic Modulus (GPa) | Poisson’s Ratio |
|                                | $f_3$ | $f_7$ | $f_{28}$ | $f_56$ | $f_90$ | $f_{180}$ | $f_{sp}$ | $f_r$ |                                |
| NAC                            |       |       |         |       |       |         |         |       |                                |
| mean                           | 22.4  | 25.6  | 34.4    | 35.6  | 37.9  | 39.4    | 2.56    | 6.10  | 26.1                      | 0.19 |
| min.                           | 21.9  | 23.6  | 33.2    | 35.2  | 37.6  | 39.1    | 2.55    | 5.81  | 25.9                      | 0.18 |
| max.                           | 22.8  | 25.7  | 36.9    | 36.3  | 38.8  | 39.5    | 2.57    | 6.39  | 26.2                      | 0.20 |
| RAC-30                         |       |       |         |       |       |         |         |       |                                |
| mean                           | 25.2  | 28.8  | 37.2    | 37.9  | 37.5  | 39.6    | 2.62    | 6.00  | 28.1                      | 0.20 |
| min.                           | 25.2  | 27.5  | 36.3    | 36.9  | 34.7  | 38.0    | 2.49    | 5.82  | 26.2                      | 0.18 |
| max.                           | 25.3  | 29.8  | 37.8    | 38.8  | 39.9  | 42.6    | 2.75    | 6.18  | 29.9                      | 0.22 |
| RAC-50                         |       |       |         |       |       |         |         |       |                                |
| mean                           | 21.0  | 25.1  | 33.1    | 34.2  | 35.0  | 37.5    | 2.67    | 5.67  | 23.6                      | 0.21 |
| min.                           | 20.9  | 23.4  | 31.9    | 33.9  | 33.0  | 37.1    | 2.66    | 5.42  | 21.5                      | 0.18 |
| max.                           | 21.2  | 26.2  | 34.3    | 34.4  | 36.0  | 37.8    | 2.68    | 5.92  | 25.9                      | 0.24 |
| RAC-100                        |       |       |         |       |       |         |         |       |                                |
| mean                           | 22.1  | 24.5  | 34.4    | 33.2  | 35.0  | 36.6    | 2.24    | 5.25  | 25.2                      | 0.20 |
| min.                           | 21.7  | 22.8  | 33.8    | 32.5  | 34.0  | 33.1    | 1.85    | 5.12  | 24.5                      | 0.17 |
| max.                           | 22.7  | 26.0  | 34.7    | 34.1  | 37.2  | 40.3    | 2.63    | 5.38  | 26.2                      | 0.22 |
| CRAC-30                        |       |       |         |       |       |         |         |       |                                |
| mean                           | 23.1  | 27.5  | 33.6    | 36.1  | 37.8  | 41.0    | 2.71    | 5.59  | 25.7                      | 0.22 |
| min.                           | 22.6  | 26.0  | 32.6    | 35.2  | 36.1  | 40.7    | 2.20    | 5.50  | 24.6                      | 0.18 |
| max.                           | 23.7  | 28.5  | 34.3    | 36.9  | 39.9  | 41.3    | 3.22    | 5.69  | 27.1                      | 0.27 |
| CRAC-50                        |       |       |         |       |       |         |         |       |                                |
| mean                           | 15.5  | 19.5  | 27.0    | 28.0  | 28.5  | 32.2    | 2.48    | 5.38  | 25.4                      | 0.20 |
| min.                           | 15.2  | 18.3  | 24.6    | 26.8  | 26.4  | 31.0    | 2.43    | 5.18  | 23.7                      | 0.19 |
| max.                           | 15.7  | 21.3  | 29.5    | 29.6  | 30.34 | 33.3    | 2.54    | 5.57  | 26.5                      | 0.23 |
| CRAC-100                       |       |       |         |       |       |         |         |       |                                |
| mean                           | 17.6  | 19.0  | 27.5    | 30.3  | 27.4  | 30.3    | 2.73    | 5.16  | 23.8                      | 0.18 |
| min.                           | 16.2  | 18.0  | 24.4    | 29.6  | 24.4  | 29.2    | 2.64    | 5.14  | 23.4                      | 0.17 |
| max.                           | 19.5  | 20.7  | 30.6    | 31.0  | 32.8  | 32.8    | 2.82    | 5.18  | 24.0                      | 0.20 |

NOTE: Compressive strength is average of three test; tensile strength is average of two tests; elastic modulus and Poisson’s Ratio are taken at 28 days.
Fig. 4 – Compressive strength test results by age

Fig. 5 – Compressive strength development of NAC, RAC and CRAC

(a) NAC vs. RAC  
(b) NAC vs. CRAC

Fig. 6 – Tensile strength at 28 days
In Table 4, in case of NAC, the shrinkage strain is 206 μm/m after 2 weeks, 690 μm/m after 3-1/2 months, and 712 μm/m after 6 months. It is seen that about 29% of the maximum shrinkage developed during the first two weeks and about 97% of the maximum shrinkage (recorded in 6-months period) developed during the first 3-1/2 months.

Table 4 also shows the shrinkage strains of RAC and CRAC normalized by that of NAC. In Table 4 and Figure 9, the shrinkage of NAC, RAC-50, and CRAC-50 is almost the same (84%-103% of NAC). The shrinkage of RAC-100 and CRAC-100 is larger than that of NAC at all ages (103%-114% of NAC). It can be suggested that the shrinkage of 50% recycled coarse aggregate concrete is the same as that of NAC and the shrinkage of 100% recycled coarse aggregate concrete is larger than that of natural aggregate concrete. In addition, there are no differences in the shrinkage behavior between RAC and CRAC for all replacement ratios tested.

Manzi et al. [19] reported shrinkage strain of about 860 μm/m at 180 days from RAC with 63.5% RA (density 2,250-2,430 kg/m³, w/c = 0.48, absorption = not disclosed) and 650-680 μm/m at 180 days from RAC with 36.5% RA. The shrinkage strains reported by Manzi et al. are comparable to those measured in this study. Other researchers reported larger shrinkage strains. Andal et al. [18] reported 50% more shrinkage from 100% replacement of RA (density = 2,310-2,320 kg/m³, absorption = 4.88-5.32%, w/c = 0.45) after 180 days. Domingo et al. [16] reported about 20% increased shrinkage from RAC with 50% RA replacement and about 70% more shrinkage with 100% RA replacement after 180 days (density = 2,460 kg/m³, absorption = 5.19-6.08%, w/c = 0.5). The reason for the relatively small amount of shrinkage strains compared to that of NAC determined from this study can be relatively good quality RA with low amount of water absorption (absorption = 3.84%, 4.88-5.32%, and 5.19-6.08% for RA used in this study, by Andal et al., by Domingo et al., respectively). Although the effective w/c can be maintained during batching, concrete using RA with higher absorption capacity will lead to more porous concrete, which can be more susceptible for moisture loss inducing increased drying shrinkage.

Figure 10 shows the shrinkage strains for NAC, RAC-50, and RAC-100 predicted by ACI 209 Technical Committee report [29]. Shrinkage strain-time curves of NAC and RAC-50 predicted by ACI 209 are in good agreement with the current test data especially after 180 days. Measured shrinkage of RAC-100 in this study is higher than the ACI 209 predicted value, while the difference is about 15%.

3.4 Creep of recycled aggregate concrete
Creep test began 35 days after casting and continued for 150 days. Creep test specimens were cured under water right after demolding until the test day (t = 35 days). Sustained load that corresponds to about 30% of the 28-day compressive strength (See Table 3) was applied at 35 days and the same sustained load was maintained for the duration of the creep test. A total of five different concretes was tested: NAC, RAC-50, RAC-100, CRAC-50, and CRAC-100. Since two creep specimens were tested using one loading frame, RAC-50 and RAC-100 were tested using the same loading frame and hence were subjected to the same sustained load. CRAC-50 and CRAC-100 were also tested using the same loading frame. NAC was tested while the overall test procedure followed ASTM C512 recommendations [30].
Table 4 – Shrinkage strain vs. time (unit: μm/m)

<table>
<thead>
<tr>
<th>Index</th>
<th>Time after shrinkage measurement</th>
<th>Shrinkage normalized by NAC at time of</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2 weeks</td>
<td>3-1/2 months</td>
</tr>
<tr>
<td>NAC</td>
<td>206</td>
<td>690</td>
</tr>
<tr>
<td>RAC-50</td>
<td>175</td>
<td>697</td>
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<tr>
<td>RAC-100</td>
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<td>747</td>
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<tr>
<td>CRAC-50</td>
<td>173</td>
<td>706</td>
</tr>
<tr>
<td>CRAC-100</td>
<td>213</td>
<td>746</td>
</tr>
</tbody>
</table>

Fig. 8 – Temperature and R.H. in environmental chamber

Fig. 9 – Shrinkage vs. time

Table 5 summarizes the creep test results. Figure 11 shows the total strain developed in all creep test specimens that include short-term strain (elastic strain) at \( t = 35 \) days and long-term strain that consists of shrinkage strain and creep strain. Both elastic strain and shrinkage strain were deducted from the total strain to determine the net creep strains, and the results are shown in Fig. 12. It needs to be noted that the sustained load level each creep test specimen is subjected to is different, and therefore the creep strains shown in Fig. 12 need to be normalized in terms of unit sustained stress (i.e. 1 MPa). The results are shown as specific creep in Fig. 13. In Table 5 and Figure 13, it is seen that NAC experiences the smallest specific creep of 81 μm/m/MPa after 150 days. The largest specific creep is determined from RAC-100 that is 112 μm/m/MPa (138% of NAC) followed by RAC-50 with 101 μm/m/MPa (125% of NAC). The creep test results show that the creep of RAC is significantly larger than that of NAC.
Current test data match well with those available in the literature. Manzi et al. [19] showed that specific creep of RAC with 63.5% RA (density 2,250-2,430 kg/m³, w/c = 0.48) substitution was 90 μm/m and 90-95 μm/m for RAC with 36.5% RA substitution after 150 days. Domingo et al. [16] reported that the specific creep of RAC with 50% RA (density = 2,460 kg/m³, absorption = 5.19-6.08%, w/c = 0.5, sustained loading about 30% of compressive strength started at 28 days) substitution increased over that of NAC by 29% and increased for RAC with 100% RA substitution by 32% over that of NAC. In addition, Rye et al. [15] after systematic analysis of existing data matrix available in literature has concluded that the creep of concrete increases with a decreasing rate with increasing RA, giving an average increase of 32% at 100% RA content. The specific creep of CRAC is smaller than that of RAC: i.e. it is 96 μm/m/MPa for CRAC-50 (119% of NAC). Also specific creep of 91 μm/m/MPa is determined from CRAC-100 that is 112% of NAC. It may needs to be noted that the specific creep of CRAC-100 is smaller than CRAC-50 after 150 days. In the longer term, the difference between the two creep test data may become smaller in Fig. 13.

Creep coefficient (Φ) is shown in Table 5 and Fig. 14 for all creep test specimens which ranges between 1.97 for NAC and 2.76 for RAC-100. Therefore the creep coefficient of RAC-100 is as large as 140% of NAC. In case of RAC-50, the creep coefficient is 136% of NAC. The creep coefficient of CRAC-50 and CRAC-100 is 115% and 120% that of NAC, respectively. The Φ value is smallest for NAC as expected followed by CRAC-50, CRAC-100, RAC-50, and RAC-100. Again the current test data match well with those published. Geng et al. [14] reported that the creep coefficient ratio (Φ/ΦNAC) is about 1.3 for RAC with w/c = 0.5 including 100% RA (RA density = 2,713 kg/m³, absorption = 5.07%). Figure 15 shows the creep coefficient predicted by ACI 209 Technical Committee report for NAC, RAC-50, and RAC-100 [29]. Creep coefficient-vs.-time curves predicted by ACI 209 are lower than the current test data shown in Fig. 14. The difference is about 30% for NAC and it is much larger for RAC-50 and RAC-100. The prediction of creep strains depends on many influencing factors such as time of loading, temperature, R.H., strength as well as some fresh concrete properties. Especially the large differences between the ACI predicted values and the current test data means that the ACI 209 formula developed exclusively for NAC is not applicable for RAC.

Table 5 – Creep test results 150 days after sustained load application

<table>
<thead>
<tr>
<th>Index</th>
<th>NAC</th>
<th>RAC-50</th>
<th>RAC-100</th>
<th>CRAC-50</th>
<th>CRAC-100</th>
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</thead>
<tbody>
<tr>
<td>Total strain (μm/m)</td>
<td>1,575</td>
<td>1,601</td>
<td>1,754</td>
<td>1,358</td>
<td>1,347</td>
</tr>
<tr>
<td>Shrinkage (μm/m)</td>
<td>321</td>
<td>262</td>
<td>292</td>
<td>303</td>
<td>351</td>
</tr>
<tr>
<td>Elastic strain (μm/m)</td>
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<td>364</td>
<td>389</td>
<td>322</td>
<td>297</td>
</tr>
<tr>
<td>Creep strain (μm/m)</td>
<td>831</td>
<td>976</td>
<td>1,074</td>
<td>733</td>
<td>699</td>
</tr>
<tr>
<td>Specific creep (μm/m/MPa)</td>
<td>81</td>
<td>101</td>
<td>112</td>
<td>96</td>
<td>91</td>
</tr>
<tr>
<td>Creep coefficient Φ/ΦNAC</td>
<td>1.97</td>
<td>2.68</td>
<td>2.76</td>
<td>2.27</td>
<td>2.36</td>
</tr>
<tr>
<td>Creep coefficient Φ/ΦNAC</td>
<td>1.0</td>
<td>1.36</td>
<td>1.40</td>
<td>1.15</td>
<td>1.20</td>
</tr>
</tbody>
</table>

NOTE: Applied stress is 10.3 MPa for NAC; 9.63 MPa for RAC-50 and RAC-100; and 7.66 MPa for CRAC-50 and CRAC-100.
Fig. 11 – Total strain vs. time

Fig. 12 – Creep vs. time after loading

Fig. 13 – Specific creep vs. time after loading
4. Conclusions

Strength development, shrinkage, and creep of RAC was investigated. RA replaced NA by 30%, 50%, and 100% by vol. For the investigation of strength development, 3, 7, 28, 56, 90, and 180 day compressive strength and 28 day split tensile strength and flexural strength have been determined. Shrinkage and creep behavior of RAC with 0%, 50%, and 100% RA replacing NA has been studied up to 180 days and 150 days, respectively. RAC including RA treated with 3-day accelerate carbonation (CRA) was also used. The following conclusions are drawn from this study.

(1) The physical properties of RAC, such as density and water absorption, improve by accelerated carbonation, but the improvement of mechanical properties is not directly related to the strength improvement of RAC incorporating the CO₂ treated RA.

(2) Compressive strength of RAC with 30% RA or CRA is similar to that of NAC and the compressive strength of RAC with 50% or 100% replacement is reduced from that of NAC, while the strength reduction is smaller than 10%.

(3) Split tensile strength is not significantly affected by RA replacement up to 100%. Flexural strength decreases with increasing amount of replacement, but the flexural strengths are above the value required by structural concrete design code.

(4) Elastic modulus of RAC tends to decrease with increasing RA replacement, but it is not significantly reduced (less than 10%).

(5) Shrinkage of RAC with 50% RA is similar to that of NAC. Shrinkage of RAC with 100% RA increases over that of NAC up to 13% after 180 days.

(6) Specific creep of RAC with 50% and 100% RA increases over that of NAC by 25% and 38%,
respectively, after 150 days of sustained loading.

(7) RAC with CO$_2$ treated RA has similar shrinkage behavior to RAC with RA not treated by CO$_2$ while creep of RAC with CO$_2$ treated RA is smaller than that of RAC with RA not treated with CO$_2$.

Acknowledgement
This work was supported by the Korea Technology and Information Promotion Agency for SMEs (TIPA) grant funded by the Korea government (Proj. No.: C0531527). Authors also gratefully acknowledge the support from Dongbu-ENT Corp. in the form of recycled coarse aggregate supply throughout this study.

References

Technical Paper

Visual investigation method and structural performance evaluation for DEF induced damaged Indian Railway PC sleepers

Rajamurugan Sundaram*, Koji Matsumoto, Kohei Nagai and Anupam Awasthi

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Abstract: Indian Railways uses pre-stressed concrete (PSC) sleepers for its tracks. In the recent years in north and central part of railways, premature cracks were observed in sleepers. Cracks were observed for sleepers 6 to 9 years after their manufacture. In this research, structural performance of damaged sleepers was evaluated in both material and structural level. For material level investigation, core samples were taken from the damaged sleepers. From the cores, reduction in static elastic modulus and compressive strength were observed for damaged sleepers when compared to undamaged sleepers. For structural level investigation, based on the level of the damage, sleepers were categorized into damaged and undamaged sleepers, and flexural test and shear test for sleepers were conducted. From the bending and shear test results, relationship between the crack pattern and capacity reduction in damaged sleepers was studied. In the flexural test, it was found out that more layers of longitudinal cracks in the central parts reduced its capacity up to 35% when compared to undamaged sleepers. Shear test results showed that it was very close to the minimum requirement for capacity of the sleepers.

Keywords: delayed ettringite formation (DEF), alkali silica reaction (ASR), sleeper, concrete.

1. Introduction

The main objective of railways is to provide safe transport for passengers. Indian Railways uses pre-cast pre-stressed concrete sleepers from factories for its track. Concrete sleepers are important part of railway track to hold the rail in the position and transfer the load to the supporting structure below. In northern and central part of the railways, these sleepers start cracking 6 to 9 years after their manufacture. Even unloaded sleepers, which are not used in the tracks, also get cracked. As per the recent survey conducted in the central part of India, in a 100-km railway track and out of 128,000 sleepers, 45,000 sleepers were damaged with premature cracking. This means, in this particular railway track of 100 km, almost 35% of the sleepers are damaged. The damaged sleepers are replaced annually: when failure cracks at insert location is observed sleepers are replaced immediately. No clear investigation has been carried to find the current capacity of these damaged sleepers. This research aims to find out the structural performance of damaged sleepers based on the existing crack patterns in the sleepers. For this purpose of investigation, research was carried out to find out sleepers with different levels of damage. This problem is a potential threat to the Indian Railways and it affects the safety of the passengers travelling in trains.

In Indian Railways, high temperature more than 70°C has been recorded during steam curing in the manufacturing process of sleepers. This may cause Delayed Ettringite Formation (DEF) and be the main cause of these premature cracking. Research was conducted in the past study in Indian Railways to find out the cause of these premature cracking. This research was conducted in the past study in Indian Railways to find out the cause of expansive damages in concrete sleepers, where it was found out the cause of these cracks were due to both DEF and Alkali Silica Reaction (ASR) [1]. The ASR is caused by the chemical reaction between cement alkali and reactive aggregate that generates expansive ASR gel. In DEF, dissolved ettringite under high temperature curing is reformed after the concrete hardening to cause expansive stress. Experimentation with German high early
strength cement concluded that delayed expansion occurs when specimens were cured above 80°C [3]. Boundary temperature conditions for occurrence of DEF were between 60°C and 70°C [4]. To understand the DEF a holistic approach for late sulphate release, micro-cracking, and exposure to water was proposed [5]. Cement composition (alkalis, C₃S, C₃A, SO₃, and MgO) and fineness also influence the effect of DEF [2]. Typical crack patterns observed in the sleepers are shown in Fig. 1. Map cracks are observed at the ends and longitudinal cracks are observed at the midspan. In the past research of premature cracking, similar problems were observed in other parts of the world. In 2004, prestressed monobloc concrete sleepers placed in Portugal had shown premature cracking [6]. Sleepers manufactured in the years between 1992 and 1996 in Sweden have started to deteriorate and cracks were observed in sleepers [7]. As per a report in Finland 20,000 sleepers are replaced every year [8]. Distress in prestressed concrete sleepers was observed in eastern coast of United States [9]. It was predicted that causes of these cracks could be either DEF or ASR. From all these research, it is observed that damaged sleepers are huge in number and it is not possible economically to replace all the sleepers immediately. For rational and efficient maintenance, it is essential to set priorities for replacement considering the structural performances of sleepers. However, the past research including the authors study do not sufficiently clarify structural performances [1]. For this purpose of investigation, sleepers with various degrees of damages were selected to study the crack patterns. Material level testing and structural level testing were conducted in the sleepers to find out the current damage conditions of the sleeper. For material testing, core samples were taken from the damaged and undamaged sleepers, where compressive strength and static elastic modulus were found from the compressive test. For structural level investigation, crack patterns in the damaged sleepers were studied where map cracks were observed at the end of the sleepers and longitudinal cracks at the central part of the sleepers. Side view and top view of the damaged sleepers are shown in Fig. 1. Cracks are located at the insert location of the sleeper, which is a failure crack in the sleepers. Based on the level of damage, cores and sleepers were collected from the damaged and undamaged sleepers. Structural level testing was also conducted on both undamaged and damaged sleepers, where flexural test and shear test were conducted on the damaged and undamaged sleepers to find out the current capacity of the sleepers. Existing crack patterns from the damaged sleepers were studied and failure pattern was analyzed. The relationship between existing crack patterns and capacity reduction in damaged concrete sleepers was studied.

Sleepers are placed on ballasted track bed as shown in Fig. 2. This ballast track bed acts as an elastic bed and transfer the load coming from sleeper to wider area of formwork. This formation is earthen embankment and its not bonded to concrete structure.

2. Proposal of visual inspection method

Visual inspection was conducted. Based on the damages observed in the sleepers, sleepers were classified into two categories such as Damaged and Undamaged sleepers (See Table 1). Within damaged and undamaged sleepers, sleepers were further classified into five categories. Undamaged sleeper’s two categories were Undamaged new and Undamaged old. Sleepers in these categories did not have any visual cracks. Damaged sleepers were categorized into three categories such as Mild, Moderate and Severe damage. In Mild damaged sleepers edge cracks were observed in the sleepers, whereas in Moderate damaged sleepers one longitudinal crack was observed in the center of the sleepers. In Severe damaged sleepers, more than 2 to 3 longitudinal cracks were observed in the center.

Description of these cracks with crack widths is shown in Table 1. Typical view of these crack patterns is shown in Fig. 11. Cracks typically occur around 6 to 9 years after their manufacture. The ballast profile and crack sequence are shown in Figs. 2 and 3, respectively. When sleepers are in service, i.e. installed condition on track, first cracks are usually seen on the side face of the sleeper. The side face is covered with ballast and cracks are visible only after opening the ballast. In later stages cracks are also seen on the top surface of the sleeper. Ultimately failure occurs near the inserts [1].

Indian Railways uses M55 grade sleepers for its tracks (See Table 2). 3 ply of 3 mm high tensile strength strands are used in sleepers (See Fig. 4). According to the specification, each reinforcing strand is to be tensioned with initial force of 27 kN. Typical tendon profile and cross section of sleepers are shown in Fig. 4.

SEM analysis was conducted in the past study from concrete samples collected from Indian Railways, where presence of DEF from the concrete samples was observed (See Fig. 5). In order to find the effect of ASR in concrete, the authors also conducted the chemical analysis. It was found from the chemical analysis that alkali content in cement samples was greater than 5 kg/m³ (Table 3). Whereas as per Japanese standards, alkali amount should not exceed more than 3 kg/m³. This high alkali content in samples also promotes ASR.
Table 2 – Concrete mix proportions in sleepers

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse aggregate, CA1 (20 mm)</td>
<td>981.49 kg (50.50%)</td>
</tr>
<tr>
<td>Coarse aggregate, CA2 (20 mm)</td>
<td>420.64 kg (21.50%)</td>
</tr>
<tr>
<td>Fine aggregate, FA</td>
<td>546.28 kg (28.00%)</td>
</tr>
<tr>
<td>Cement (53-S)</td>
<td>445.50 kg/m³</td>
</tr>
<tr>
<td>Admixture</td>
<td>2.227 kg (0.5%) of cementitious material</td>
</tr>
<tr>
<td>Water</td>
<td>142.56 Liters</td>
</tr>
<tr>
<td>W/C ratio</td>
<td>0.32</td>
</tr>
<tr>
<td>A/C ratio</td>
<td>4.373</td>
</tr>
</tbody>
</table>

Fig. 1 – Typical cracking pattern observed in the sleeper of Indian Railways
Fig. 2 – Ballast profile track slab

(a) First Step (Cracks at side face)  (b) Second Step (Cracks visible on top surface)

(c) Third Step (Cracks near insert location)

Fig. 3 – Crack sequences

(a) Cross section of sleepers (unit: mm)  (b) 3 x 3mm wire strands

Fig. 4 – PS tendon profile
SEM analysis was conducted in the past study from concrete samples collected from Indian Railways, where presence of DEF from the concrete samples was observed (See Fig. 5). In order to find the effect of ASR in concrete, the authors also conducted the chemical analysis. It was found from the chemical analysis that alkali content in cement samples was greater than 5 kg/m$^3$ (Table 3). Whereas as per Japanese standards, alkali amount should not exceed more than 3 kg/m$^3$. This high alkali content in samples also promotes ASR.

<table>
<thead>
<tr>
<th>Factories</th>
<th>Component analysis result of cement (mass %)</th>
<th>Alkaline amount (kg/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Na$_2$O</td>
<td>K$_2$O</td>
</tr>
<tr>
<td>1st Factory</td>
<td>0.27</td>
<td>1.28</td>
</tr>
<tr>
<td>2nd Factory</td>
<td>0.27</td>
<td>1.30</td>
</tr>
<tr>
<td>3rd Factory</td>
<td>0.27</td>
<td>1.28</td>
</tr>
<tr>
<td>Average</td>
<td>0.27</td>
<td>1.29</td>
</tr>
</tbody>
</table>

(a) Typical side view of Severe damaged sleeper
(b) Mild damage
(c) Moderate damage
(d) Severe damage

Fig. 6 – Cut cross section in sleepers
3. Internal crack pattern and tests of core samples

3.1 Cross section cut in sleepers

Sleepers were cut to find whether cracks have infiltrated inside the sleepers. Sleepers were collected from both damaged and undamaged sleepers (See Fig. 6), where manufacturing year of sleeper is also mentioned (See Table 4).

Except “Severe damaged sleeper” where three sleepers were taken for cross section cut, in rest of the categories only a sleeper was cut. Typical side view of cross section cut is shown in Fig. 6(a), where three cuts were made on each sleeper. Critical cross sections based on the category with more cracks are shown in this Fig. 6. For both undamaged new and undamaged sleepers no cracks were observed inside the sleepers, crack infiltration up to 20 mm was only observed in the sleepers and cracks did not infiltrate fully inside the sleepers. It is observed from the study that DEF expansion occurs ununiformly and only the inner portion is expanded, outer side is put in tension in the circumferential direction as we observed in the ring tension behavior, resulting in large cracks only in the outer side of the sleepers.

Table 4 – List of all the specimens used for cross section cut

<table>
<thead>
<tr>
<th>Category</th>
<th>Manufacture year</th>
<th>No. of sleepers</th>
<th>Crack width Left side</th>
<th>Central part</th>
<th>Right side</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undamaged new</td>
<td>2015</td>
<td>1</td>
<td>No cracks</td>
<td>No cracks</td>
<td>No cracks</td>
</tr>
<tr>
<td>Undamaged old</td>
<td>1986</td>
<td>1</td>
<td>No cracks</td>
<td>No cracks</td>
<td>No cracks</td>
</tr>
<tr>
<td>Mild damage</td>
<td>2006</td>
<td>1</td>
<td>1 mm cracks</td>
<td>1 mm cracks</td>
<td>No cracks</td>
</tr>
<tr>
<td>Moderate damage</td>
<td>2006</td>
<td>1</td>
<td>1 mm</td>
<td>1 layer of 1-2 mm</td>
<td>1-2 mm</td>
</tr>
<tr>
<td>Severe damage-I</td>
<td>2002</td>
<td>1</td>
<td>1-2 mm</td>
<td>3 layers of 2-3 mm</td>
<td>1-2 mm</td>
</tr>
<tr>
<td>Severe damage-II</td>
<td>2006</td>
<td>1</td>
<td>1-2 mm</td>
<td>3 layers of 2-3 mm</td>
<td>1-2 mm</td>
</tr>
<tr>
<td>Severe damage-III</td>
<td>2002</td>
<td>1</td>
<td>1-2 mm</td>
<td>3 layers of 2-3 mm</td>
<td>1-2 mm</td>
</tr>
</tbody>
</table>

Fig. 7 – Side view of damaged and undamaged sleepers for cores samples

Table 5 – Cores samples for compressive test

<table>
<thead>
<tr>
<th>Category</th>
<th>Damaged sleeper</th>
<th>Manufacturing year</th>
<th>Number of cores</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undamaged</td>
<td>New</td>
<td>2015</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Old</td>
<td>2002</td>
<td>2</td>
</tr>
<tr>
<td>Damaged</td>
<td>Mild</td>
<td>2002</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Severe</td>
<td>2002</td>
<td>2</td>
</tr>
</tbody>
</table>
3.2 Compressive test of core sample

Cores were collected from both damaged and undamaged sleepers for compressive test (See Fig. 7), where manufacturing year of sleeper is also included (See Table 5). One typical view of core sample is shown in Fig. 7. Compressive strength and static elastic modulus were obtained from the compressive test of core samples. From the compressive test results, peak load of core samples was obtained and compressive strength of core samples were calculated. It was observed that average compressive strength for several damaged specimens was 13.2 MPa and whereas average compressive strength of 53 MPa was observed for Mild damaged core. Figure 8 shows the compressive strength results of the core samples. Static elastic modulus was calculated and the results are compared in Fig. 9. Minimum static elastic modulus of 5 GPa was recorded in the Severe

<table>
<thead>
<tr>
<th>Manufacturing year</th>
<th>Number of sleepers</th>
</tr>
</thead>
<tbody>
<tr>
<td>2015</td>
<td>3</td>
</tr>
<tr>
<td>1986</td>
<td>3</td>
</tr>
<tr>
<td>2006</td>
<td>3</td>
</tr>
<tr>
<td>2006</td>
<td>3</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Category</th>
<th>Number of sleepers and manufacturing year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undamaged new</td>
<td>3 Nos. in 2015</td>
</tr>
<tr>
<td>Undamaged old</td>
<td>3 Nos. in 1986</td>
</tr>
<tr>
<td>Mild damage</td>
<td>1 Nos. in 2006</td>
</tr>
<tr>
<td>Moderate damage</td>
<td>3 Nos. in 2006</td>
</tr>
<tr>
<td>Severe damage</td>
<td>2 Nos. in 2006</td>
</tr>
</tbody>
</table>

Fig. 8 – Compressive strength of core samples

Fig. 9 – Static elastic modulus of core samples

Fig. 10 – Schematic view of flexural test arrangement (Dimensions in mm)
damaged core, whereas maximum static elastic modulus is recorded in undamaged old core of about 37 GPa (Fig. 8). Even though no visual cracks were observed in the cores, reduction in material strength up to 75% was observed in Severe damaged sleepers when compared to Mild damaged sleepers. It was found from the study that outer crack is an indication to show the capacity reduction in sleepers, but this is not the main reason for capacity reduction in cores. Main reason is concrete is affected in material level due to expansion in concrete due to DEF and ASR.

![View of crack patterns of sleepers before loading](image)

![Result of flexural test](image)

**Fig. 11** – View of crack patterns of sleepers before loading

**Fig. 12** – Result of flexural test
4. Loading test for sleepers

4.1 Flexural test

Fifteen sleepers with different levels of damage were collected from Indian Railways (See Table 6). Within these 15 sleepers, sleepers were further categorized into five categories such as “Undamaged new”, “Undamaged old”, “Mild damage”, “ Moderate damage” and “Severe damage” based on the level of damage (See Table 7). Flexure test was conducted to find out the flexural capacity of the sleepers (See Fig. 10). Transducers were fixed at the loading points and at the center part of the sleeper to find midspan deflection. Sleepers were loaded in flexure and allowed to be loaded until the failure is observed from the sleeper [1].

Fig. 13 – Typical view of sleeper after failure (Severe damaged)

Fig. 14 – Side view – comparison of capacity in damaged sleeper
Table 8 – Number of tested sleepers based on level of damage for shear test

<table>
<thead>
<tr>
<th>Category</th>
<th>Manufacturing year</th>
<th>Number of sleepers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undamaged new</td>
<td>2015</td>
<td>1</td>
</tr>
<tr>
<td>Severe damage</td>
<td>2002</td>
<td>3</td>
</tr>
</tbody>
</table>

Fig. 15 – Schematic shear test arrangement

Fig. 16 – Typical view of crack patterns of sleepers

a) Undamaged New

b) Severe Damaged Sleeper 3 mm cracks
Fig. 17 – Result of shear test

Fig. 18 – Typical view of sleeper after shear failure

Fig. 19 – Existing crack patterns and shear test results
4.1 Results of flexure test

From the flexure test results, peak loads were obtained for all categories of sleepers (See Fig. 12). It is clearly seen that not only peak load but also stiffness is significantly reduced in the case of severe damaged sleepers. One of the typical views of sleeper after the failure is shown in Fig. 13. Mode of failure is supposed to be flexural compression failure which occurs before yielding of PC strands.

4.1.1 Results of shear test

Shear test was also conducted for sleepers. Illustrated view of shear test arrangement is shown in Fig. 15. Transducers were fixed near the loading points and at the central part of the sleeper to find midspan deflection. Shear test was conducted on one side first and after completion the test was repeated over the other side on the same sleeper. Sleepers were categorized into “Undamaged new” and “Severe damaged” (See Table 8). Typical view of these categories of sleepers is shown in Fig. 16.

4.1.2 Relationships between flexural capacities and visible crack pattern

From the flexural test results, it shows that Severe damaged sleepers show capacity reduction (See Fig. 14). Average peak load of Severe damaged sleeper is 56% capacity reduction. Average peak load of Moderate damaged sleeper is same as Undamaged old sleepers, but in one of the cases Moderate damaged sleeper shows 8% capacity reduction. Average peak load of Mild damaged sleeper is 7% less than Undamaged old sleepers, but in the cases Mild damaged sleeper shows 12% capacity reduction. By comparing the results of both Undamaged old and new sleepers less than 2% difference was observed. If 2 to 3 layers of longitudinal cracks were observed in the central part of sleeper these cracks reduce the capacity of the sleeper.

Minimum peak load as per Indian standards for flexural test is 60 kN. Even though severe damaged sleeper showed reduction in peak load, still the minimum peak load observed in these sleepers was 127 kN. Therefore, sleepers are safe in flexure. The evaluation method in which damage level is classified are appropriate. The evaluation method in which damage level is classified are appropriate, where damage sleepers show lesser capacity compared to undamaged sleepers.

4.2 Shear test

Shear test was conducted for sleepers. Illustrated view of shear test arrangement is shown in Fig. 15. Transducers were fixed near the loading points and at the central part of the sleeper to find midspan deflection. Shear test was conducted on one side first and after completion the test was repeated over the other side on the same sleeper. Sleepers were categorized into “Undamaged new” and “Severe damage” (See Table 8). Typical view of these categories of sleepers is shown in Fig. 16.

4.2.1 Results of shear test

From the shear test results, peak loads were obtained from Undamaged new and Severe damaged sleeper (See Fig. 17). View of the sleeper after failure is shown in Fig. 18. Shear capacity and member stiffness in severe damaged sleepers significantly reduced as observed in the flexure tests.

4.2.2 Relationship between shear capacities and visible crack patterns

Figure 19 shows the existing crack patterns available in sleepers. Average peak load of Severe damaged sleeper is 36% less than the Undamaged old sleeper, but in one of the cases Severe damaged sleeper shows 42% capacity reduction. Minimum requirement for the peak load as per Indian standards is 230 kN. In one of the cases peak load is 250 kN and this is close to the minimum requirement for the peak load. Cracks were observed in severe damage sleeper, where more layers of end cracks and more layers of longitudinal cracks were observed in damaged sleepers. It indicates that shear capacities are also correlated.

Future deteriorations such as corrosion in the mild and moderate damage sleepers seem not to be a significant issue since the surrounding environment of the sleepers is rather mild (no chlorides, no sulfate, etc.), therefore verifications of durability performances are not an emergent problem. However, considering that environmental conditions in the other regions are different from those in this study, investigation on the durability performance of damaged sleepers is also an important issue in the future.

5. Conclusions

In this study, the evaluation method for damage level by visual inspection was proposed for damaged PC sleepers of Indian Railways. The proposed evaluation method was validated by comparing results on the material properties and structural performances. As a result, following conclusions are obtained.

1. The evaluation method in which the damage level is classified into “no damage”, “mild damage”, “moderate damage” and “severe damage” based on the width and number of cracks observed on the outer surface of OC sleepers was proposed.

2. The PC sleepers were cut and the inner crack patterns were investigated. As a result, the cracks do not infiltrate to the deeper region even in the case of “severe damage”. It indicates that the expansion caused by DEF and ASR is not uniform but it occurred only in the inner region. Thus, crack patterns observed on the outer surface are affected by not only amount but also spatial distribution of the expansion strain.

3. Compressive tests of core samples taken from the damaged PC sleepers were conducted. As a result, compressive strength and elastic modulus of the concrete are significantly reduced especially in the cases of “severe damage”. DEF
and ASR affect the mechanical properties in the material level.

(4) Loading tests of PC sleepers were conducted. As a result, both flexural and shear capacities were significantly decreased in the cases of “severe damage”. Especially, shear capacities of the “severe damage” cases closed to the minimum requirement level of Indian Railways, indicating that replacement of those sleepers is an urgent issue. In addition, it was confirmed that there is a good correlation between the structural performances and damage levels, indicating that the evaluation method for damage level proposed in this study is appropriate to determine the priority for the replacement.

(5) Mild and Moderate damaged sleepers can be continued to use by Indian Railways. When compared material strengths and capacity of the Mild and Moderate damaged sleepers to those of the undamaged new core samples, significant reductions were not noticed. However, considering the further damage progress in the future, it is recommended that damaged sleepers are replaced from severer ones.

(6) Due to the presence of high alkali content (5 %) in cement samples. Recommendation has been made to Indian Railways to maintain alkali content to less than 3 %.

References
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Singapore Concrete Institute
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Thailand Concrete Institute
Vietnam Concrete Institute