Fire resistance calculation of existing concrete structures using modified Tabulated Data Method

Duinkherjav Yagaanbuyant and Khishgee Radnaabazar

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Abstract: During its service operation, a gradual reduction of the material strength and the load bearing capacity of a concrete structure is expected and the same is also true for its fire resistance capacity. Although it is needed to estimate the remaining service life of the structure considering this reduction of fire resisting capacity for the design for the building renovation or expansion, the proper calculation guidance or the methodology still doesn’t exist in Mongolian national building code. This paper discusses the assessment of the actual fire resistance capacity of the existing pre-cast concrete industrial/residential buildings that were built during 1962-1999 in Ulaanbaatar, Mongolia. Modifications to the Tabulated data method are offered based on the findings of the study. The estimated values of the actual fire resistance limit of PC walls and slabs are proposed in tables and graphical forms that can be used in the calculation of the actual fire resistance of the existing buildings in use to determine renovation requirements.

Keywords: pre-cast concrete, reinforcement, fire severity, inspection, Tabulated data method.

1. Background

During its service life of a concrete structure, a gradual deterioration of structural performance including a reduction of its fire resistance performance is expected. When the deterioration is significant, the fire resistance capacity assessment of an existing structure becomes one of the requirements for further renovation and expansion. Roitman has studied the fire resistance of existing buildings in terms of effects of the reinforcement corrosion and the damages to concrete, but the service life of the structure has not been considered [1]. In this paper, the operational condition and the service life of structures are taken into account in the assessment of the fire resistance capacity of concrete structures for their further occupation.

In general the deterioration of the industrial buildings can be often of major concern. But from recent studies by the authors for the residential buildings, even though their operational conditions are not as severely reduced as those of the industrial buildings, the accumulated deterioration in the structures through years can have significant adverse effects on its fire resistance capacity and therefore should be included in the design of the building expansion and/or the renovation. In recent years such consideration of structural deterioration reducing its fire resisting capacity has become even more important due to the increased demand of building expansion through addition of stories and re-planning in urban areas in Ulaanbaatar. As the currently used Mongolian national building code doesn’t include a proper guideline to estimate this, the authors offer a modification to currently used Tabulated data method of the fire resistance based on the actual service condition and the service life of the existing concrete structures.

2. Fire resistance assessment methods

The fire resistance of reinforced concrete structures can be assessed experimentally through fire tests of the actual structures or analytically by engineering calculations [1~3]. A brief summary is given below.

2.1 Experimental Method

The results of the fire tests on the actual structures are most reliable and accurate for the assessment of the fire resistance capacity. Usually the fire tests are carried out using a fire chamber equipped with the load testing machine and the furnace. Calculation tables or tabulated data as developed from
the multiple test data to be used for the structural assessment based on the structure’s geometry, material type, and loading and boundary conditions. The disadvantages of the fire test are time, cost, energy consumption, and a need of special facility and specialists which make the analytical methods more favorable. The fire tests better be pursued for selective structural elements or assemblages, a new type of material or design of structural elements.

2.2 Analytical Methods

The analytical method of the fire resistance capacity determination of a structure can be carried out in two steps: thermal analysis and static analysis. The thermal analysis determines the developed temperature field across the cross section of the structure and the static analysis determines the load bearing capacity of the structure under loading taking into account the effects of the changes of the material properties due to elevated temperatures. In general, the fire resistance capacity estimation is carried out by incremental iteration method from the initial ignition of the fire through thermal and static analysis steps. In Fig. 1, the strength reduction over time is shown as an example of static analysis result. The fire resistance limit \( T_{w} \) is considered as the time period of the ultimate load bearing capacity reduction of the structure to its critical state \( F_{cr} \). In Fig. 2, a typical thermo-mechanical scheme of a RC structure during fire exposure is shown.

In BS EN 1992-2, three fire resistance assessment methods are suggested: (1) Tabulated data method, (2) Simplified calculation method, and (3) Advanced methods. It is also specified that the Tabulated data method can be used as an appendix to the national standards.

In Russian and Mongolian practice, the experimental method and above mentioned three analytical methods are both used for assessment of the structural fire resistance and, for this purpose, the guidelines and procedures of ref. [7–9] are followed. Currently, in Mongolia, the most frequently used method for the fire design of a new building is the Tabulated data method. But it cannot be used for the design for the building expansion, addition of stories in an existing building, replanning of the existing building. In the Tabulated data method, the resistance limit of reinforced concrete slabs and walls is given based on element thickness and concrete cover depth of reinforcement, but in the case of an existing building in use, actual values of concrete cover and thickness could be reduced due to defects in concrete cover and reinforcement corrosion. Therefore certain modifications are required in the use of the tabulated method for the actual fire resistance estimation of the existing buildings in use.

The Simplified calculation method is based on applying the strength reduction factors at elevated temperatures to simplified cross-sections for which the thermal gradient through the structural member is known or, alternatively, by utilizing a reduced cross-section whereby a damaged zone is ignored in the calculation and only the residual undamaged part is included in the calculation of the resistance [13]. The relevant parameters for the Simplified calculation method are taken from the information supplied on the material properties of the structural component.

Generally the Advanced method includes the numerical models for heat transfer from the fire into the structural member and the mechanical response models to determine the structural response of the member or members to the effects of both mechanical and thermal actions. General guidance is available in the European Standard on the use of the Advanced calculation models for the design of concrete structures. Unlike the Tabulated data
method, the Advanced method is suitable for any type of fire exposure. However, knowledge on the degradation of the material properties for the particular fire exposure condition is required.

2.3 Tabulated Data Method

From the guidelines of ref. [7,8], the fire resistance of reinforced concrete slabs and walls based on concrete type, element thickness, and concrete cover depth of reinforcement are shown in Table 1 and Table 2. It must be noted that, in the Tabulated data method, the resistance limit of reinforced concrete slabs and walls is given based on element thickness and concrete cover depth of reinforcement, but in the field, other effects such as defects in concrete cover, reinforcement corrosion, and concrete strength actually play important roles too [1,10].

3. Actual fire resistance of existing PC residential buildings

From the tabulated data, the design limit of fire resistance $T_{fr,s}$ can be determined. But for the existing building in use, the actual fire resistance of the structure $T_{fr,s}$ is lower than this design value due to the deteriorated state of the structures. The actual fire resistance of the structure $T_{fr,s}$ can be determined by Eq. (1), where $C_1$ is the reduction factor of the service life and $C_2$ is the reduction factor of the actual service conditions [1].

$$T_{fr,s} = T_{fr,s}^p \cdot C_1 \cdot C_2$$

The reduction factor $C_2$ that accounts the service conditions can be expanded further as $C_{2,1}, C_{2,2}, ..., C_{2,n}$ taking into account specific characteristics of the condition such as concrete strength, corrosion,
<table>
<thead>
<tr>
<th>Thickness $t_c$ and concrete cover depth of rebar $a$, mm</th>
<th>Fire resistance limit, hrs</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_c$, mm</td>
<td>0.5</td>
</tr>
<tr>
<td>$a$, mm</td>
<td>10</td>
</tr>
<tr>
<td>10</td>
<td>0.49</td>
</tr>
<tr>
<td>15</td>
<td>0.49</td>
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<tr>
<td>20</td>
<td>0.48</td>
</tr>
<tr>
<td>25</td>
<td>0.47</td>
</tr>
<tr>
<td>30</td>
<td>0.46</td>
</tr>
<tr>
<td>35</td>
<td>0.45</td>
</tr>
<tr>
<td>40</td>
<td>0.43</td>
</tr>
<tr>
<td>45</td>
<td>0.42</td>
</tr>
<tr>
<td>50</td>
<td>0.40</td>
</tr>
</tbody>
</table>

Table 4 – Fire resistance limit of reinforced concrete slabs based on service condition and service life (boundary condition, $l_{y}/l_{x} \geq 1.5$)

<table>
<thead>
<tr>
<th>Thickness $t_c$ and concrete cover depth of rebar $a$, mm</th>
<th>Fire resistance limit, hrs</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_c$, mm</td>
<td>0.25</td>
</tr>
<tr>
<td>$a$, mm</td>
<td>10</td>
</tr>
<tr>
<td>10</td>
<td>0.25</td>
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<tr>
<td>15</td>
<td>0.24</td>
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<tr>
<td>20</td>
<td>0.23</td>
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<tr>
<td>25</td>
<td>0.23</td>
</tr>
<tr>
<td>30</td>
<td>0.22</td>
</tr>
<tr>
<td>35</td>
<td>0.20</td>
</tr>
<tr>
<td>40</td>
<td>0.19</td>
</tr>
<tr>
<td>45</td>
<td>0.18</td>
</tr>
<tr>
<td>50</td>
<td>0.16</td>
</tr>
</tbody>
</table>

Table 5 – Fire resistance limit of reinforced concrete slabs based on service condition and service life (boundary condition, $l_{y}/l_{x} < 1.5$)

<table>
<thead>
<tr>
<th>Thickness $t_c$ and concrete cover depth of rebar $a$, mm</th>
<th>Fire resistance limit, hrs</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_c$, mm</td>
<td>0.25</td>
</tr>
<tr>
<td>$a$, mm</td>
<td>10</td>
</tr>
<tr>
<td>10</td>
<td>0.25</td>
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<tr>
<td>15</td>
<td>0.24</td>
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<tr>
<td>20</td>
<td>0.23</td>
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<td>25</td>
<td>0.23</td>
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<td>45</td>
<td>0.18</td>
</tr>
<tr>
<td>50</td>
<td>0.16</td>
</tr>
</tbody>
</table>
freeze-thaw action etc. as shown in Eq. (2). In this study, the concrete strength and the rebar corrosion reduction factors are taken in the example calculation.

\[ \tau_{f,p}^2 = \tau_{f,p}^{n1} \cdot C_1 \cdot C_{2,1} \cdot C_{2,2} \cdot \ldots \cdot C_{2,n} \]  

(2)

To determine the reduction factor of the service life \( C_l \), the damage state of structures should be determined first through a detailed inspection of each structure in terms of important characteristics of structure for its technical serviceability such as section dimension, strength, and cracking etc.

A total of 1,077 pre-cast concrete buildings had been built in 1962-1999 in ten districts of Ulaanbaatar. Buildings have 5 to 12 stories with the structural pre-cast reinforced concrete wall and slab panel system without framing. The inspections of more than 60 buildings from these pre-cast concrete buildings in 2001-2015 to assess the structural state of the buildings have been carried out by the authors. The structural deterioration in these buildings was mainly caused by damaged concrete cover, reinforcement corrosion, and multiple freeze-thaw action resulting in the reduced fire resistance. In this paper, test results of 30 buildings located in 10th Micro-district of Ulaanbaatar have been used. From the study, it is observed that external walls, basement floor slabs and walls, and roof slabs are more damaged than the other structural elements due to the reinforcement corruptions. These damage conditions not only deteriorate the load bearing capacity of the structures but also the fire resistance.

The damages of each structural element are registered and a damage degree \( \omega \) is determined as a ratio of the number of the damaged element to the total number of elements in each structural type. From the data, the correlation between the damage degree \( \omega \) and the service life of the structure \( t \) is observed and the following relations are established through regression equations:

<table>
<thead>
<tr>
<th>Structural Element</th>
<th>( \omega ) Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>External wall</td>
<td>( \omega = 0.1209 + 0.0095t )</td>
</tr>
<tr>
<td>Internal wall</td>
<td>( \omega = 0.0829 + 0.0019t + 0.00001r^2 )</td>
</tr>
<tr>
<td>Roof slab</td>
<td>( \omega = 0.0862 + 0.0032t + 0.0001t^2 )</td>
</tr>
<tr>
<td>Floor slab</td>
<td>( \omega = 0.0872 + 0.0004t + 0.0001t^2 )</td>
</tr>
</tbody>
</table>

Based on Eq. (4), Eq. (6), and Table 1 and Table 2, the actual reduction of fire resistance of PC slabs and walls can be determined. Results are shown in Figs. 3 through 5 and Tables 3 through 5.

4. Examples of calculation

4.1 Calculation of fire resistance of reinforced concrete slab of 5-story residential building

Data: Basement floor reinforcement concrete slab, 5-story residential building located in the 15th horoo, Ulaanbaatar. Slab dimension \( l = 5.4 \text{ m}; b = 3.6 \text{ m}; h = 0.12 \text{ m}. \) Number of tensile rebars – 18 Class Bp-I, \( d_s = 5 \text{ mm}. \) Concrete class B15, concrete cover depth from the rebar centroid \( \delta = 0.02 \text{ m}, l/l_x = 5.4/3.6 = 1.5 \)

4-1-a Geometric characteristics required for the calculation
- By load bearing reduction “\( R \)”: \( a = \delta + 0.5ds = 0.02 + 0.0025 = 0.0225 \text{ m} \)
- By insulation capacity “\( I \)”: \( h = 0.12 \text{ m} \)

4-1-b Design fire resistance value for floor slab, 5 story pre-cast panel reinforced concrete building
- By load bearing reduction “\( R \)”, Table 1: if \( a = 0.015 \text{ m} \) then \( (\tau_{f,p}^{n1}) = 0.5 \text{ hr}; \) if \( a = 0.025 \text{ m} \) then \( (\tau_{f,p}^{n1}) = 1.0 \text{ hr}; \) if \( a = 0.0225 \text{ m} \) then \( (\tau_{f,p}^{n1}) = 0.875 \text{ hr} = 52.5 \text{ min} \)
- By insulation capacity “\( I \)”, Table 8 [7]: \( h = 0.12 \text{ m}, (I) = 2 \text{ hr} = 120 \text{ min} \)

4-1-c Actual fire resistance based on service condition and life
- Service life = 34 years
- By load bearing reduction “\( R \)”, Table 3 or Fig 2: if \( a = 0.015 \text{ m} \) then \( (R) = 0.41 \text{ hr}; \) if \( a = 0.025 \text{ m} \) then \( (R) = 0.91 \text{ hr}; \) if \( a = 0.0225 \text{ m} \) then \( (\tau_{f,p}^{n1}) = 0.785 \text{ hr} = 47.1 \text{ min} \)
- Actual fire resistance based on service condition and life: \( \tau_{f,p}^{n1} = 47.1-0.92-0.91 = 39.4 \text{ min} \)

It is calculated that the actual fire resistance of the floor slab is reduced by 25 percent based on the service condition and service life.

4.2 Calculation of fire resistance of reinforced concrete wall of 5-story residential building

Data: Reinforcement concrete partition wall, \( l = 5.4 \text{ m}; b = 2.8 \text{ m}; h = 0.14 \text{ m}. \) Number of tensile rebars – 18 Class A400, \( d_s = 12 \text{ mm}. \) Concrete class B15, concrete cover depth \( \delta = 0.015 \text{ m} \)

4-2-a Wall shall be exposed to the fire from one side. It is very rare for walls to experience fire from both sides. If the wall is exposed to fire from both sides then no deflection takes place and wall still be working under compression [7].

4-2-b Design fire resistance limit of the wall is taken from Table 2.
4-2-c Calculation of geometric characteristics required for the calculation
- By load bearing capacity reduction “R”: \( a = \delta + 0.5d_r = 0.015 + 0.006 = 0.021 \) m
- By insulation capacity “T”: \( h = 0.14 \) m

4-2-d Calculation of fire resistance of reinforced concrete partition wall, 5-story pre-cast reinforced concrete panel building
- By load bearing capacity reduction “R”: Table 2 fire resistance limit: if \( a = 0.02 \) m then \( \tau_{f,r} = 1.5 \) hr; if \( a = 0.03 \) m then \( \tau_{f,r} = 2.0 \) hr; if \( a = 0.021 \) m then \( \tau_{f,r} = 1.55 \) hr = 93 min
- By insulation capacity “T”: Table 2, if \( h = 0.14 \) m, then \( (I)= 1.5 \) hr = 90 min

4-2-e Calculation of the actual fire resistance of reinforced concrete partition wall based on service condition and service life
- Service life = 34 years
- By load bearing capacity reduction “R”: Table 4 [7] if \( a = 0.02 \) m then \( \tau_{f,r} = 1.1 \) hr; if \( a = 0.03 \) m then \( \tau_{f,r} = 1.5 \) hr; if \( a = 0.021 \) m then \( \tau_{f,r} = 1.14 \) hr = 68.4 min. Considering reduction of service condition (reduction of concrete strength and rebar corrosion), \( \tau_{f,r} = 68.4 \cdot 0.92-0.91 = 57.2 \) min

It is calculated that the actual fire resistance of the partition wall is reduced by 38 percent from the design value based on the service condition and service life.

5. Conclusions

During renovation and expansion of the existing buildings in use, not only the structural load bearing capacity, and stability requirements but also fire resistance requirements should be considered and satisfied to provide safe living condition to occupants. The Tabulated data method is the most widely used method for practicing engineers today but its application for the calculation of the fire resistance of structures of the existing buildings needs certain modification that accounts the deteriorated state of the structures due to the actual service conditions. From this study, up to 25 to 38 percent reduction in the fire resistance of structures depending on the actual service condition and the service life is observed. Based on the findings of the field study of 30 pre-cast concrete buildings, the empirical regression equations that establish relations between damage degree of structural members and their service life are proposed. Based on these equations, the estimated values of the actual fire resistance limit of PC walls and slabs are suggested in tables and graphical forms that can be used in the calculation of the actual fire resistance of structures of the existing buildings in use for their further renovation. Example calculations are also included.

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Effectiveness of recycled nylon fibers as reinforcing material in mortar

ORASUTTHIKUL Shanya*; UNNO Daiki; YOKOTA Hiroshi; and HASHIMOTO Katsufumi

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Abstract: Disposing of waste fishing nets has been a major issue in the sea environment. Although the storage of such nets has not caused a serious safety hazard to date, it is important to find suitable recycling solutions. In this study, the authors investigate the utilization of recycled waste fishing nets in fiber-reinforced mortar and compare the mechanical properties of such mortar made with recycled waste fishing nets to those of mortar made with PVA (polyvinyl alcohol) short fibers. Two types of recycled nylon fiber were investigated: straight fiber and fiber with a knot at each end. The straight recycled nylon fibers were obtained by manually cutting waste fishing nets to the lengths of 20 mm, 30 mm, and 40 mm, and adding them to mortar at the volume ratios of 1.0%, 1.5%, and 2.0%. The 40-mm-long knotted fibers were added to mortar at the volume ratios of 0.5%, 0.75%, and 1.0%. The mechanical test results showed improvements in first-crack strength, toughness, and ductility for mortar reinforced with recycled nylon fibers. The addition of recycled nylon fibers improves first-crack strength more than that PVA fibers do. However, the compressive strength decreases with increase in fiber fraction, and decreases with increase in fiber aspect ratio.

Keywords: recycled nylon fiber, waste fishing net, recycled materials, fiber reinforced mortar.

1. Introduction

In recent decades, the world has been suffering from the dumping of wastes, especially plastics left in seas and oceans. Waste fishing nets account for some of these wastes: 640,000 tons of fishing nets are disposed of in the ocean annually [1]. As the nets become totally entangled, separating them for disposal is impractical. These nets can be harmful to marine life, such as turtles, seals, and other marine mammals, which can become entangled and suffer injury or drowning [2]. In addition, the marine food web could be disrupted. As abandoned nets and plastic garbage tend to gather at or near the surface of seas and oceans, they keep sunlight from reaching small creatures such as planktons and algae. Therefore, the animals that feed on these small creatures are also directly affected.

Most fishing nets are made of nylon and are non-biodegradable. Even though their storage is not hazardous, it is very important to find suitable ways of recycling these nets. Spadea et al. [3] investigated the use of waste fishing nets as recycled nylon fiber for reinforced cement mortar. They found that adding the fibers to mortar significantly improves mechanical properties, such as increased first-crack strength (i.e., the modulus of rupture, MOR), toughness, and ductility. Two decades later, many researchers are studying the use of fiber reinforced mortar (FRM) as a way of achieving higher MOR, fracture toughness, impact resistance, and of controlling shrinkage. The use of synthetic fibers such as polyvinyl alcohol (PVA) and polypropylene was found to be successful in significantly improving the mechanical properties of the base material [4-7].

In recent years, many researchers have become interested in using recycled materials [8-15]. Not only are they interested in improving the mechanical properties, but they are also concerned with achieving favorable environmental outcomes and realizing economical products. Additionally, durability under alkaline conditions is very important for the fibers, if they are to enhance mortar composites. There is some evidence in the literature that recycled nylon fibers have excellent alkali resistance [3]. However, a test by Ochi et al. [16] found that the tensile strength of PVA fibers decreases by 44% from alkali exposure.

This study aims to investigate the effectiveness of recycled nylon (R-nylon) fibers retrieved from...
waste fishing nets to improve certain mechanical properties of mortar. The authors address the preparation of R-nylon FRM and the identification of its mechanical properties (e.g., compressive strength, MOR, toughness, and residual strength) in comparison to those of PVA FRM. The effects of using different fiber volume ratios and lengths are analyzed.

2. Outline of Experimental Test

The waste fishing nets used in this study were collected by fishermen in Hokkaido, Japan. The fibers were manually cut in three different lengths of 20 mm, 30 mm, and 40 mm, and were mixed in mortar at several different volume fractions.

The volume fractions of the knotted R-nylon fibers (1.0%, 1.5% and 2.0%) were lower than those of the straight fibers (0.5%, 0.75%, and 1.0%), because knotted R-nylon fibers tend to form tenacious clumps. The PVA fibers were included in the mix at 1.0 vol.% and 1.5 vol.%. In the case of the recycled nylon fiber, the authors carried out uniaxial tensile tests according to ASTM C1557-03 standard [17] in order to determine the tensile strength and Young’s modulus. The fiber types and the mechanical properties are given in Fig. 1 and Table 1, respectively. Before the fibers were added, cement and sand were mixed in a small mixer. Then, the fibers were gently added to prevent the formation of fiber balls, and all the dry components were mixed by hand in order to obtain a uniform distribution of fibers. Water was gradually added, and a mixing machine was used at low speed for 2 minutes until a homogeneous mixture was achieved. The mortar was cast in prism molds of 40 mm × 40 mm × 160 mm for the bending test and in cylinders of 50 mm diameter × 100 mm in height for the compressive strength test. The specimens were cured in water at 20˚C for 28 days and then tested. For the mortar mixes, various combinations of fiber volume and fiber aspect ratio were used as summarized in Table 2. The unreinforced mortar is notated as UR, and the FRM specimens are notated as SRny (straight recycled nylon fiber), KRny (knotted recycled nylon fiber), and PVA (PVA fiber), followed by “fiber length-volume fraction.”

3. Results and Discussions

3.1 Flowability

The flowability test was conducted in compliance with ASTM C 1437 [18]. The flow diameters were measured and are listed in Table 2. It can be seen that flowability tends to decrease with increase in fiber length and amount. A comparison of KRny and SRny40 at 1.0 vol.%, shows that the FRMs mixed with knotted fibers have greater flow diameters due to the balling of fibers, which leads to fiber–mortar separation. Moreover, a comparison of PVA and SRny30, which have similar aspect ratios, shows fresh FRM containing PVA fibers to have a smaller flow diameter than SRny30 FRMs has, at a

<table>
<thead>
<tr>
<th>Fiber type</th>
<th>Diameter (μm)</th>
<th>Tensile strength (MPa)</th>
<th>Young’s modulus (GPa)</th>
<th>Density (g/cm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R-nylon fiber</td>
<td>350</td>
<td>440</td>
<td>3.0</td>
<td>1.13</td>
</tr>
<tr>
<td>PVA</td>
<td>200</td>
<td>975</td>
<td>27</td>
<td>1.30</td>
</tr>
</tbody>
</table>

Table 1 – Mechanical properties of fibers

Fig. 1 – Types of fibers
Table 2 – Test results of fresh mortar properties

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>Fiber fraction by volume (%)</th>
<th>Fiber length, L (mm)</th>
<th>Diameter, D (mm)</th>
<th>Aspect ratio (L/D)</th>
<th>Flow diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UR</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>262</td>
</tr>
<tr>
<td>KRny-0.5</td>
<td>0.5</td>
<td>40</td>
<td>0.35</td>
<td>114</td>
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<tr>
<td>KRny-0.75</td>
<td>0.75</td>
<td>40</td>
<td>0.35</td>
<td>114</td>
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</tr>
<tr>
<td>KRny-1.0</td>
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<td>40</td>
<td>0.35</td>
<td>114</td>
<td>227</td>
</tr>
<tr>
<td>SRny20-1.0</td>
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<td>20</td>
<td>0.35</td>
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<td>0.35</td>
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<tr>
<td>SRny20-2.0</td>
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<td>0.35</td>
<td>57</td>
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<tr>
<td>SRny30-1.0</td>
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<td>0.35</td>
<td>86</td>
<td>216</td>
</tr>
<tr>
<td>SRny30-1.5</td>
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<tr>
<td>SRny30-2.0</td>
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<td>0.35</td>
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</tr>
<tr>
<td>SRny40-1.0</td>
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</tr>
<tr>
<td>SRny40-1.5</td>
<td>1.5</td>
<td>40</td>
<td>0.35</td>
<td>114</td>
<td>207</td>
</tr>
<tr>
<td>SRny40-2.0</td>
<td>2.0</td>
<td>40</td>
<td>0.35</td>
<td>114</td>
<td>183</td>
</tr>
<tr>
<td>PVA-1.0</td>
<td>1.0</td>
<td>18</td>
<td>0.2</td>
<td>90</td>
<td>213</td>
</tr>
<tr>
<td>PVA-1.5</td>
<td>1.5</td>
<td>18</td>
<td>0.2</td>
<td>90</td>
<td>169</td>
</tr>
</tbody>
</table>

Given volume fraction, due to greater number of fibers.

3.2 Compressive strength

The compressive strengths of the FRMs are presented in Table 3. The compressive test was conducted in compliance with ASTM C 39 [19]. Table 3 indicates that the addition of the R-nylon fibers causes a reduction in compressive strength of the examined mortars up to 48%. Moreover, it can be clearly seen that the compressive strength decreases with decrease in fiber length, and decreases with increase in R-nylon fiber amount. An explanation is that nylon fibers have a much lower modulus of elasticity than mortar has, and therefore, the inclusion of fibers creates voids in the mortar [20]. Also, the addition of fibers, especially long fibers, leads to increases in the volume of the interfacial transition zone (ITZ), which results in the reduction of strength and stiffness of FRM [21]. This might suggest that the lower compressive strength of the KRny FRM is the result of a greater reduction in the modulus of elasticity of the FRM from the inclusion of knots as shown in Table 3. When the specimens are subjected to compressive load, lateral tensile strain in the mortar occurs due to the Poisson’s effect. As the load increases, the longer fibers play a more important role in the mortar’s lateral tensile strength than the shorter fibers play. These longer fibers postpone crack enlargement by increasing the mortar’s lateral tensile strength [13]. In the case of the PVA fibers, the addition of fibers results in a decrease in compressive strength of the mortar, especially at greater amounts of addition. An explanation is that the modulus of elasticity of PVA fibers and mortar are almost the same, but the fibers are much less dense. Therefore, the mortar was filled with a high amount of fibers, which resulted in reductions in the elastic modulus of the mortar [22].

3.3 Flexural strength

After 28 days of curing, flexural strength tests were conducted in accordance with ASTM C 293 [23]. The authors performed three-point bending tests, and the peak load (Pcr), the first-crack strength (i.e., MOR), and other results are listed in Table 3. The results show that the addition of KR-nylon, SR-nylon, and PVA fibers increases the MOR by up to 22%, 41%, and 22%, respectively. Spada et al. [3] and Orasuthikul et al. [24] proposed that using R-nylon fibers from waste fishing nets is very effective at reinforcing mortar, with the MOR found to be improved by up to 35% and 42% in the respective reports.

The first-crack strength can be calculated as follows:

\[ M = 3P_{cr}l/2bd^2 \]  

where, \( M \) = modulus of rupture (MOR) or first-crack strength; \( P_{cr} \) = maximum applied load;
Table 3 – Test results of compressive and flexural strength at 28 days

<table>
<thead>
<tr>
<th>Specimen type</th>
<th>Compressive strength test</th>
<th>Flexural strength test</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No. of specimens</td>
<td>$f'_c$ (MPa)</td>
</tr>
<tr>
<td>UR</td>
<td>3</td>
<td>65.67</td>
</tr>
<tr>
<td>KRny-0.5</td>
<td>3</td>
<td>56.61</td>
</tr>
<tr>
<td>KRny-0.75</td>
<td>3</td>
<td>55.96</td>
</tr>
<tr>
<td>KRny-1.0</td>
<td>3</td>
<td>36.77</td>
</tr>
<tr>
<td>SRny20-1.0</td>
<td>3</td>
<td>52.56</td>
</tr>
<tr>
<td>SRny20-1.5</td>
<td>3</td>
<td>48.66</td>
</tr>
<tr>
<td>SRny20-2.0</td>
<td>3</td>
<td>34.14</td>
</tr>
<tr>
<td>SRny30-1.0</td>
<td>3</td>
<td>53.61</td>
</tr>
<tr>
<td>SRny30-1.5</td>
<td>3</td>
<td>46.84</td>
</tr>
<tr>
<td>SRny30-2.0</td>
<td>3</td>
<td>35.06</td>
</tr>
<tr>
<td>SRny40-1.0</td>
<td>3</td>
<td>55.54</td>
</tr>
<tr>
<td>SRny40-1.5</td>
<td>3</td>
<td>48.24</td>
</tr>
<tr>
<td>SRny40-2.0</td>
<td>3</td>
<td>35.27</td>
</tr>
<tr>
<td>PVA-1.0</td>
<td>3</td>
<td>61.70</td>
</tr>
<tr>
<td>PVA-1.5</td>
<td>3</td>
<td>59.57</td>
</tr>
</tbody>
</table>

Note: $f'_c$ - compressive strength, SD - standard deviation, CV - coefficient of variation, %Δ$f'_c$ - percent difference in compressive strength between control specimens (UR) and FRMs, Δ$R$ - percent difference in modulus of rupture between control specimens (UR) and FRMs.

$b$ = specimen width; $d$ = specimen depth; and $l$ = span length.

From Fig. 2(a) to (e), it can be clearly seen that R-nylon FRM shows a significant reduction in load after the first crack. This is because the R-nylon fibers used in this study have a smooth surface, so the friction interlocking between the fiber surface and the mortar is poor, which results in low frictional resistance to slippage. When PVA FRMs were subjected to external load, the PVA fibers tended to rupture before the fiber and the matrix debonded from each other. This caused abrupt reductions in the post-peak load of PVA FRMs to nearly the same or slightly lower than that of SRny40 FRM with a 2.0% fiber fraction when mid-span deflection was increasing. After the bending test, the fiber surface was examined to analyze the frictional resistance and interfacial bond between the fiber and the matrix. The R-nylon surface seen in Fig. 3(a) shows the fiber had no serious change in scratching as a result of low frictional resistance between the fiber and the matrix. For the PVA fiber, due to the stronger bond between the fiber and the matrix, some cement paste is seen on the fiber surface as shown in Fig. 3(b).

3.4 Toughness and residual strength

Toughness is an important mechanical characteristic of FRM. In accordance with ASTM C 1018 [25], toughness indices I5, I10, and I20 were obtained by dividing the area under the load-deflection curve up to 3.0, 5.5, and 10.5 times the first-crack deflection, respectively, by the area under the curve up to the first-crack deflection (see Fig. 4). Table 4 summarizes the toughness indices and residual strength factors of FRMs at 28 days. The addition of the fibers to the mortar appears to afford outstanding improvements in toughness, especially for higher fiber fractions and greater fiber lengths (see Fig. 5). The toughness and residual strength of FRMs are dependent on fiber characteristics, such as geometric shape, tensile strength, and modulus of elasticity, as well as on the bond strength between the fiber and the surrounding mortar. SRny30 fiber and PVA fiber have similar aspect ratios, but SRny30 FRMs have lower toughness indices and residual strength factors than PVA FRMs have. However, the fracture toughnesses of
the SRny40 FRMs with a 2.0% fiber fraction (SRny40-2.0) fall between those of the PVA FRMs with a 1.0% fiber fraction (PVA-1.0) and the PVA FRMs with a 1.5% fiber fraction (PVA-1.5). In this study, the fiber fractions of the KRny fiber are low, and therefore the toughness and residual strength of the FRMs are not effectively improved. The PVA fibers generally have a stronger bond to the surrounding cementitious matrix because it is hydrophilic. Redon et al. [26] found that small-diameter PVA fibers ruptured before achieving their full pullout length. Therefore, when the PVA FRMs were loaded, the PVA fibers broke from tension rather than from pullout.

4. Conclusions

In this study, the effectiveness and potential of using waste fishing nets as recycled nylon fiber to
reinforce mortar were experimentally tested and discussed. Compressive and three-point bending tests were performed to investigate compressive strength, peak load, modulus of rupture, toughness indices, and residual strength factors. This study found the effects of adding R-nylon fibers to mortar to be as follows.

(1) The addition of R-nylon short fibers recycled from waste fishing nets to mortar improves the mechanical properties of the mortar, except for compressive strength, which decreases as a result of that addition.

(2) The addition of R-nylon fibers, whether straight or knotted, results in a reduction in mortar workability. Mortar flowability tends to decrease with increase in the fiber fraction. Mixes that incorporate longer fibers demonstrate smaller flow diameters than those with shorter fibers.

(3) The addition of R-nylon fibers to mortar affords a more ductile mode of failure than the addition of plain mortar affords. Post-cracking ductility is enhanced, which results in an increase in the load carrying capacity of FRM in bending. The post-peak loads are observed to be higher with higher fiber fractions and longer fibers.

(4) Mechanical analyses show that the addition of fibers leads to reductions in compressive strength of 14-44% for KR-nylon fiber and 20-48% for SR-nylon fiber. The MOR increased by up to 22% for KRny FRM and by up to 41% for SRny FRM.

(5) The addition of PVA fibers to mortar causes a greater reduction in flow diameter than the addition of R-nylon fibers at a similar fiber aspect ratio and the same fiber fraction.

(6) Post-peak load, toughness indices, and residual strength factors differ significantly according to fiber characteristics such as geometric shape, tensile strength, and modulus of elasticity, as well as according to the bond between the fiber and the cementitious matrix. As evidenced by
the flexural test, R-nylon fiber shows a significantly lower post-peak load than PVA fiber affords, due to the smooth surface of R-nylon fiber. Despite the stronger chemical bond between the PVA fiber and the matrix, the PVA fibers tended to rupture instead of pulling out. This caused the post-peak load of the PVA FRM to dramatically drop after the peak, due to breaking of the fiber. However, R-nylon fiber shows greater MOR improvement than PVA fiber affords.

(7) For every type of fiber, its addition to the mortar results in a decrease in compressive strength, especially the addition of fibers with a low modulus of elasticity, such as R-nylon fiber.

It must be noted, however, that the R-nylon FRMs analyzed in this study have been proven beneficial in terms of mechanical properties as PVA fiber, even if a higher amount of fibers may be required to match the performance. Moreover, the use of waste fishing nets as recycled nylon fibers results in an environmental benefit.

Acknowledgements
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References


Relative evaluation of performance of limestone calcined clay cement compared with Portland pozzolana cement

Ashok Kumar Tiwari* and Subrato Chowdhury

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Abstract: Cement is most widely used binder to produce concrete and the most common construction material today. Though concrete is a material with the lowest greenhouse emission, cement has the highest. With the carbon footprint of cement accounting for over 7% of total world emissions, it becomes single most important material of environmental concern around the world. This concern has led to a search for lower carbon emitting binders and use of blended cements, incorporating large number of natural and industrial by-products. This paper describes the performance of a composite cement binder consisting of calcined clay, limestone, and Portland cement clinker as compared to a traditionally used fly ash based Portland pozzolana cement. This study reports behavior of the two cement binders with respect to strength development, hydration, porosity of hydrated pastes, normal consistency, and admixture response with ageing. This study finds that, though the clay based cement attains higher early age strength, the later age strength in mortar is lower as compared to commercial fly ash based cement. Further the clay based cement has higher water demand, but lower porosity compared to composite cement binder.

Keywords: binder, calcined clay, limestone, microstructure, hydration, porosity, water demand.

1. Introduction

Concrete is most widely used construction material in the world today with over 25 billion tons placed every year [1]. It is made from graded aggregate system, cemented together by a binder, normally called Portland cement. Often this concrete also includes various industrial by-products, which may have either pozzolanic or self-cementing properties. Modern concrete also has superplasticizers to make the concrete workable and pumpable for various applications. In spite of various developments, cement remains the only binder in concrete. The cement is highly energy intensive to produce and is responsible for about 7% of greenhouse emissions in the world today [2]. Even though carbon footprint of concrete is lowest among building materials used in construction (see Fig. 1), the same is highest due to sheer volume of concrete being used today. Researchers in academia and industry have been working on reducing the carbon footprints of cement and have been successful in reducing the same by about 20% in last one decade as reported in World Cement Sustainability Initiative Report published in 2012.

Cement production has been progressively increasing and currently over 4 billion tonnes of cement is being produced annually across the world. Realising the problems of greenhouse gases, the cement industry has undergone a lot of changes. The changes are in process engineering as well as in variety of cements being produced. Today, utilization of blended cements is usually preferred due to their economic and technical benefits and indirect advantages such as lower level of CO₂ emissions by reducing clinker production in plants. While the emerging family of cementitious materials has been expanding to a larger number, namely fly ash, silica fume, calcined clay, metakaolin etc., materials like fly ash (FA) and ground granulated blast furnace slag (GGBS) are being widely used for cement production.

The primary objective of usage of wide range of cementitious materials, both natural and artificial is to reduce the CO₂ footprint and progressively increase usage of non-bio degradable industrial waste. These major industrial wastes are fly ash and GGBS. Other minor industrial waste could be used as cementitious material. Introduction of calcined clay pozzolana as cementitious material has been
CO₂ discharge in production process of construction materials reported long back elsewhere globally. In India, it began in late 1970s only, when BIS standard for calcined clay became available for manufacturing of Portland pozzolana cement [5]. Many public actions report that performance of mortar and concrete composed with calcined clay closely compares with mortar/concrete made with fly ash and GGBS. The calcined clay is the potentially rich cementitious material and its usage would greatly contribute in reducing carbon foot print.

The objective of this study is to evaluate the performance of limestone calcined clay cement (LC3) and commercially available Portland pozzolana cement (PPC) towards understanding the products in greater details. The blended cements have been in use widely for more than two decades in reducing the CO₂ footprint in relation to conventionally used ordinary Portland cements. Most of the countries today produce and use blended cement, either binary or ternary blended for various construction applications [3-7]. Accordingly the specification of cement are becoming performance oriented rather than current one of prescription oriented [8]. In India, due to large availability of fly ash and to some extent GGBS, over 75% of cement used is blended cement. The trend is similar across the world and even in ordinary Portland cements, performance improvers or minor additional compounds such as GGBS, limestone, and fly ash are replacing clinkers to the extent of up to 5% [9]. With these efforts, clinker conversion factor, which is a measure of how much cement is produced per unit clinker, has gone to 1.6 from traditional level of 1.03 reducing carbon foot prints by over 50%.

In countries, where fly ash is not abundantly available, construction industry has been using limestone powder as clinker substitute. Limestone replacement into Portland cement has been widely studied for several years [9-11]. Limestone not only works as micro filler, but also takes part in hydration process of clinker, improving workability, strength, and durability [12]. The limestone can also be used in ternary blends in combination with fly ash, calcined clay and other pozzolana.

This study compares the behavior of two blended cements, one produced at small cement production unit for this experimental study, the other one being commercially available fly ash based Portland pozzolana cement. The tests were conducted in accordance with Indian standards [13-14].

2. Production of LC3 Cement

A limestone calcined clay cement (LC3) consisting of limestone, calcined clay, and portland cement clinker was used. Cement has 15% limestone, 31% calcined clay, and about 50% portland cement clinker with remainder being gypsum. Properties of the raw materials are given in Table 1. Limestone and calcined clay were ground separately and intermixed with ground clinker. A commercially available Portland pozzolana cement (PPC), made from fly ash conforming to Indian Standard [5] were used for bench marking.

The clay was calcined in a rotary kiln at a temperature of 900 degree Celsius for optimum calcination. A weight loss of 0.3% was observed and lime reactivity of calcined clay was observed to be 7.8 MPa, which was significantly higher than the requirements of pozzolanic materials and certainly better than those of fly ash.

Table 1 – Oxide composition of raw materials

<table>
<thead>
<tr>
<th>Element</th>
<th>Clinker, %</th>
<th>Limestone, %</th>
<th>Clay, %</th>
<th>Gypsum, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>21.1</td>
<td>11.02</td>
<td>54.47</td>
<td>2.77</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>4.32</td>
<td>1.55</td>
<td>4.93</td>
<td>0.36</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>4.65</td>
<td>2.53</td>
<td>27.29</td>
<td>0.62</td>
</tr>
<tr>
<td>CaO</td>
<td>65.16</td>
<td>44.24</td>
<td>0.06</td>
<td>32.62</td>
</tr>
<tr>
<td>MgO</td>
<td>2.13</td>
<td>1.96</td>
<td>0.13</td>
<td>1.20</td>
</tr>
<tr>
<td>SO₃</td>
<td>0.77</td>
<td>0</td>
<td>0.01</td>
<td>38.75</td>
</tr>
<tr>
<td>Na₂O</td>
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<td>0.5</td>
<td>0.12</td>
<td>0.06</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.20</td>
<td>0.28</td>
<td>0.25</td>
<td>0.04</td>
</tr>
<tr>
<td>LOI</td>
<td>0.96</td>
<td>36.96</td>
<td>10.28</td>
<td>23.02</td>
</tr>
</tbody>
</table>

3. Materials and Test Methods

3.1 Cement

Cement, after proper sampling, was tested as per Indian Standards [13-14] for the following items:

- fineness, by Blaine test apparatus;
- full physical properties, including particle size distribution and retentions on different sieve
sizes like 90 micron (R90), 75 micron (R75), and 45 micron (R45);
- full chemical analysis including insoluble residue (IR), sulphate (SO₃), loss on ignition (LOI), magnesium oxide (MgO), total & soluble alkalis;
- hydration study using calorimeter (ICP) at w/c of 0.4 at 24, 72, 144, and 672 hours; and
- hydration study of LC3 by scanning electron microscopy (SEM).

3.2 Cement paste
- Workability retention study using marsh cone and mini-slump
  - At w/c of 0.5 % by wt. without admixture and at 0.4 with admixture
  - Fluorescent microscopy on cement paste porosity
  - Mercury intrusion porosimeter Quanta chrome MIP with high pressure 60,000 psi system for porosity of paste

3.3 Cement mortar
- Cement mortar with standard stand, ratio 1:3 for compressive strength

Cement pastes were tested for admixture compatibility using a marsh cone and retention was measured using a mini slump. The mini-slump test which was originally developed by Kantro [15] and later modified by Zhor & Bremner [16], measures the consistency of cement paste and is commonly used for evaluating admixture-cement response for flow and retention across the world. The mini-slump cone is a small version of the slump cone. The mini-slump cone is placed in the centre of a piece of plane rigid and non-absorbent surface / table. The paste was prepared at a water-binder ratio (w/b) of 0.55 and retention of flow was measured up to 120 minutes.

4. Results and Discussions

A limestone calcined clay cement (LC3) consisting of limestone, calcined clay, and Portland cement clinker was used. Limestone and calcined clay were ground separately and intermixed with cement. A commercially available Portland pozzolana cement (PPC), fly ash based, conforming to Indian Standards [5] was used for bench marking. Properties of cements, tested as per relevant Indian standards [13-14] are given in Tables 2 and 4.

4.1 Cement properties
The chemical composition of cements is given in Table 2 whereas the physical properties of cement are listed in Tables 3 and 4. The LOI of LC3 are much higher than the PPC. The higher LOI is primarily contributed from the limestone addition. The requisite variety of limestone for the purpose of usage in LC3 are having intrinsic LOI in the range of 32–37% typically whereas typical LOI of fly ash is in the range of 0-6 %. SO₃ range is 1.87–2.83%. Crystals of tri-calcium silicate present in commercially available cement are mixed with impurities such as alkalis, sulphates, phosphorous, and host of trace minerals. The hydration process of alite or tricalcium silicate initiates in presence of alkali sulphate or alkaline sulphate environment. Alite activates when sulphate concentration (SO₃) of the solid solution is close to 2% [17]. However PPC is having SO₃ concentration of 2.83% owing to possible usage of high sulphur bearing clinker. Insoluble residue (IR) for all the cement is ranging 22.68–24.23%. The contributory factors for IR are the percentage of fly ash addition and calcined clay.

The Blaine fineness of LC3 is significantly higher in comparison with PPC, though mean particle size is similar (see Tables 3 and 4). The Blaine fineness of the cement is influenced by the material characteristics and the comminution system adopted. Calcined clay and the limestone are vastly different from clinker and fly ash as far as the material hardness and grinding efficiency is concerned. In general, the mean size of cements was similar, though d50 of LC3 was significantly lower as compared to PPC. The normal consistency of LC3 and PPC are in a close range of 30–31% despite the fact that LC3 Blain fineness level is more than the double of PPC.

Table 2 – Composition of cement

<table>
<thead>
<tr>
<th>Cement</th>
<th>LOI</th>
<th>IR</th>
<th>SO₃</th>
</tr>
</thead>
<tbody>
<tr>
<td>LC3</td>
<td>7.18</td>
<td>22.68</td>
<td>2.15</td>
</tr>
<tr>
<td>PPC</td>
<td>2.78</td>
<td>24.23</td>
<td>2.83</td>
</tr>
</tbody>
</table>

Table 3 – Particle size distribution of cements

<table>
<thead>
<tr>
<th>Sample</th>
<th>d₁₀, µm</th>
<th>d₅₀, µm</th>
<th>d₉₀, µm</th>
<th>Mean size, µm</th>
</tr>
</thead>
<tbody>
<tr>
<td>LC3</td>
<td>1.26</td>
<td>13.11</td>
<td>57.48</td>
<td>22.25</td>
</tr>
<tr>
<td>PPC</td>
<td>1.70</td>
<td>18.09</td>
<td>51.80</td>
<td>23.04</td>
</tr>
</tbody>
</table>

Table 4 – Physical properties of cements

<table>
<thead>
<tr>
<th>Cement</th>
<th>Blaine fineness, m²/kg</th>
<th>Normal consistency, %</th>
<th>Compressive strength in mortar, MPa, at the age of</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>1 day</td>
</tr>
<tr>
<td>LC3</td>
<td>685</td>
<td>31</td>
<td>7.3</td>
</tr>
<tr>
<td>PPC</td>
<td>322</td>
<td>30</td>
<td>7.4</td>
</tr>
</tbody>
</table>
The compressive strength of the mortar cubes of all cements are shown in Table 4. One day strength of all the cement are in a very close range. However, 28 day strength of LC3 is remarkably low in relation to PPC. This may be contributed to lower clinker concentrations. The LC3 exhibited early strength development up to 7 days and the development between 7 days to 28 days was lower in comparison with PPC.

The compressive strength development from 7 to 28 days of LC3 is 15% of 28 days strength whereas, it is 46% for PPC. The hydration kinetics is closely linked with the intrinsic material characteristics. In the case of fly ash, the products of hydration closely reassembles calcium–silica hydrates produced by hydration of Portland cement. However, the reaction does not start until sometimes after mixing. In the case of fly ash class F, this can be as long as one week or even more. The glass material in fly ash is broken down only when the pH value of the pore solution is at least about 13 [18]. The strength recovery of PPC from 7 days to 28 days is much higher than in relation to LC3.

The lime reactivity of calcined clay was much higher than fly ash resulting in higher demand of Portlandite (CH) availability in the hydration system. The LC3 cement exhibited higher one day strength and higher recovery of strength from 1 to 7 days in comparison with PPC. The higher rate of reactions with available pore solution resulted in total consumption of Portlandite generated in system. This could cause availability of unreacted calcined clay. Pozzolana cement resulted in lower growth or lower strength recovery from 7 to 28 days. Reaction of alumina and calcium carbonate with the progress of hydration process of LC3 is possibly contributing to improved early strength of LC3 up to 7 days in comparison with PPC.

4.2 Hydration behavior

The isothermal (heat conduction) calorimetry is an efficient tool to study the stages related to the hydration of cement pastes or mortars at constant temperature. The calorimeter continuously measures and displays the heat flow related to the hydration reactions taking place in the cement paste after mixing. The respective cement was studied for heat liberation using conduction calorimeter at a w/c ratio of 0.40. The heat flow curve and the total heat liberation curve are shown in Fig. 2 along with that of PPC. The heat liberated at different age are given in Table 5.

It has been observed that LC3 hydrated faster with heat liberation almost double when compared with PPC. However, the same tapers down after 144 hours without significant gains till 28 days. PPC catches up and surpasses LC3 at 28 days. This is in agreement with compressive strength development of the two cements.

4.3 SEM analysis of hydration products

Changes with time in the morphology and nature of the hydration products of LC3 cement, at water to cement ratio of 0.5, were studied by scanning electron microscope. The microstructure was observed at different intervals of hydration: i.e. 1, 3, 7, and 28 days. The samples were studied in both fractured surface and polished section using SE and BSD at variable pressure mode. The hydration products such as calcium silicate hydrate (C-S-H), portlandite (CH), ettringite (AFt), monosulfate (AFm), C-S-A-H, limestone particles, and deleterious materials like quartz and feldspar were observed. The initial products at 1 day of hydration are amorphous looking like fibrous shape. These products mainly appear on the surface of the unhydrated grains, filling in void space as they grow. Fibrous like C-S-H having size <200 nm, CH, Aft, and AFm phases appear more at 1 day.

Interlocking structure of C-S-H and rod like ettringite appear at 3 days. During the first 7 days of
hydration, the surface of the C3S grain are covered by the radiating fibrous particles of C-S-H, honeycomb structure, and interlocking space between the grains.

As time of hydration increased, the fibrous structure developed into a needle like C-S-H at 28 days. At 28 days hydration, the paste displayed a massive tabular structure with platy with occasionally fibrous hydration products. The morphology of C-S-H is similar at 3 and 7 days.

Both type I and II C-S-H were observed in all the hydration period. The size of C-S-H increased form 200 nm to 1.5 micron as the hydration period in creased from 1 to 28 days. The morphology of ettringite also changes from needle to rod like structure as the hydration period increases. Lime stone particles are frequently present at all the ages indicating incomplete reaction with cement particles. The amount of AFm and CH also decreases as hydration period increases.
4.4 Porosity of cement pastes

The porosity of cement paste composed with LC3 and PPC with ratio of water to cementitious material 0.4 was determined using mercury intrusive porosimeter, both in terms of inter particle and intraparticle porosity [19]. The results are listed in Table 6. It is observed that hydrated LC3 paste has lower porosity in comparison with that of PPC. The total porosity of LC3 paste is 0.92% against 1.13% of PPC.

4.5 Admixture demand of cement paste

Admixture compatibility of cements, tested with common admixture showed that the optimum dosage for LC3 was 1.4%, while it was 1.0% for PPT. The flow behavior of cement paste showed that LC3 has higher admixture demand, almost 40% higher as compared to binary blend cements PPC (see Figs. 7 and 8), even though the water demand for normal consistency was similar. This may be due to the higher fineness of LC3.

4.6 Retention of flow

Retention of flow of cement paste shows that LC3 has a low initial flow and poor retention as compared to PPC (see Fig. 9). This may be due to fine limestone powder and higher fineness of LC3.

5. Conclusions

This study observed that the limestone calcined clay cement behaves differently as compared to commercially available fly ash based blended cement. The compressive strength development of limestone calcined clay cement is also different to fly ash based blended cement in terms of low improvement at later ages. The hydration behavior shows a distinct difference in terms of high heat evolution at early age, unconventional of blended cements. Not only the cement paste has a high water demand, resulting in lower workability, but also it needs higher dosage of superplasticizers, if used. Porosity of LC3 cement paste is lower than that of PPC paste showing an improved durability.

Acknowledgements

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References

Experimental investigation of the use of CFRP grid for shear strengthening of RC beams

Ngoc Linh Vu *; Kimitaka Uji; and Vu Dung Tran

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Abstract: Carbon fiber reinforced polymer (CFRP) sheet and grid have been widely applied to improve both shear and flexural strength of reinforced concrete (RC) beams. Compared to CFRP sheet, CFRP grid and sprayed mortar have advantages in dealing with the damaged concrete surface. This work examined the effectiveness of CFRP grid and sprayed mortar in enhancing the shear capacity of RC beams. Three RC beams were fabricated and two of them were strengthened by CFRP grid and sprayed mortar. Then four-points bending test was carried out to collect the data on the behavior of CFRP grid and stirrups in the three beams. The results are presented and discussed in this paper. This study evaluated the strengthening effectiveness of CFRP grid and sprayed mortar and the behavior of stirrups and CFRP grid. The difference in behaviors between stirrups and CFRP grid was analyzed.

Keywords: CFRP grid, sprayed mortar, stirrup, shear strengthening, RC beam.

1. Introduction

After a period of rapid economic growth, repairing and retrofitting of existing infrastructures that have been aging rapidly, such as buildings, bridges, and tunnels, have been among the most important civil engineering challenges all over the world. For example, in Japan, the percentage of highway bridges that are more than 50 years old was approximately 18% in 2013 and will be 43% in 2023. For tunnels, this percentage is 20% and 34%, respectively [1]. The other reason for the demand for strengthening and rehabilitation of structures is the upgrading of their load carrying capacity and resistance to withstand underestimated loads, to ameliorate the increased perceived risk from earthquakes. Since 1990s, carbon fiber reinforced polymer (CFRP) has been used in civil infrastructures to increase the load carrying capacity due to the limited durability of traditional materials. Using CFRP has been considered as a very effective strengthening and rehabilitation method for such work. CFRP has many advantageous engineering characteristics such as high strength-to-weight ratio, corrosion resistance, and ease of application and construction. CFRP currently plays a key role in strengthening and retrofitting concrete structures in Japan. The area of CFRP sheet used was 980,000 m² in 2006 and 1,310,000 m² in 2013 [2]. CFRP is usually used in the form of sheet. The consumption of CFRP in the form of grid was only about 5% of that of CFRP sheets in 2006. CFRP grid and sprayed mortar have advantages in dealing with damaged concrete surface. Scheme of CFRP grid application is shown in Fig. 1.

Fig. 1 – Schematics of CFRP grid reinforcement application

Many experimental studies have been conducted over the past decade to study the performance of concrete beams strengthened in shear with externally bonded FRP composites. Bukhari [3] reviewed the existing design guidelines for strengthening continuous beams in shear with CFRP sheets and proposed a modification to Concrete Society Technical Report. Chen [4] studied the shear behaviour of RC beams with FRP grid and concluded that RC beams strengthened with CFRP grid have good shear behaviour in terms of both increasing th
shear capacity and controlling of the crack width. Guo [5] examined the effect of shear capacity of RC beams reinforced with a haunch using the PCM shotcrete method with CFRP grid and investigated the adhesive properties of the reinforced interface between PCM and the existing concrete. Steffen [6] concluded that the use of CFRP grid could provide a quick, efficient method for providing corrosion resistant concrete deck reinforcement. CFRP grid reinforcement represents a suitable replacement for steel rebars in some concrete structural members subjected to aggressive environmental conditions that accelerate corrosion of the steel reinforcement and cause deterioration of the structures [7]. These studies have not focused on the difference in behaviours between stirrups and CFRP grid. The main purpose of this research was to study the capacity of CFRP grid and the behavior of stirrups and CFRP grid in shear strengthening. This paper illustrates the use of CFRP grid combined with sprayed mortar to improve the shear strength of RC beams.

The existing guidelines are related to the application of CFRP sheet, and there are no specific guidelines for the application of CFRP grid. The material factor in calculating the effectiveness of CFRP grid in providing shear resistance capacity was evaluated. Based on the experimental result, the effectiveness of CFRP grid and sprayed mortar in enhancing the shear strength of RC beams was also evaluated. The difference in behavior between stirrups and CFRP grid was analyzed in detail.

### 2. Experimental Program

Three RC beams were fabricated and tested. The control RC beam was not strengthened while the two other beams were shear strengthened by CFRP grid and sprayed mortar. The three RC beams were designed such that their flexural strength was much higher than their shear strength to ensure that failure was controlled by the shear force.
2.1 Materials

The three RC beams were fabricated using ready-mixed concrete with a compressive strength of 34.1 N/mm$^2$ and maximum aggregate size of 20 mm. In the concrete mix, high-early-strength Portland cement was used and the water-cement ratio was 0.556 with the addition of water-reducing and air-entraining admixtures. The other parameters of the concrete mix are shown in Table 1. The mortar to be sprayed was made by mixing 25 kg premixed mortar, 1.21 kg polymer, and 4.1 kg water. In order to increase interface adhesion, epoxy primer was applied to the surface of concrete before spraying mortar. The properties of the polymer and epoxy primer are shown in Table 2.

D32 was used as the main reinforcing bar in the tension zone. D6 and D10 were used as the stirrup and reinforcing bar, respectively. The CFRP grid used in this test was CFRP-CR8 (The CR8 label is given by the manufacturer) with a grid spacing of 100 mm $\times$ 100 mm. The mechanical properties of concrete and mortar are given in Table 3. Mechanical properties of reinforcing bars and CFRP grid are listed in Table 4.

2.2 Test Specimens

The RC beams had the dimensions 200 mm $\times$ 500 mm $\times$ 2,750 mm. They were labelled RC beam 1 (control beam), RC beam 2, and RC beam 3. RC beam 1 and RC beam 2 were reinforced with 6D32 longitudinal rebars at the bottom and 2D10 longitudinal rebars on the top. The stirrup spacing was 200 mm (see Fig. 4). In RC beam 1, D10 bar was used for stirrups while, in RC beam 2, D6 bar was used as stirrups, which is smaller than the type used in the RC beam 1. With the assumption that the shear strength in this beam was lost due to corrosion, the cross-sectional area of the stirrups was reduced. RC beam 3 was reinforced with 6D32 longitudinal rebars at the bottom and 2D10 longitudinal rebars on the top. The stirrup spacing was 200 mm (see Fig. 4).

Table 5 shows the total shear reinforcement area of each beam. Reinforcement area in RC beam 1 is the total of cross sections of D10, while, in RC beam 3, it is the total cross sections of CFRP grid (vertical grid only) and, in RC beam 2, it is the total of cross sections of D6 and CFRP grid in 200 mm beam length of each beam.

2.3 Casting RC Beams

First, the RC beams were cast in wooden formwork and cured by moisture-retaining cover (see Fig. 5). Eight days after the RC beam 2 and RC beam 3 were cast, their web’s surface was sand blasted. The roughness of the beam surface after sand blasting was 0.15 mm. Four days later, CFRP grid was fixed on both sides of the beam by steel bolt anchors. On the next day, epoxy primer was applied. The roughness of the beam surface after applying epoxy was 0.13 mm. After the epoxy layer had dried, the repair mortar was sprayed (see Fig. 6). Finally, a curing compound was sprayed on the mortar surface. Tests of the RC beams were carried out at the age of 28th, 29th, and 30th day.

2.4 Instrumentation and Test method

Four-point-bending test was performed (see Fig. 7) on the three beams, with a span length ($L$) of 2,350 mm and a shear span ($a$) of 1,100 mm. The effective depth ($d$) was 423 mm. The shear span to effective depth ($a/d$) ratio was 2.6. During the test, when the first flexural crack and the first diagonal crack appeared, the specimens were unloaded to mark the cracks and take photographs. After that the load was continually increased until the beams failed. The failure processes were monitored by strain gauges installed on concrete, reinforcing bars and the CFRP grid, and by displacement transducers placed at mid-span and two end supports of each beam. Fig. 8 through Fig. 10 show locations of the displacement transducers and strain gauges installed on rebar, concrete, and CFRP grid in the three RC beams.

3. Results and Discussions

3.1 Load – displacement behavior, crack development and failure mode

Fig. 8 through 10 describe cracks generated during the test. The cracks shown in bold lines were the largest cracks at the ultimate state of each beam.
Table 5 - Total cross-sectional area of shear reinforcement: CFRP grid and stirrups

<table>
<thead>
<tr>
<th>No.</th>
<th>Shear reinforcement</th>
<th>Total reinforcement area (mm²/200 mm length)</th>
<th>Ratio of shear reinforcement $P = \frac{A_s}{b\times s}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC beam 1</td>
<td>D10 @200 mm</td>
<td>142.66</td>
<td>0.37%</td>
</tr>
<tr>
<td>RC beam 2</td>
<td>D6 @200 mm and CFRP grid CR8 @100 mm</td>
<td>169.34</td>
<td>0.35%</td>
</tr>
<tr>
<td>RC beam 3</td>
<td>CFRP grid CR8 @100 mm</td>
<td>105.60</td>
<td>0.22%</td>
</tr>
</tbody>
</table>

NOTE: $A_s$ - area of shear reinforcement (mm²); $b$ - beam width ($b_{RC1} = 200$ mm; $b_{RC2} = b_{RC3} = 240$ mm); $s$ - shear reinforcement spacing (mm).

Fig. 4 – Longitudinal and transverse cross-sections of three RC beams and strengthening scheme using CFRP grid and sprayed mortar

Fig. 5 – RC beams fabrication

Fig. 6 – Fixing CFRP grid and spraying mortar
Based on the observation of cracks for the three beams, it is supposed that all beam failures were due to diagonal tension. The test results of all beams are summarized in Table 6. RC beam 2 exhibited the highest maximum load (757 kN) while RC beam 3 exhibited the lowest (617 kN) and RC beam 1 showed a value between the two extremes (690 kN). Compared to the control beam (RC beam 1), the maximum test load of RC beam 2 was higher by 9.7% while that of RC beam 3 was lower by 10.6%.

Table 6 – Summary of test results

<table>
<thead>
<tr>
<th>Beam index</th>
<th>Cracking load (kN)</th>
<th>Maximum load (kN)</th>
<th>Max. mid-span displ. (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Design</td>
<td>Test</td>
</tr>
<tr>
<td>RC beam 1</td>
<td>225</td>
<td>354</td>
<td>690</td>
</tr>
<tr>
<td>RC beam 2</td>
<td>300</td>
<td>697</td>
<td>757</td>
</tr>
<tr>
<td>RC beam 3</td>
<td>300</td>
<td>656</td>
<td>617</td>
</tr>
</tbody>
</table>

The total cross-sectional area of shear reinforcement placed in 200-mm beam length was 142.6 mm² (two bars of D10) in RC beam 1, while, in RC beam 2 and RC beam 3, it was 169.3 mm² (two rebars of D6 and four bars of CFRP CR8) and 105.6 mm² (four bars of CFRP CR8), equivalent to 118.7% and 74.3% of that of RC beam 1, respectively. The ratio of shear reinforcement of RC beam 1 was 0.37%, while, in RC beam 2 and RC beam 3, it was 0.35% and 0.22%, equivalent to 94.6% and 59.5%, respectively. The total cross-sectional areas of reinforcing bars and CFRP grid and the ratios of shear reinforcement are shown in Table 5. From test results, in two cases of strengthening, CFRP grid and sprayed mortar proved the effectiveness for enhancing the shear capacity. It could be concluded that concrete, rebar, mortar, and CFRP grid worked well together.

Fig. 11 shows the load versus mid-span displacement curves for the three beams. The maximum mid-span displacements of the three beams are listed in Table 6. RC beam 1 had the highest mid-span displacement of 8.4 mm while RC beam 2 and RC beam 3 had lower values of 7.4 mm and 7.1 mm (lower than 11.9% and 15.5% compared to RC beam 1), respectively. Each load versus mid-span displacement curve (see Fig. 11) could be divided into two stages. The first stage is before the cracking (225 kN for RC beam 1, 300 kN for RC beam 2 and RC beam 3). In this stage, RC beam 2 and RC beam 3 showed higher stiffness than RC beam 1 because their width was larger (240 mm) compared to that of RC beam 1 (200 mm). In the second stage after the cracking, there is a distinct difference between RC beam 1 and RC beam 3. The stiffness of RC beam 3 became lower than that of RC beam 1 due to cracks developed and the cross-section reduced. The Young’s modulus of the stirrups is higher than that of CFRP grid (200,000 N/mm² versus 100,000 N/mm², respectively, in Table 4). RC beam 3 had CFRP grid but no stirrups. Therefore stirrups provide higher stiffness to the beam and result in smaller displacement value compared with the CFRP grid. In general, after strengthened by CFRP grid and sprayed mortar, both RC beam 2 and RC beam 3 had higher stiffness, ductility characteristic, and the cracking loads were also improved (except in the final period for RC beam 3, when the load was over 500 kN).

3.2 Behavior of stirrups in RC beam 1 and CFRP grid in RC beam 3

When the load increased, the RC beams deformed, cracks appeared and developed, and the beams failed. The behavior of the stirrups in RC beam 1 and that of the CFRP grid in RC beam 3 were different. Fig. 12 illustrates the load versus strain curves of stirrups in RC beam 1 and CFRP grid in RC beam 3. Strain gauges S3 and S4 with locations shown in Fig. 8 were installed on the stirrups in RC beam 1. Strain gauges G13 and G35 with locations shown in Fig. 10 were installed on the CFRP grid in RC beam 3. These strain gauges were near the cracks in each beam.

From Fig. 12, in general, strain values on CFRP grid in RC beam 3 are smaller than those on stirrups in RC beam 1. The strain recorded by G13 was the highest in RC beam 3 and the strain recorded by S3 was the highest in RC beam 1. These gauges were close to the ultimate cracks in each beam (see Fig. 8 and Fig. 10). The load versus strain curves consist of 3 stages: In the first stage, there are no cracks in each beam. Cracking load of RC beam 3 was around 300 kN while, for RC beam 1, it was around 225 kN (see Fig. 13). In this stage, at the same load levels, the strain of G35 was smaller than that of S4 and the slope of the experimental load-strain curves of RC beam 3 was higher.
than that of RC beam 1. The reason was the beam width of RC beam 3 was 240 mm (two layers of 20 mm including CFRP grid and sprayed mortar were applied on two sides of the RC beam) compared to the 200 mm width of RC beam 1. Thus, the bending stiffness of RC beam 3 increased by about 10%.

In the second stage, when the diagonal cracks appeared and developed, the length and the width of the cracks increased, the cross-sectional area reduced gradually, and the shear capacity of both beams provided by concrete and mortar reduced. The shear strength of RC beam 1 was mainly provided by stirrups whereas that of RC beam 3 was mainly provided by CFRP grid. Moreover, the Young’s modulus of CFRP grid ($1 \times 10^5$ N/mm$^2$) is a half of that of the stirrup ($2 \times 10^5$ N/mm$^2$) as shown in Table 4. Therefore, strains in RC beam 3 are higher than those in RC beam 1 as shown in Fig. 13 and Fig. 14.

In the third stage, when the load continued increasing, the stress on the stirrup reached the yield
Fig. 11 – Load vs. mid-span displacement curves

Fig. 12 – Load vs. strain curves of stirrups in RC beam 1 and CFRP grid in RC beam 3

Fig. 13 – Load vs. strain curves of S4 (RC beam 1) and G35 (RC beam 3)

Fig. 14 – Load vs. strain curves of S3 (RC beam 1) and G13 (RC beam 3)
strength. From the values given in Table 4, the stirrup is expected to yield at a strain of
\[ \varepsilon_{\text{yield}} = \frac{\varepsilon_{\text{yield}}}{E_s} = \frac{417}{2 \times 10^6} = 2.085 \times 10^{-6}. \] In test, stirrup S3 yielded at a strain of 2.180 \times 10^{-6} equivalent to a load of 475 kN as shown in Fig. 14. When the stirrup yielded, the modulus of shear reinforcing bars reduced rapidly, whereas CFRP grid was elastic until fracture. This is the reason why strains in RC beam 3 were smaller than those in RC beam 1.

3.3 Difference in behavior between CFRP grid and stirrups in RC beam 2

Bonding between concrete and sprayed mortar is one of the most important factors that influence the effectiveness of reinforcement. The difference in behaviours between the stirrups and CFRP grid at the same positions in RC beam 2 may provide the information on the bonding between concrete and sprayed mortar. The strain gauges on CFRP grid G4, G14, G30, and G35 and those on the stirrups S1, S2, S3, and S4 were respectively at the same positions in RC beam 2. Locations of these strain gauges are shown in Fig. 9. The data of the bending test are listed in Table 7. Fig. 15 illustrates the load versus strain curves of the stirrups and CFRP grid in RC beam 2.

The strain curves for the load between 300 kN and 550 kN (see Table 7 and Fig. 15) show the following behaviour: When the load is greater than 350 kN, the strain values on the stirrup are higher than that on the CFRP grid (except stirrup S1), S3/G30 and S4/G35 are higher than S2/G14. At a load greater than 700 kN, the difference in strains between stirrup S4 and CFRP grid G35 becomes larger. At the ultimate stage, the strain recorded by S4 is about 5 times higher than that by G35 (4.9 times higher at a load of 750 kN). Thus, strain gauge readings at the same position differed significantly. This observation proved that, these materials (the CFRP grid and the stirrup) no longer worked together and the bonding between concrete and mortar was damaged. The possible reasons are as follows: First, when the strain on stirrups S2, S3, and S4 was greater than 2,085 \times 10^{-6} (as calculated in section 3.2), the stirrups yielded and deformed rapidly after this point. Second, when the load was increased from 320 to 425 kN, cracks appeared and propagated towards the loading point, the compression area of the cross-section reduced. Most of these cracks were located on the same positions as S2, S3, and S4 (see Fig. 9). Once a crack crossed the locations of stirrups and CFRP grid, the adhesion of mortar to concrete surface was also damaged.

3.4 Contribution of CFRP grid in shear strengthening of RC beams

Design shear capacity values were calculated for different values of shear strength as shown in Table 6. Flexural cracks appeared when the flexural cracking strength of concrete was exceeded by the applied stress. After that, if the stress reached the shear strength of concrete, diagonal cracks would occur. The design shear capacity of RC beam 1 is the sum of shear resistances contributed by concrete and reinforcement while those of RC beam 2 and RC beam 3 are the sums of shear resistances contributed by concrete, reinforcement, and CFRP grid. According to Eq. (9.2.3) of Ref [8], the design shear capacity was calculated using following equations:

\[ V_{yd} = V_{cd} + V_{zd} \]  (1)
Table 7 – Comparison of strains on stirrups and CFRP grid in RC beam 2

<table>
<thead>
<tr>
<th>Load (kN)</th>
<th>CFRP ( )</th>
<th>Stirrup – RC beam 2 ( )</th>
<th>Comparison</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>G6</td>
<td>G14</td>
<td>G30</td>
</tr>
<tr>
<td>50</td>
<td>1.0</td>
<td>1.0</td>
<td>−2.0</td>
</tr>
<tr>
<td>100</td>
<td>8.0</td>
<td>8.0</td>
<td>0.1</td>
</tr>
<tr>
<td>150</td>
<td>12.0</td>
<td>11.0</td>
<td>1.0</td>
</tr>
<tr>
<td>200</td>
<td>13.0</td>
<td>13.0</td>
<td>−3.0</td>
</tr>
<tr>
<td>250</td>
<td>14.0</td>
<td>16.0</td>
<td>2.0</td>
</tr>
<tr>
<td>300</td>
<td>18.0</td>
<td>13.0</td>
<td>200</td>
</tr>
<tr>
<td>350</td>
<td>40</td>
<td>−140</td>
<td>418.0</td>
</tr>
<tr>
<td>400</td>
<td>126.0</td>
<td>11110</td>
<td>10640</td>
</tr>
<tr>
<td>450</td>
<td>338.0</td>
<td>17850</td>
<td>13580</td>
</tr>
<tr>
<td>500</td>
<td>636.0</td>
<td>23050</td>
<td>17890</td>
</tr>
<tr>
<td>550</td>
<td>864.0</td>
<td>27480</td>
<td>21260</td>
</tr>
<tr>
<td>600</td>
<td>12940</td>
<td>32730</td>
<td>26220</td>
</tr>
<tr>
<td>650</td>
<td>14510</td>
<td>37590</td>
<td>31680</td>
</tr>
<tr>
<td>700</td>
<td>15730</td>
<td>42430</td>
<td>38820</td>
</tr>
</tbody>
</table>

Design shear capacity in RC beam 1:

\[ V_{y_{d1}} = V_{cd1} + V_{v_{d1}} \]  \hspace{1cm} (2)

Design shear capacity in RC beam 2:

\[ V_{y_{d2}} = V_{cd2} + V_{sd2} + V_{CFRP} \]  \hspace{1cm} (3)

Design shear capacity in RC beam 3:

\[ V_{y_{d3}} = V_{cd3} + V_{CFRP} \]  \hspace{1cm} (4)

- \( V_{cd} \): design shear capacity without shear reinforcement

\[ V_{cd} = \beta_d \times \beta_p \times \beta_n \times f'_{vcd} \times b_w \times d / \gamma_b \]  \hspace{1cm} (5)

\[ f'_{vcd} = 0.20 \sqrt{f_{cd}} \text{ (N/mm}^2\text{)} \] where \( f_{cd} \leq 0.72 \text{ (N/mm}^2\text{)} \)

\[ \beta_d = \sqrt{\frac{1000}{d}} \text{ (d in mm) when } \beta_d > 1.5, \beta_d \text{ is taken as 1.5} \]

\[ \beta_p = \sqrt{\frac{1000}{p_v}} \text{ when } \beta_p > 1.5, \beta_p \text{ is taken as 1.5} \]

\[ \beta_n = 1 \]

\[ b_w: \text{ web width} \]

\[ d: \text{ effective depth} \]

\[ p_v = A_v/(b_w \times d) \]

where, \( A_t \): area of tension reinforcement (mm²); \( f'_{cd} \): design compressive strength of concrete (N/mm²); \( f_{cd} \) is taken as 34.1 N/mm² with concrete and 36.7 N/mm² with mortar (see Table 3); \( \gamma_b \): member factor, may generally be taken as 1.3; \( V_{cd} \): shear resistance contributed by concrete; \( V_{sd} \): shear resistance contributed by concrete and sprayed mortar; \( V_{v_{d1}} \): shear resistance contributed by concrete and sprayed mortar.

- \( V_{sd} \): design shear capacity of shear reinforcement, taken from Eq.(9.2.6) of Ref [8]

\[ V_{sd} = A_w \times f_{w_{yd}} \times z / S_z / \gamma_b \]  \hspace{1cm} (6)

where, \( A_w \): total area of shear reinforcement placed in \( S_z \); \( f_{w_{yd}} \): design yield strength of shear reinforcement (yield strength of stirrup, D10 for RC beam 1 and D6 for RC beam 2, was taken, \( f_{w_{yd}} \) of RC beam 1 is as 413 N/mm² and \( f_{w_{yd}} \) of RC beam 2 is 417 N/mm² in Table 4); \( z \): distance from location of compressive resultant to centroid of tension steel, which may be taken as \( \sqrt{h \times 1.15} \); \( S_z \): spacing of shear reinforcement; \( \gamma_b \): member factor, may generally be taken as 1.1.

- \( V_{CFRP} \): design shear capacity of CFRP grid. In Japan, there is no standard specification for shear strengthening concrete structure using CFRP grid. In this case, the CFRP grid was taken as shear reinforcement.

According to Eq.(9.2.6) of Ref [8], the following equation was suggested for calculating \( V_{CFRP} \).

\[ V_{CFRP} = A_{CFRP} \times f_{CFRP} \times z / S_z / \gamma_b \]  \hspace{1cm} (7)

where, \( A_{CFRP} \): total area of shear reinforcement placed in \( S_z \); \( f_{CFRP} \): design yield strength of
shear reinforcement. In this experiment, there is no yield strength of CFRP and CFRP can be considered as a linear-elastic material. \( f_{\text{CFRP}} \) was taken as tensile strength, equal to 1,400 N/mm\(^2\); \( \gamma_b \) - member factor of CFRP material (as there is no standard specification for CFRP grid, the safety ratio of 1.3 for CFRP sheet taken from “Guideline for repair and strengthening using CFRP sheet for concrete structure” [9] was used).

According to test results in Table 6, RC beam 1 and RC beam 2 showed the maximum test loads of 690 kN and 757 kN, respectively, much higher than design loads of 354 kN and 697 kN, respectively, due to the following reasons. First, the value of the design load is calculated for linear-elastic material, while at the load of 690 kN, the stirrup in RC beam 1 already yielded; second, there is safety ratio included in the formula in standard specifications. For RC beam 3, the design loads are higher than the experimental maximum load, due to the following reasons. First, applying the formula for stirrups to calculate shear strength of CFRP grid was not appropriate as some coefficients in the formula were not reasonable for CFRP grid, the stirrup has yield strength but CFRP grid does not. Second, when computing the design shear strength, it was assumed that the CFRP grid would work at full capacity. Actually, the maximum stress in CFRP grid was less than 1,400 N/mm\(^2\) as shown in Table 4. At the ultimate state of strengthened beams, the maximum load of RC beam 3 was 617 kN and the maximum strain values of CFRP grid in RC beam 3 were recorded by strain gauges G13 and G35 (location of strain gauges are shown in Fig. 10). The maximum strains were 11,943 \( \times \) 10\(^{-6}\) and 6,377 \( \times \) 10\(^{-6}\) as shown in Fig. 16, respectively. The stress on the CFRP grid was calculated as follows.

\[
\sigma_{\text{CFRP}}^{G13} = \frac{\sigma_{\text{CFRP}}}{E_{\text{CFRP}}^\theta_{\text{CFRP}}} = 1 \times 10^5 \times 11,943 \times 10^{-6} = 119.43 \text{ N/mm}^2
\]

\[
\sigma_{\text{CFRP}}^{G35} = \frac{\sigma_{\text{CFRP}}}{E_{\text{CFRP}}^\theta_{\text{CFRP}}} = 1 \times 10^5 \times 6,377 \times 10^{-6} = 637.7 \text{ N/mm}^2
\]

The maximum stress values on the CFRP grid recorded by G13 and G35 were 85.3% and 45.5% of the tensile strength of CFRP grid (1,400 N/mm\(^2\)) as shown in Table 4. Guideline for repair and strengthening using CFRP sheet for concrete structure [9] suggests that the material factor for calculating CFRP sheet capacity is 1.3. There is no mention of the corresponding value for CFRP grid. Here, using material factor of 1.3, the shear design load of RC beam 3 would be 656 kN, which is not appropriate. In the present research work, a coefficient of 1.5 is proposed, indicating that the CFRP grid works at 66.7% of its tensile strength. Using the coefficient of 1.5, the design load will be 637 kN for RC beam 2 and 600 kN for RC beam 3.

4. Conclusions

Experiments were conducted to study the performance of CFRP grid and sprayed mortar for shear strengthening of RC beams. CFRP grid consists of vertical and horizontal components but this study only focused on researching the vertical component of the CFRP grid. Based on the collected data, the following conclusions were drawn.

1. CFRP grid and sprayed mortar could be significantly effective in shear strengthening. In this experiment, RC beams strengthened with CFRP CR8 and sprayed mortar (RC beam 2 and RC beam 3) attained 110.7% and 89.4% of shear capacity compared with the ultimate load of the control beam (RC beam 1). The cross-sectional area of reinforcing materials in RC beam 2 and RC beam 3 were equivalent to 118.7% and 74.3%, respectively, of the cross-sectional area of stirrups in RC beam 1. In general, the stiffness, ductility characteristic, and the cracking load of the strengthened RC beams (RC beam 2 and RC beam 3) using CFRP grid and spayed mortar were improved.

2. The behavior between the CFRP grid and the stirrups reflects the bonding between concrete and sprayed mortar. It is one of the most important factors that influences the reinforcement effectiveness. According to experimental results, when the load increased, variations of the stirrup strains and the CFRP grid strains at the same position in a strengthened RC beam had similar tendencies. After the stirrups yielded and the cracks developed in a strengthened RC beam, the bonding between concrete and sprayed mortar was considerably affected.

3. In an application of CFRP grid and sprayed mortar, CFRP grid could not work at 100% of the tensile strength. Ref [9] reports the material factor of 1.3 for calculating CFRP sheet capacity, but the maximum stress of CFRP grid was 85.3% of the tensile strength in our experiments. Therefore, a material factor of 1.5 is proposed for the CFRP grid when applying the formula in Ref [8] to calculate the shear strength of a RC beam strengthened with CFRP grid and sprayed mortar. Using a coefficient of 1.5 means the CFRP grid works at 66.7% of its capacity.
It is concluded that the shear-strengthening of RC beams by CFRP grid and sprayed mortar is effective. CFRP grid and sprayed mortar are useful in strengthening and retrofitting of concrete structures. CFRP grid can support or replace stirrups in RC beams in providing shear strength. The availability of CFRP grid and sprayed mortar should be more intensively investigated and widely applied in concrete structures in the future.

References

2. "CFRP Repair and Reinforcement Method Technology Research Association"