CHAPTER 6

FLEXURAL BEHAVIOUR OF GEOPOLYMER CONCRETE
BEAMS EXPOSED TO ELEVATED TEMPERATURES

6.1 INTRODUCTION

The Flexural behavior, namely deformation characteristics, moment–curvature relationship and cracking of fly ash based geopolymer concrete beams exposed to elevated temperatures (200 °C, 400 °C, 600 °C and 800 °C) are presented in this chapter.

6.2 BEAM DETAILS

The mixture proportion used for casting beams was the same as that used for the study of engineering properties of GP concrete after exposure to elevated temperatures (Chapter 5). Accordingly, the following parameters have been considered for the preparation of GP concrete beams.

- Aggregate content by volume = 70%
- Mass ratio of fine aggregate to total aggregate = 0.35
- Ratio of alkali to fly ash by mass = 0.55
- Molarity of NaOH = 10
- Ratio of Na₂SiO₃ to NaOH = 2.5
- Curing temperature = 100 °C
- Temperature curing time = 24 hours

Details of mixing, casting and heating of specimen are explained in section 3.3 of chapter 3. For the present study, the target temperatures selected were 200 °C, 400 °C, 600 °C and 800 °C.

Reinforced geopolymer beams of size 150 mm (W) x 200 mm (D) x1100 mm (L) were used for the present study. Figure 6.1 depicts the reinforcement details of the beam considered.
Two ribbed bars of 10 mm diameter (HYSD) were used as bottom reinforcement. 6 mm diameter ribbed bars (HYSD) were used for shear reinforcement. The shear reinforcement in the form of closed links was spaced at 80 mm center to center. The top hanger bar consists of two 8 mm diameter ribbed (HYSD) bars.

Three groups of beam were cast by varying the clear cover to the bottom reinforcement (20 mm, 30 mm and 40 mm). In each group, 5 beam specimens were cast. Air cooling was adopted to bring down the temperature to ambient after heating to target temperature (200 °C, 400 °C, 600 °C and 800 °C).

![Fig. 6.1. Reinforcement details of GP concrete beam](image)

**6.3 TESTING OF BEAMS**

The beams were tested under two point load, applied at one third span. Figure 6.2 shows the experimental set up for testing GP concrete beam specimens.

![Fig. 6.2. Experimental set up for loading GP concrete beam](image)

Demountable mechanical gauge (DEMEC) of 200 mm gauge length was used for measuring strain across the depth of the beam at the midspan of the specimen.
Loading on the beam was applied at an increment of 3 kN and for each increment of load, DEMEC gauge readings were taken. Observations like deflection, load at first crack, crack width, crack propagation etc. were also noted wherever applicable, at every load increment.

6.4 ANALYSIS OF RESULTS

Table 6.1 shows the load at first crack and ultimate load on GP concrete beam tested after exposure to different temperatures.

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Cube compressive strength of GP concrete (MPa)</th>
<th>Load at fist crack (kN)</th>
<th>Ultimate load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>20 mm cover 30mm cover 40mm cover</td>
<td>20 mm cover 30mm cover 40mm cover</td>
</tr>
<tr>
<td>Ambient</td>
<td>57.30</td>
<td>45 43 40</td>
<td>101 99 98</td>
</tr>
<tr>
<td>200°C</td>
<td>42.52</td>
<td>42 42 36</td>
<td>94 95 92</td>
</tr>
<tr>
<td>400°C</td>
<td>37.33</td>
<td>36 39 33</td>
<td>92 92 78</td>
</tr>
<tr>
<td>600°C</td>
<td>30.82</td>
<td>33 36 33</td>
<td>85 90 75</td>
</tr>
<tr>
<td>800°C</td>
<td>32.88</td>
<td>30 33 30</td>
<td>68 75 66</td>
</tr>
</tbody>
</table>

From Table 6.1 it could be observed that the load at first crack for beam at ambient temperature reduces marginally with increase in clear cover to the reinforcement. However, for temperature above 200 °C GP concrete beam with a clear cover of 30 mm shows slightly higher load capacity against first crack compared to beams with 20 mm and 40 mm cover.

The ultimate load on beams after exposure to temperatures above 200 °C is also slightly higher for the beam with 30 mm cover compared to that of the beam with 20 mm and 40 mm cover. However, considering the possible variations in the test results, it could be concluded that, the variation of cover to reinforcement up to 40 mm has no significant influence on the first crack load and on the ultimate load of GP beam after exposure to elevated temperatures.

It could be noted from Table 6.1 that, even though the cube strength of GP concrete is not reduced between 600 °C and 800 °C, the load caring capacity of beams reduces rapidly beyond 600 °C. This could be primarily due to the rapid strength reduction of reinforcing steel in the beam at these temperatures. Details of mechanical properties of GP concrete after being exposed to different temperatures are presented in
Table D.1 of APPENDIX D. Details of stress strain curve and strength of reinforcing steel after exposure to different temperatures are presented respectively in Table D.2 and Fig. D.1 to D.5 of APPENDIX D.

Figure 6.3 shows the typical load deflection graph of GP concrete beam after exposure to elevated temperatures (load deflection graph of all the beams are presented in Fig. D.6 to Fig. D.8 of APPENDIX D).

As expected, for a given load the deflection is more for a GP concrete beam exposed to higher temperature. Larger deformation with temperature increase is due to the development of more number of micro cracks as well as due to the reduced strength of materials (concrete and steel) at elevated temperatures. It may be noted from Fig. 6.3 that, the rate of increase of the deflection of beams slightly reduces when the temperature is increased from 600 °C to 800 °C, as against the rate of increase of deflection of beams exposed to a temperature up to 600 °C. This behaviour is more predominant after the initiation of crack and is primarily due to the slight strength gain of GP concrete beyond 600 °C.
Figure 6.4 shows the typical moment curvature (m-\(\phi\)) relationship of GP concrete beam after exposure to elevated temperatures (m-\(\phi\) relationship of all beams is shown in the Figs. D.9 to D.10 of APPENDIX D).

![Graph showing moment-curvature relationship for different temperatures](image)

**Fig. 6.4. Typical moment curvature curve of GP concrete beam after exposure to elevated temperatures**

It could be seen from Fig.6.4 that, for lower temperature exposures, the m-\(\phi\) relationship shows a bilinear curve, which is similar to that of RCC beams [149]. A definite yield stage could be observed for GP concrete beams when they are exposed to a temperature up to 400 °C. However, beyond 400 °C, the m-\(\phi\) curve becomes multi-linear. Further, a clear yield stage of the beams is not visible in m-\(\phi\) relationship for temperature exposure beyond 400 °C. The curvature of the beam also increases beyond 400 °C. This is due to the development of more number of internal cracks as well as due to the low residual strength of materials beyond 400 °C.

Figure 6.5 compares the experimental m-\(\phi\) relationship of the beam with the theoretical values for two extreme temperature ranges as a typical case.
The theoretical $m$-$\phi$ curves have been obtained based on strain compatibility criteria and by considering the residual strength and modulus of elasticity of materials at the temperature considered. A sample calculation is presented in D.3 of APPENDIX D.

For the temperature exposure within the extreme temperatures presented (ambient and 800 °C) the $m$-$\phi$ relationship lies between these two extreme curves. These details are presented in Fig. D.11 APPENDIX D.

From Fig.6.5, it could be observed that, the experimental $m$-$\phi$ relationship of geopolymer concrete beams has been predicted correctly at ambient temperature. However, as the exposure temperature increases, the theoretical values underestimate the curvature up to the yield moment. While the theoretical curve shows a bilinear behaviour, the experimental curve (800 °C temperature exposure) shows a multi-linear variation. However, towards the ultimate moment, the theoretical curvature tends to meet the experimental value reasonably well.

Figure 6.6 compares the variation of curvatures at cracking and yielding stages of the beam exposed to different temperatures.
From this figure, it is clear that, the curvature varies linearly with temperature between cracking and yielding of reinforcement.

Ultimate moment of resistance of GP concrete has been calculated theoretically in a way similar to the calculation for R.C.C. beams and by considering the residual strength of steel and concrete at different exposure temperatures. The predicted value is only marginally lower than that of the experimental results (within 12%). Hence it could be concluded that, the m-φ relationship of the geopolymer concrete beam at ambient temperature behaves similarly to RCC beams and it could be predicted well by adopting strain compatibility criteria. However as the exposure temperature increases, the theoretical method very much underestimates the curvature between values corresponding to the first cracking and yielding.

Table 6.2 shows the ductility ratio of GP concrete beam (typical) after exposure to elevated temperatures. The ultimate curvature has been considered as the curvature.

<table>
<thead>
<tr>
<th>Temp. (°C)</th>
<th>Mu (kNm)</th>
<th>My (kNm)</th>
<th>Mu/My</th>
<th>φ_u (radian/mm)</th>
<th>φ_y (radian/mm)</th>
<th>φ_u/φ_y</th>
</tr>
</thead>
<tbody>
<tr>
<td>28</td>
<td>15.8</td>
<td>14.1</td>
<td>1.12</td>
<td>0.000188</td>
<td>0.00004</td>
<td>4.7</td>
</tr>
<tr>
<td>200</td>
<td>15.0</td>
<td>13.3</td>
<td>1.13</td>
<td>0.000176</td>
<td>0.00004</td>
<td>4.4</td>
</tr>
<tr>
<td>400</td>
<td>14.1</td>
<td>12.2</td>
<td>1.16</td>
<td>0.000165</td>
<td>0.000048</td>
<td>3.4</td>
</tr>
<tr>
<td>600</td>
<td>13.3</td>
<td>11.2</td>
<td>1.19</td>
<td>0.00016</td>
<td>0.00008</td>
<td>2.0</td>
</tr>
<tr>
<td>800</td>
<td>10.83</td>
<td>9.6</td>
<td>1.13</td>
<td>0.000158</td>
<td>0.000095</td>
<td>1.7</td>
</tr>
</tbody>
</table>
corresponding to 95% of the ultimate load. From the Table 6.2 and Fig 6.4, it could be observed that, while the ratio of ultimate moment to yield moment does not vary much with temperature, the ductility of the GP concrete beam reduces as the exposure temperature increases. This is because of the fact that, while both ultimate moment (Mu) and yield moment (My) reduces more or less at a constant ratio with temperature, curvature at yield (φy) increases towards curvature at ultimate stage (φu) with increase in temperature.

Figure 6.7 shows typical crack pattern of GP concrete beam. It could be observed from Fig.6.7 that, the crack pattern is similar to that of R.C.C beam.

Different codes of practices propose different permissible maximum crack width based on the exposure conditions. These maximum permissible crack widths ranges from 0.1 mm to 0.3 mm in BS [120] and BIS [119] code of practices and ranges from 0.1mm (0.04 in) to 0.4 mm (0.16 in) in the case of ACI code [129].

Once the beams are exposed to elevated temperatures, existing cracks if any may widen under service load, leading to unacceptable serviceability conditions. So a beam after exposure to elevated temperature may have to have either reduced service load or to have additional protection, primarily corrosion protection to reinforcing bars. Hence it is important to understand the extent of the crack development at service load stage after beams are exposed to elevated temperatures.
So, the present study focuses on the cracking behaviour of GP concrete at service load and when exposed to elevated temperatures. For the present study, the service load has been assumed as 2/3rd of ultimate load of the reference beam that was not exposed to elevated temperature.

To the best of the author’s knowledge, there is no equation available to predict the crack width of GP concrete beam after exposure to elevated temperatures. Hence the suitability of available equations for RCC beams has been checked for the prediction of crack width of geopolymer concrete beams. The equations proposed by different investigators and code of practices [119,121,124,125] have been considered in the present study. As these equations are proposed primarily for crack width calculation at ambient temperature, appropriate residual strength parameters of GP concrete and steel have been used in the equations for determining crack width at elevated temperatures.

Figures 6.8 and 6.9 illustrate a typical graph comparing the experimental results with the theoretically calculated crack width at different temperature exposure.

![Graph comparing theoretical and experimental crack width at different temperature exposure](image)

Fig. 6.8. Comparison of theoretical and experimental crack width at different temperature exposure (30 mm cover)

From these figures it could be observed that, while some equations underestimate the crack width of GP concrete at elevated temperatures, others overestimate it. However the rates of development of crack width with temperatures calculated based on the equations considered is more or less the same as that of the experimental curve.
Figure 6.9. Comparison of theoretical and experimental crack width at different temperature exposure (40 mm cover)

Figure 6.10 shows the variation of crack width with temperature for GP concrete beams at different load stages.
The graph has been plotted considering the load stage in terms of the ultimate load (l/ul). The plot has been limited to the crack width between 0.1mm and 0.3 mm (serviceability condition).

From Fig. 6.10 it could be observed that, the development of crack width is almost in a linear pattern with temperature rise for all beams (with cover 20 mm to 40 mm) and under all load stage (l/ul from 0.4 to 0.7). The average slope of the curve depicted in Fig. 6.10 could be assessed as 1 in 1000.

Since the cracking pattern of GP concrete exposed to elevated temperatures is more or less in a linear pattern, the crack width at any temperature can be linearly interpolated. So, the total crack width of a cracked GP concrete beam exposed to elevated temperatures could be calculated by knowing the average slope of the curves and the initial crack width.

In order to get crack width corresponding to a particular load stage of GP concrete beams at ambient temperature, Fig. 6.11 has been plotted, which is a scattergram between crack width and (l/ul) for all the beams tested (beams with 20 mm, 30 mm and 40 mm cover). From this scatter diagram, it could be seen that, a linear equation could be proposed to predict the crack width of a GP concrete beam tested at ambient temperature. Accordingly, the following equation could be proposed for the determination of crack width (between 0.1 mm and 0.3mm) of the GP concrete beam subjected to load (l/ul) at ambient temperature.

\[ C_{wa} = \frac{6667}{10000}(l/ul) - \frac{2274}{10000} \quad 0.4 < (l/ul) < 0.8 \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (6.1) \]

Where, \( C_{wa} \) is the crack width at ambient temperature in mm and is valid for crack width between 0.1mm and 0.3mm.

Since the slope of the curves showing the variation of crack width of GP concrete beam after exposure to elevated temperatures comes to about 1 in 1000, the crack width of GP concrete caused due to an increase in temperature of \( T \) can be calculated as \((T/1000)\). Hence the following equations have been proposed to assess the crack width of GP concrete beam exposed to elevated temperatures.

\[ C_{wt} = C_{wa} + \frac{T}{10000} \quad , \quad 28 < T \leq 800 \quad \& \quad 0.4 < l/ul < 0.8 \quad \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots \ldots (6.2) \]

Where, \( C_{wt} \) is the crack width in mm at a temperature exposure of \( T \) °C.
So, using equations 6.1 and 6.2, the service load on GP concrete beams exposed to elevated temperatures could be predicted for a limiting value of crack width (between 0.1 mm and 0.3mm) or vice versa.

6.5 CONCLUSIONS

Following conclusions could be made based on the results presented and discussions carried out in this chapter.

1. Once exposed to elevated temperatures, geopolymer concrete beams develop cracks at an early load stage.

2. The load carrying capacity of geopolymer concrete beam reduces rapidly beyond temperature exposure of 600 °C, even though its corresponding cube compressive strength is not affected by the temperature exposure beyond 600 °C.

3. With temperature, both ultimate moment and yield moment of geopolymer concrete beams reduces more or less at in a constant rate. However with increase in temperature, the curvature at yield of geopolymer concrete beam increases and there by a reduced ductility has been observed. For the present study, compared to the ductility at ambient temperature, the ductility of geopolymer concrete beams exposed to 800 °C reduces by 63.8%.
4. Appropriate equations have been proposed to predict the crack width of geopolymer concrete beams exposed to elevated temperatures. These equations could be used to limit the service load on GP concrete beams exposed to elevated temperature (up to 800 °C) for a predefined crack width (0.1 mm to 0.3 mm) or vice versa.

5. The moment-curvature relationship of geopolymer concrete beams at ambient temperature is similar to that of RCC beams and this could be predicted using strain compatibility approach.

6. Once the beams are exposed to elevated temperatures, the strain compatibility approach underestimates the curvature of geopolymer concrete beams between first cracking and yielding point.

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