Ultimate Strength of Ferro-Geopolymer Composite Built-Up I Joist

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Abstract: An experimental study was carried out to study the behaviour of ferro-geopolymer built-up I-joist with different types of mesh reinforcements under flexure. Mesh reinforcements considered in this study are square welded meshes, square woven meshes and hexagonal meshes. First crack load as well as ultimate strength of ferro-geopolymer built-up I-joist in flexure was obtained. An attempt was made to predict the first crack load and ultimate moment capacity of the specimen.

Key words: Geopolymer, Ferrocement, First Crack Load and Ultimate Strength.

1. Introduction
Ferrocement can be described as a composite, where the brittle cement mortar matrix is reinforced layers of wire mesh throughout the matrix, resulting in better performance than the individual one. Ferrocement members are very thin and usually the thickness does not exceed 40 mm. The product formed due to the polymerisation reaction of any source material which is rich in silicon and aluminium such as fly ash, slag, metakaolin etc. with an alkali solution can be termed as Geopolymer. These can be utilised as a substitute for cement and thereby reducing the overall production of greenhouse gases. Ferro-geopolymer composite is obtained when the geopolymer mortar matrix is reinforced with the wire meshes to form the desired structural element. Many studies have been conducted in the past on investigating the flexural strength of ferrocement elements. Collen [1] tested trough-sectioned ferrocement beams of varying the span, based on which he designed and constructed roof of flour mills upto span of 7.82 m. Balaguru [2], Desayi et al. [3], Logan and Shah [4] also conducted studies on flexural strength of ferrocement elements. However very limited studies are available in the field of ferro-geopolymer composite. Considering the gap it is proposed to conduct an investigation on ferro-geopolymer composites.

2. Materials and methods
2.1 Materials
In the present study, three ferro-geopolymer composite built-up I joists have been cast and tested. The different types of wire meshes used and volume fraction are the variables included in the study. The specimens were tested for flexure under two-point loading. The ultimate loads as well as maximum deflection were measured and attempts have been made to develop methods for predicting the ultimate strength in flexure. The computed values have been compared with the experimental results. The materials used for this study consisted of wire meshes, HYSD reinforcement bars of 6 mm diameter,
fly-ash Class-F, manufactured sand and alkaline solution. Three different types of wire mesh i.e., square welded mesh, square woven mesh and hexagonal wire mesh; details are provided in Table 1. The number of layers of wire meshes were chosen such that volume fraction along the loading direction (the equivalent volume fraction, \( V_f \)) was kept constant in all the three beams.

| Mesh type               | Designation gauge | Proof Stress, \( f_p \) (MPa) |
|-------------------------|-------------------|-------------------------------|
| Square Welded Mesh      | 2/21              | 366                           |
| Square Woven Mesh       | 5/22              | 275                           |
| Hexagonal Mesh          | C24               | 230                           |

Note: 1. 2/21 is a square welded mesh of 21-gauge wire; i.e., mm wire at 12.7 mm c/c.
2. 5/22 is a square woven mesh of 22-gauge wire; i.e., 0.81 mm dia wire at 5.08 mm c/c.
3. C24 is a hexagonal chicken mesh of 24-gauge (0.56 mm dia) wire.

Fly-ash (class F) was obtained from a thermal power plant near Coimbatore, Tamil Nadu. The manufactured sand was available locally; it was carefully sieved to obtain the grading size as per table 3.1, ACI 549. The geopolymerisation reaction occurs under the presence of alkaline activators. A solution of Sodium Silicate to Sodium Hydroxide (14M) was prepared in the ratio 2.5:1. The Solution was prepared 24 hours prior to casting in order to ensure proper reactivity. The mix details are as shown in Table 2. In case of geopolymer, as there is no standard code available, the optimum mix design was arrived at based on the method suggested by Rangan. The cross sectional detail of the built-up I-joist and the reinforcement details of the channel specimen are as shown in Figure 1a and 1b respectively.

![Figure 1a. Cross Sectional Details of the Specimen.](image)

![Figure 1b. Reinforcement distribution.](image)

| S. No. | Ratios                              | Values |
|--------|-------------------------------------|--------|
| 1      | Alkaline activator solution to Binder ratio | 0.4    |
| 2      | Sodium Silicate to Sodium Hydroxide ratio | 2.5    |
| 3      | Binder to Aggregate ratio           | 2.5    |
| 4      | Water to Binder ratio               | 0.1    |
| 5      | Super plasticiser to Binder ratio   | 0.01   |

The test specimens comprised of three built-up I joists having three different types of wire mesh reinforcements. Table 3 gives the details of wire mesh and bar reinforcements used in the channel specimen. Diameter of the holes was increased wherever required with the help of a driller as depicted in the Figure 2a. The bolts were designed properly before providing in the shear span. Figure 2b depicts the final built-up I-joist.
Table 3. Details of reinforcement

| Specimen Designation | No. of 6mm dia bars in longitudinal direction | No. of 6mm dia U shaped bars in transverse direction | No of layers | Volume fraction in loading direction |
|----------------------|-------------------------------------------|-----------------------------------------------|--------------|--------------------------------------|
| B1                   | 4                                         | 14                                           | 3            | 0                                    | 0.510                          |
| B2                   | 4                                         | 14                                           | 0            | 2                                    | 0.533                          |
| B3                   | 4                                         | 14                                           | 2            | 3                                    | 0.535                          |

All the three beams were of 2.5 m effective span. Four bars of 6 mm diameter were provided along the longitudinal direction, which acted as the skeletal reinforcement for the channel section. U shaped bars were provided along the transverse direction, spacing was reduced in the shear span so that shear failure is avoided. A brick masonry mould was prepared of the required dimension. The skeletal steel along with meshes on either side was tied securely by binding wire and placed over the mould. The mortar was mixed using rotary pan mixer having a capacity of 8 m$^3$/hr. Admixture in the form of plasticiser (Complast SP 430) was added in order to enhance workability of the mix. The mortar was applied into the meshes using trowels and floats were used to obtain a perfect finish, simultaneously mortar cubes were cast to check the compressive strength. The specimen was left for open steam curing under a pressure of 0.5 psi at 60°C for a period of 24 hours. Thick tarpaulin sheets were used to cover the whole arrangement so that the steam would be retained inside. The built-up I joist was prepared by connecting the channel sections back to back using 20 mm dia bolts in a staggered arrangement along the shear span on either side of the joist. The specimen was tested under two-point loading. 300 tonne capacity UTM was used. The deflections, stain, crack patterns were noted at regular intervals. All the three elements failed in flexure exhibiting high curvature. Table 4 gives the results of the flexure test. Figure 3 shows the load-deflection plots for all the three elements.

3. Results and Discussion

The first crack load as well as ultimate load of the members under flexure was determined analytically. Equations were adopted from previous literatures.

3.1 Cracking Moment

The cracking moment, $M_{cr}$, is calculated using the Eq. (1) given by

$$M_{cr} = f_{cr} \times l_0/y_t$$

Where $f_{cr}$ is the modulus of rupture of mesh-mortar mix computed from Eq. (2),

$$f_{cr} = 0.1962\sqrt{f_c}$$

(1) (2)
The following equation was adopted from the work carried out by Desayi et al. [4]. $f_c' = 0.8f_c$, where $f_c$ is the compressive strength of mesh-mortar mix and is calculated using the Eq. (3)

$$f_c = 0.8614f_{cu} \left[ 1 + 1.095 \frac{V_f}{f_{cu}} \right]$$  \hspace{1cm} (3)

where, $V_f = \text{total volume fraction in the specimen}$

$I_g = \text{gross moment of inertia}$

$y_t = \text{distance from neutral axis to extreme fibre}$

$f_{cu} = \text{maximum compressive strength of the geopolymer mortar cube specimen}$

$f_s = \text{proof stress or yield strength of the wire mesh adopted}$

$P_m = \text{ratio of area of wire mesh in longitudinal direction to the gross area of the specimen.}$

Also the cracking load, $P_{cr}$, is given by Eq. (4),

$$P_{cr} = \frac{M_{cr}}{\left(\frac{L'}{L}\right)}$$  \hspace{1cm} (4)

Where $L'$ is the shear span. Table 5 shows the results of the cracking load obtained and the ratio of computed to experimental values of cracking load for all the three specimens.

| Table 4. Strength of Test Specimens |
|-----------------------------------|
| Specimen Designation | Type of Failure | Mortar Cube Strength (MPa) | First Crack Load (kN) | Ultimate Load (kN) |
|-----------------------|-----------------|---------------------------|-----------------------|-------------------|
| B1                    | Flexure         | 36                        | 5.9                   | 27.47             |
| B2                    | Flexure         | 35                        | 5.9                   | 23.54             |
| B3                    | Flexure         | 36                        | 5.9                   | 21.60             |

As the experimental first crack loads were deduced from the Load-Deflection plots, the deviation from the computed value was found to be higher for all the three beams.

| Table 5.First Crack Load of Test Specimens |
|--------------------------------------------|
| Specimen Designation | Experimental First Crack Load (kN) | Computed First Crack Load (kN) | Ratio of Computed Load to Experimental Load |
|-----------------------|------------------------------------|-------------------------------|--------------------------------------------|
| B1                    | 5.9                                | 3.89                          | 0.66                                       |
| B2                    | 5.9                                | 3.45                          | 0.59                                       |
| B3                    | 5.9                                | 3.22                          | 0.55                                       |

3.2 Ultimate Moment

Two different methods were used to determine the ultimate moment of resistance of the section, $M_{UR1}$. In the first method, the ultimate moment is determined using the procedure similar to that employed for reinforced concrete sections. Flexural compressive stress distribution in mortar at ultimate is assumed to be rectangular in shape with a value equaling to 68% of that of the cube strength. Tensile strength contribution of mortar is neglected. The wire meshes are assumed to be uniformly distributed over the entire depth of the section with a value equaling to the yield stress, as shown in the Figure 4. Table 6 shows the computed as well as the experimental ultimate loads of all the three specimens.
In the second method, the ultimate resisting moment, $M_{UR2}$ is determined using the Eqs. (5-7)

$$M_{UR2} = M_{fu} + M_{su}$$

$$M_{fu} = f_{cb} \frac{d_{c}}{y_{t}}$$

$$M_{su} = \min \{A_{sc} \sigma_{y} (d - t), A_{st} \sigma_{y} (d - t)\}$$

where,

$A_{sc}$ = Area of steel in compression

$A_{st}$ = Area of steel in tension

$\sigma_{y}$ = Yield strength of steel reinforcement bar

$d$ = Effective depth of cross section
t = Thickness of the section

### Table 6. Ultimate Loads by Method I & Method II

| Specimen Designation | Experimental Ultimate Load (kN) | Computed First Crack Load (kN) | Method I | Method II | Method I | Method II | Ratio of Computed Load to Experimental Load |
|----------------------|---------------------------------|--------------------------------|----------|----------|----------|----------|---------------------------------------------|
| B1                   | 27.47                           | 28.96                          | 24.31    | 1.05     | 0.88     |          |                                             |
| B2                   | 23.54                           | 23.24                          | 23.98    | 0.98     | 1.01     |          |                                             |
| B3                   | 21.60                           | 22.87                          | 23.95    | 1.06     | 1.10     |          |                                             |
| Average              |                                 |                                |          | 1.03     | 0.99     |          |                                             |
| Coefficient of Variation |                               |                                |          | 0.06     | 0.10     |          |                                             |

The ultimate moment as well as loads of all the three specimens computed using the second method is shown in Table 6. The results are compared with the experimental values. From both the tables it can be observed that, both the methods agreed satisfactorily with the test results.

### 4. Conclusions

All the three joists exhibited high ductility and failed in flexure and following conclusions are arrived at:

- The beam reinforced with square welded meshes was observed to have the maximum load carrying capacity among the three.
- Maximum ductility was exhibited by the beam reinforced with the combination of hexagonal wire mesh and welded square mesh, even though the specimen had the least load carrying capacity.
- The first crack load as well as ultimate moment capacity of the section was calculated and compared with the experiment values.
- The ultimate moment of resistance was computed using two different methods; it was found that the values predicted by Method I showed a smaller coefficient of variation when compared to that of Method II, hence Method I provides value closer to the experimental value.
- Computation using Method II is relatively easier than Method I, hence it can be employed for rough checks.

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