The effect of reinforcement ratio on the flexural performance of alkali-activated fly ash-based geopolymer concrete beam

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ABSTRACT

Alkali activated fly ash (AAFA) based geopolymer concrete structure is getting attention due to its eco-friendly construction characteristics and improved engineering properties. However, comprehensive studies on the structural performance of hardened properties of AAFA geopolymer concrete is not well addressed, especially on non-linear fracture behavior. This paper aims to present the reinforcement ratio effect on the flexural performance and non-linear fracture characteristics of alkali activated fly ash based geopolymer concrete beams. Sixteen finite element (FE) and four experimental models were used to study the effect of reinforcement ratio on the flexural performance and non-linear fracture characteristics. Four groups of concrete specimens with an average compressive strength of 19.30 MPa, 32.60 MPa, 38.20 MPa, and 41.70 MPa were utilized under this study. To investigate the effect of reinforcement ratio on flexural performance of the beams, reinforcement ratios of 0.03, 0.042, 0.045, and 0.063 were used for each compressive strength class. The result showed that the ultimate load carrying capacity of the beam showed significant improvement by about 36.38% by increasing of reinforcement ratio from 0.03 to 0.063 by keeping the compressive strength of concrete constant. However, it was observed that the effect of compressive strength was not such substantial as reinforcement ratio in enhancing the ultimate load bearing capacity. The experimental result showed that the increase in ultimate load by keeping the reinforcement ratio constant is about 12.20% for different compressive strength. Furthermore, the crack formation in the concrete was highly associated with the tensile reinforcement ratio, i.e., smaller reinforcement ratio led to higher strain growth in the concrete. Moreover, the validation study between the numerical simulation and test results showed a good agreement.

1. Introduction

Sustainable and eco-friendly construction materials have a substantial role in the mitigation of greenhouse gas emissions to the environment. An exponential urbanization and infrastructure development have created a vital need for a sustainable construction material with lower carbon dioxide (CO₂) emission [1, 2]. Researches confirmed that there is about one kilogram of carbon dioxide emission per one kilogram production of Ordinary Portland Cement (OPC) [3, 4, 5, 6]. Thus, the construction industry is a major source of greenhouse gas emissions. As a result, an innovative and eco-friendly cementitious material is required as alternative binder material in the construction industry. Currently, alkali-activated fly ash (AAFA) based geopolymer concrete is getting momentum as alternative material due to its environmental benefits and an improved properties in hazardous environment and mechanical loads [7, 8]. Alkali-activated geopolymer concrete is a concrete derived from the reaction of an alkali solution with a pozzolanic binder materials which have an adequate alumina and silica content dominant materials [9, 10]. Previous studies reported that geopolymer concrete resulted an improved hardened engineering and physical properties as compared to Ordinary Portland Cement concrete [11, 12, 13, 14].

Several studies were conducted on the engineering properties of alkali activated fly ash geopolymer concrete. According to previous researches, the mechanical properties of geopolymer concrete such as compressive strength and tensile strength is higher than as compared to the OPC concrete [15, 16, 17, 18, 19, 20, 21]. It is also found that...
the geopolymer concrete has superior flexural strength under the static loading [22]. On the other hand, the elastic modulus of geopolymer concrete is lower than conventional cement-based concrete [23, 24, 25]. However, the utilization of steel fiber enhances the elastic modulus of geopolymer concrete [25]. Furthermore, it has been reported that the comparative study for the bond strength of geopolymer concrete shows higher bond strength than OPC concrete for an equivalent compressive strength of concrete [26, 27]. This is due to the higher splitting tensile strength of geopolymer concrete than that of Ordinary Portland Cement concrete of the same compressive strength. According to Abdulrahman et al. [28], the bond strength between the interface of reinforcing bar and the concrete matrix depends highly on the splitting tensile strength than the compressive strength of concrete. Mahdi et al. [11] reported that geopolymer concrete depicted an improved bond strength than OPC concrete as compared to OPC concrete [29, 30, 31].

The flexural performance of reinforced concrete structures depends on several factors such as concrete strength, cross-sectional parameters, reinforcement ratio, fiber content, shear span, water to cement ratio, aggregate types, curing and environmental conditions and related factors. However, the effect of each factor has its own degree of contribution that ranges from minor to significant outcome on the strength of concrete. For instance, a research work conducted by Du et al. [32] and Wu et al. [33] shows that the beam depth and reinforcement ratio have significant effect on flexural performance of geopolymer concrete beam. Recently, the utilization of fiber in geopolymer concrete is becoming a best choice to enhance the structural performance. According to previous researches, the flexural performance of fiber reinforced geopolymer concrete beams proven a superior performance as compared to specimens with zero fiber content, especially the ductility characteristics of geopolymer concrete members [34, 35, 36, 37, 38].

An adequate ductility in reinforced concrete structure is very important to avoid a brittle failure of the structure by ensuring acceptable curvature at the ultimate limit state. Usually the ductility behavior of material is described as the capacity of the material to undergo sufficient deformation without a significant loss in its load-carrying capacity [39, 40]. The ductile behavior of reinforced concrete can be affected by several factors such as concrete compressive strength, reinforcement ratio, addition of fibers, aggregate type [30, 41, 42, 43]. According to Muhammed et al. [44], the experimental investigation for the effect of fiber type and content on the flexural behavior of high strength concrete beams shows that increasing in reinforcement ratio resulted dropping of cracking to ultimate moment ratio. In addition, the higher dosage of steel and short carbon fiber resulted in lesser structural ductility of reinforced concrete beams for constant reinforcement ratios. This is reported as because of the increase in the stress intensity factor. Thus, the dosage of reinforcement and fiber influences the ductile behavior of reinforced concrete structures.

Despite the advantages, geopolymer concrete needs further studies to improve its brittle failure characteristics. The main objective of this study is to investigate the effect of reinforcement ratio on the ductility behavior in alkali-activated geopolymer beam using experimental and finite element analysis (FEA). A refined nonlinear finite element analysis results were reported on the fracture behavior geopolymer concrete beam in this study. This study also includes a comparative analysis between the results from FEA and experimental tests.

2. Material data and experimental setups

2.1. Materials

Twenty beam specimens were utilized to investigate the effect of reinforcement ratio on the flexural performance of alkali-activated fly ash based geopolymer beam. Before casting of geopolymer concrete, the fly ash was first used X-ray fluorescence (XRF) to determine the chemical composition and its type. The chemical composition of fly ash used in the current study is listed in Table 1. In accordance of ASTM-C-618-19 [45], the fly ash type under this study is classified as Class F fly ash since the summation of percentage chemical composition for Silicon dioxide (SiO2), aluminum oxide (Al2O3) and iron oxide (Fe2O3) is greater than 70% of the total composition. Furthermore, specific gravity test was conducted for the fly ash before mix design preparation and it was obtained as 2.67. Alkali solution from the mixture of eight mole sodium hydroxide (NaOH) and sodium silicate (Na2SiO3) was utilized. The sodium silicate solution has a composition of Na2O-SiO2 and H2O content of 15%, 30% and 55% by mass, respectively. Mix design is primarily prepared to obtain an appropriate sample to guarantee satisfactory properties of concrete. Table 2 shows the mix design utilized in the current study. Type D admixture was used in the mix and its mass is taken as 2% by mass of fly ash. Locally available crushed aggregate having 10 mm maximum aggregate size and river sand (fine aggregate) in the saturated surface dry condition were used to prepare concrete test specimens. For the development of mix design, the density of geopolymer concrete taken as 2400 kg/m³.

2.2. Tests conducted for the geopolymer concrete

To evaluate the engineering properties of hardened fly ash based geopolymer concrete and for the further use in finite element, mechanical properties such as compressive strength, tensile strength, flexural strength and stress-strain behavior tests were conducted. Three cylindrical specimens having a diameter of 10 cm were prepared and tested to obtain an average result to determine the compressive strength and splitting tensile strength. Curing condition is an important parameter for the strength gain of geopolymer concrete. Unlike Portland cement concrete, geopolymer concrete are not soaked in water for curing. Thus, all specimens were kept at room temperature for one day after casting. Then after one day, the specimens were demolded and cured for 28 days in a moist condition. All specimens (cylinders and beams) were tested at the age of 28 days. The compressive strength tests were carried out using Universal Testing Machine according to ASTM C 39/C 39M [46] specification. Prior of conducting compressive strength test, the specimens were capped to ensure a uniform loading surface. Indirect tensile strength tests were carried out according to ASTM C496/C496M [47] to determine the tensile strength of concrete.

Twenty flexural test beam coupons that are grouped based on the compressive strength of the geopolymer concrete were utilized to investigate the effect of reinforcement ratio on flexural performance of

### Table 1. Chemical composition of fly ash.

| Chemical Composition | SiO<sub>2</sub> | Al<sub>2</sub>O<sub>3</sub> | Fe<sub>2</sub>O<sub>3</sub> | TiO<sub>2</sub> | CaO | MgO | Cr<sub>2</sub>O<sub>3</sub> | K<sub>2</sub>O | Na<sub>2</sub>O | SO<sub>3</sub> | MnO<sub>2</sub> |
|----------------------|----------------|-----------------|-----------------|--------|-----|-----|-----------------|--------|--------|--------|--------|
| Result (%wt.)        | 48.47          | 26.05           | 12.54           | 0.92   | 5.2 | 2.8 | 0.02            | 1.66   | 0.47   | 1.1    | 0.19   |

### Table 2. The mix design used for the preparation of geopolymer concrete.

| Materials | Coarse Aggregate | Fine Aggregate | Fly ash | Na<sub>2</sub>SiO<sub>3</sub> | NaOH | Admixture |
|-----------|-----------------|---------------|--------|-----------------|------|----------|
| Mass (kg/m³) | 1080            | 720           | 390    | 150             | 60   | 7.8      |
the beams. The geometric dimensions and the compression reinforcement were kept constant for all beam specimens. In overall, sixteen beams were used for finite element analysis and the remaining four beams are tested experimentally. Specimen details of the reinforced geopolymer concrete beams, material properties and reinforcement utilized are presented in Table 3. The codes for the specimens given in Table 3 is based on beam number, number and diameter of tension reinforcement bar. For example, the label “BM1-2T10” can be interpreted as follows; BM1 means beam 1 and 2T10 stands for two tension reinforcement bar having diameter of ten, respectively.

Longitudinal reinforcement and stirrups used in the present study have a nominal diameter of 10 mm and 12 mm. The tensile strength test was conducted prior to casting the beam specimens using universal test machine at the laboratory of concrete, advanced materials and mechanical computation of Institut Teknologi Sepuluh Nopember, Indonesia. Three coupons of tensile test bar were prepared for each reinforcement bar types. An average result for each bar types from the uniaxial tensile test are presented in Table 4.

Table 3. Specimen details for the flexural test.

| Specimen | Cross section (L mm) | Compressive strength, f ″ck (MPa) | Longitudinal Reinforcement | Reinforcement ratio, ρ |
|----------|---------------------|----------------------------------|---------------------------|------------------------|
|          | L (mm)              | b (mm)                          | h (mm)                    | Tension                | compression            |
| BM1-2T10 | 1100                | 150                             | 150                       | 2p10                   | 2p10                   | 0.03                   |
| BM2-3T10*|                                                                  |                                  |                           | 3p10                   | 3p10                   | 0.045                  |
| BM3-3T10 |                                                                  |                                  |                           | 3p10                   | 3p10                   | 0.045                  |
| BM4-2T12 |                                                                  |                                  |                           | 2p12                   | 2p10                   | 0.042                  |
| BM5-3T12 |                                                                  |                                  |                           | 3p12                   | 2p10                   | 0.063                  |
| BM6-2T10 | 1100                | 150                             | 150                       | 2p10                   | 2p10                   | 0.03                   |
| BM7-3T10 |                                                                  |                                  |                           | 3p10                   | 2p10                   | 0.045                  |
| BM8-2T12 |                                                                  |                                  |                           | 2p12                   | 2p10                   | 0.042                  |
| BM9-3T12 |                                                                  |                                  |                           | 3p12                   | 2p10                   | 0.063                  |
| BM10-3T12|                                                                  |                                  |                           | 3p12                   | 2p10                   | 0.063                  |
| BM11-2T10| 1100                | 150                             | 150                       | 2p10                   | 2p10                   | 0.03                   |
| BM12-3T10|                                                                  |                                  |                           | 3p10                   | 2p10                   | 0.045                  |
| BM13-3T10|                                                                  |                                  |                           | 3p10                   | 2p10                   | 0.045                  |
| BM14-2T12|                                                                  |                                  |                           | 2p12                   | 2p10                   | 0.042                  |
| BM15-3T12|                                                                  |                                  |                           | 3p12                   | 2p10                   | 0.063                  |
| BM16-2T10| 1100                | 150                             | 150                       | 2p10                   | 2p10                   | 0.03                   |
| BM17-3T10|                                                                  |                                  |                           | 3p10                   | 2p10                   | 0.045                  |
| BM18-3T10|                                                                  |                                  |                           | 3p10                   | 2p10                   | 0.045                  |
| BM19-2T12|                                                                  |                                  |                           | 2p12                   | 2p10                   | 0.042                  |
| BM20-3T12|                                                                  |                                  |                           | 3p12                   | 2p10                   | 0.063                  |

* Experimentally tested beam.
** An average compressive strength from three specimens.

2.3. Test procedure and instrumentation for flexural test

The schematic representation of the geopolymer concrete beam and flexural setup is indicated in Figure 1. All beam specimens owning the 150 × 150 mm cross-section were tested under four-point loading. The overall length and shear span of the beam is measured as 1100 mm and 900 mm, respectively.

The flexural test specimen and experimental setup is prepared according ASTM C78/C78M-18 [48] specification. Three specimens were tested for each compressive class so that an average test result is taken for the report as mentioned in ASTM C78/C78M-18 specification. The load was applied to the concrete beam using a hydraulic jack through a high mechanical computation of Institut Teknologi Sepuluh Nopember, Indonesia. Three coupons of tensile test bar were prepared for each reinforcement bar types. An average result for each bar types from the uniaxial tensile test are presented in Table 4.

Three linear variable displacement transducers (LVDT) were mounted at the mid-bottom surface of the beam to record the vertical mid-span deflection during flexural test. Thus, the mid-span deflection is taken as an average result from these two LVDT.

3. Finite element analysis program

3.1. Finite element analysis program for modeling and loading conditions

Two-dimensional nonlinear finite element analysis was used to investigate reinforcement ratio on the flexural performance of beam specimens captioned in Table 3 by focusing on post ductile failure characteristics. All beam specimens are modeled as a plane stress/strain element. Total strain rotating crack model was assumed to model the concrete. According to previous study, the rotating model usually results in a lower limit failure load because it does not suffer as much from spurious stress locking as compared to the fixed model. The stress-locking phenomena present in the fixed crack model results in a considerable overestimation of the failure load [49]. For the modeling of longitudinal reinforcement and stirrups, a von mises plasticity model by incorporating plastic hardening was utilized. An embedded constraint type is utilized for the interaction between concrete and reinforcing bars. The finite element setup for the loading and boundary condition is depicted in Figure 2. The flexural load is applied in the form of prescribed deformation at the midpoint of the loading plate. Therefore, 0.1 mm prescribed deformation is applied to capture smooth load versus displacement curve.
3.2. Uniaxial behavior of concrete

For the simulation of the beam specimen, the concrete damage plasticity (CDP) model was used. This model utilizes the constitutive behavior of concrete by incorporating the scalar damage variables for compression and tension. The uniaxial tensile and compressive response of concrete reported by Alfarah et al. [50] was used as shown in Figure 3. This concrete damage model is chosen based on the previous publication.
The strain components in Figure 3(a), \( \varepsilon_{cm}^{ch} \) and \( \varepsilon_{cm}^{el} \), are the crushing and elastic undamaged components of strain; \( \varepsilon_{t}^{pl} \) and \( \varepsilon_{t}^{el} \) are plastic and elastic damaged strain components of the concrete. In Figure 3(b), \( \varepsilon_{t}^{ch} \) and \( \varepsilon_{t}^{el} \) are plastic and elastic damaged strain components. The ascending branches of the stress-strain curve in Figure 3(a) is presented according to model code recommendation [52]. The first branch up to 0.4\( f_{cm} \), is given by \( \sigma_{c(1)} = E_{0} \varepsilon_{c} \). The compression curve located between 0.4\( f_{cm} \) and \( f_{cm} \) in Figure 3(a) is given by Eq. (1). The modulus of concrete \( E_{ci} \), for the zero stress is calculated \( E_{ci} = 10^{4}(f_{ck})^{1.5} \) in MPa, and \( E_{0} \) is the undamaged elastic modulus of the concrete that extends up to 0.4\( f_{cm} \). The initial elastic modulus is calculated from \( E_{0} = (0.8 + 0.2f_{cm}/88)E_{ci} \) in MPa. Finally, the third segment in Figure 3(a) is calculated from Eq. (2) [50, 51, 53, 54]\( \varepsilon_{c} \):

\[
\sigma_{c(2)} = \frac{E_{ci} \varepsilon_{c}}{1 + \frac{\varepsilon_{t}^{pl}}{\varepsilon_{ch}} - \gamma_{c} + \frac{\varepsilon_{t}^{el}}{2\varepsilon_{cm}}} \varepsilon_{c} \varepsilon_{cm}^{2}/C_{0}^{2} + \gamma_{c} \varepsilon_{c}^{2}/C_{17} (1 - b) + b f_{cm} E_{0}/C_{18} \varepsilon_{cm}^{2}/C_{0}^{2} - \gamma_{c} f_{cm} E_{0}/C_{10}^{2} \varepsilon_{cm}^{2}/C_{0}^{2} + \gamma_{c} b f_{cm} E_{0}/C_{10}^{2} \varepsilon_{cm}^{2}/C_{0}^{2} (2)
\]

The value for \( \gamma_{c} \) is calculated from Eq. (3) and the term 'b' in Eq. (3) is computed from the ratio of plastic strain to crushing strain as, \( b = \varepsilon_{t}^{pl} / \varepsilon_{ch}^{c} \) [50, 51].

\[
\gamma_{c} = \frac{\pi^{2} f_{cm} E_{0}}{2 \varepsilon_{cm}^{2} (1 - b) + b f_{cm} E_{0}/C_{0}^{2}} (3)
\]

Table 5. Concrete material properties utilized for FEA.

| Material properties | Compressive strength, \( f_{ck} \) (MPa) | Young’s modulus, \( E \) (MPa) | Poisson’s ratio | Tensile strength, \( f_{t} \) (MPa) | Fracture energy in tension, \( G_{f} \) (J/m²) |
|---------------------|--------------------------------------|-----------------------------|----------------|-------------------------------|----------------------------------|
| 19.30               | 26,824                               | 0.077                       | 2.17           | 0.132                         | 20.95                            |
| 32.60               | 31,945                               | 0.077                       | 3.08           | 0.142                         | 24.74                            |
| 38.20               | 33,679                               | 0.077                       | 3.42           | 0.146                         | 26.54                            |
| 41.70               | 34,677                               | 0.077                       | 3.63           | 0.147                         | 27.69                            |

Figure 4. Strain analysis for beam corresponds to 38.20 MPa compressive strength.
The crushing energy is a function of concrete compressive strength, tensile strength, and the fracture energy. The crushing energy is calculated from
\[ G_{ch} = \frac{C_1 f_{cm}}{C_2 f_{tm}^2} G_F, \]
in which \( G_F \) is fracture energy and calculated from 
\[ G_F = 0.073 f_{0.18}^{0.5} \.
\] The term \( l_{eq} \) is the characteristic length that is usually dependent on the mesh size, the finite element type and the crack direction [53, 55]. The characteristic length is determined using the square root of the finite element area for two dimensional elements, [56]. In the case of three dimensional element, the characteristic length is determined from the cubic root of finite element [57]. Thus, the concrete material properties utilized under this study is summarized under Table 5.

### 4. Results and discussions

#### 4.1. Effect of reinforcement ratio on fracture characteristics

The fracture characteristics study of geopolymer beam is presented based on positive principal total strain (\( E_1 \)) and (\( E_{XX} \)) contour from finite element analysis. To overview the effect of reinforcement ratio on fracture behavior of the beam under this study, a representative sample was taken to illustrate the discussion. Thus, a beam specimen having a compressive strength of 38.20 MPa was used to study crack patterns in the specimen for different reinforcement ratio. The concrete strain is highly associated with the tensile reinforcement ratio, i.e., smaller...
reinforcement ratio led to higher strain growth in the concrete as shown in Figure 4. It can be clearly seen that from positive total strain depicted in Figure 4(a), specimens BM11-2T10 and BM14-2T12 that corresponds to a 0.03 and 0.042 reinforcement ratio does not satisfy a tension-controlled section criteria. The tension-controlled sections ensure that the longitudinal steel at tension zone yields well before the concrete crushes by providing sufficient ductility [58, 59, 60]. An open shear and extensive cracks were observed for specimens having reinforcement ratio of 0.03 and 0.042 at ultimate load. The dominant cracks at the tension zone extended up to the compression reinforcing bar, which further creates the diagonal crack on the top surface of the specimen as depicted in Figure 4(b). Apart from the shear crack noticed in specimen BM11-2T10 and BM14-2T12, a band of open diagonal cracks occurs around the free ends of the beam. Comparing specimen BM15-3T12 to other specimens, the formation of cracks is minimized. It was observed that the is no visible compression crack formed in this specimen, which implies neither compression softening nor crushing of concrete was observed.

4.2. Effect of reinforcement ratio on ultimate load resistance

The comparative investigation of load-displacement response from finite element analysis and experimental result is shown in Figure 5 for the beam specimens presented in Table 3. Usually, consumption of higher reinforcement ratio enhances the load carrying capacity, which can be observed in Figure 5(a)-(f). For the four concrete grades used under this study, an increase in tensile reinforcement enhances the ultimate load carrying capacity of the beam. By just keeping the reinforcement ratio constant, increasing the compressive strength of concrete from 19.30 MPa to 41.70 MPa, the load carrying performance of the concrete enhanced by about 12.20% for reinforcement ratio equals to 0.03. However, it has been observed that the load carrying capacity of the beam is significantly improved by increasing the reinforcement ratio. For instance, for the beam owing compressive strength of 32.60 MPa, increasing of reinforcement ratio from 0.03 to 0.063 yield an increase of ultimate load about 36.38%. This implies that increasing of reinforcement ratio within permissible limit helps to obtain an improved loading carrying capacity of geopolymer concrete beam rather than increasing the concrete strength. Recent work published by Sharma et al. [61] for the cracking and damage assessment in steel-reinforced concrete and glass fibre polymer-reinforced concrete beams with varying reinforcement ratio is a good reference for the result obtained under this study. According to Sharma et al., the increase in longitudinal reinforcement ratio improves the ultimate load carrying capacity even up to 42%.

As shown in Figure 5(a), it has been observed that the finite element analysis resulted stiffer behavior as compared to the experimental result from the load versus displacement plot for the branch earlier of cracking load. This phenomenon is substantial for low concrete grade. This phenomenon occurred due to the heterogeneous behavior of the concrete matrix. During the finite element simulation, the effect of shrinkage and hardening cracks of the actual experimental test were not considered. In addition, the interface between the reinforcing bars and the concrete matrix was considered as perfect bond in finite element analysis. The presence of aggregates and voids in the concrete matrix in actual condition obstructs the occurrence of perfect bond. Moreover, the FEA model follows the concrete damage model for Portland cement concrete since there is no clearly defined model for geopolymer concrete. In general, lower compressive strength of geopolymer concrete causes less tensile strength leading the early first cracks. The ductility behavior of the experimental beam is higher than the FEA model that is because the chemical bonding between the concrete matrix in the actual experimental specimen brings such improvement.

4.3. Validation to theoretical flexural strengths of beam

The comparative study was conducted between numerical and experimental test is depicted in Table 6. The validation study between numerical simulation and test result shows a 5.79%–10.08% deviation for the specimens with 0.045 and 0.063 reinforcement ratio value. It has been observed that the closeness of the result is dependent on the concrete strength. From Table 6, it can be clearly seen than the finite element and experimental test results are closer as compressive strength of concrete increases and vice versa. The deviation between the numerical simulation and the experimental test minimized when the compressive strength of concrete increases even for constant reinforcement ratio.

5. Conclusions

Under this study, twenty (sixteen FEA and four experimental) specimens were used to investigate the effect of reinforcement ratio in alkali activated fly ash (AAFA) based geopolymer concrete beams. To check the validity of the current study, four beam specimens were utilized to conduct a comparative study. From this study, the following conclusions were drawn:

- The ultimate load carrying capacity of alkali activated fly ash (AAFA) based geopolymer concrete beam is intensely affected by reinforcement ratio. It has been observed that the load carrying capacity of the beam significantly enhanced with the increase in reinforcement ratio by keeping the compressive strength of concrete constant. However, the flexural capacity enhancement of the beam was less significant by increasing the compressive strength of concrete as compared to reinforcement ratio.
- The finite element analysis showed that the provision of less reinforcement ratio results the crushing of concrete before tensile reinforcement yields. The result from the positive principal total strain (εL) and (εXY) contour depicted that the utilization of less reinforcement ratio yields extensive cracking in the tensile zone of concrete, which further develops the crushing of the concrete in compression zone.
- The result from the experimental test presented that increasing the compressive strength and longitudinal reinforcement ratio increases the ultimate load carrying capacity. But higher compressive strength and reinforcement ratio showed a tendency of sudden drop of load after ultimate load reached in the specimen.
- It has been observed that utilization of reinforcement ratio beyond 0.045 tends the beam to behave like brittle mode after the ultimate flexural carrying reaches. This phenomenon resulted less ductility in geopolymer concrete beam.
- In addition, experimental result shows that wide crack opening predominately occurs at the mid span of the beam and several cracks are not occurred near the mid span of the beam as it happens in Portland cement concrete.
- As observed from this study, utilization fiber for geopolymer concrete with high reinforcement ratio may result even an improved carrying capacity by bridging the occurrence of cracks in geopolymer concrete.
- The concrete damage model adopted in the finite element analysis of this study fairly estimates the experimental result. It was observed

| Specimen  | f_ck (MPa) | ρ  | Ultimate load (kN) | Deviation (%) | (P_{exp}/P_{FEA})/P_{FEA} |
|-----------|------------|----|--------------------|---------------|-----------------------------|
| BM3-2T10  | 19.30      | 0.045 | 77.4               | 69.6          | 10.08                       |
| BM12-3T10 | 38.20      | 0.045 | 84.6               | 78.15         | 7.67                        |
| BM17-3T10 | 41.70      | 0.045 | 86.19              | 81.2          | 5.79                        |
| BM9-3T12  | 32.60      | 0.063 | 97.27              | 88.40         | 9.11                        |
that as compressive strength of the concrete increases, the finite element analysis yields more precise result in estimating the maximum flexural load.

Declarations

Author contribution statement

Keflaye Zerfu, Januarti Jaya Ekaputri: Conceived and designed the experiments; Performed the experiments; Analyzed and interpreted the data; Contributed reagents, materials, analysis tools or data; Wrote the paper.

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Data availability statement

The data that has been used is confidential.

Declaration of interest’s statement

The authors declare no conflict of interest.

Additional information

No additional information is available for this paper.

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References

[1] S.K. Rahman, R. Al-Ameen, A newly developed self-compacting geopolymer concrete under ambient condition, Constr. Build. Mater. 267 (2021), 121822.
[2] X.D. Kangyin Dong, Hongdian Jiang, Renjin Sun, Driving forces and mitigation under ambient condition, Construct. Build. Mater. 267 (2021), 121822.
[3] P. Chindaprasirt, Workability and strength of coarse high calcium fly ash geopolymer concrete, Cem. concr. Compos. 29 (2007) 224–239.
[4] R. Shrestha, D. Baweja, K. Neupane, D. Chalmers, P. Sleep, Mechanical Properties of Geopolymer Concrete: Applicability of Relationships Defined by AS 3600, Constr. Inst. Aust. Conf., 2013.
[5] M. Chi, Effects of dosage of alkali-activated solution and curing conditions on the properties and durability of alkali-activated slag concrete, Constr. Build. Mater. 35 (2012) 240–245.
[6] N.A. Farhan, M.N. Sheikh, M.N.S. Hadi, Investigation of engineering properties of normal and high strength fly ash based geopolymer and alkali-activated slag concrete compared to ordinary Portland cement concrete, Constr. Build. Mater. 196 (2019) 26–42.
[7] P. Zhang, Z. Gao, J. Wang, J. Guo, S. Hu, Y. Ling, Properties of fresh and hardened fly ash/slag based geopolymer concrete: a review, J. Clean. Prod. 270 (2020), 122389.
[8] F.N. Okoye, J. Dargaprasad, N.B. Singh, Mechanical properties of alkali activated flyashes/kaolin based geopolymer concrete, Constr. Build. Mater. 98 (2015) 685–691.
[9] A. Erfanimaneh, M.K. Sharbatdar, Mechanical and microstructural characteristics of geopolymer paste, mortar, and concrete containing local zeolite and slag activated by sodium carbonate, J. Build. Eng. 32 (2020), 101781.
[10] M. Amran, A. Al-Fakh, S.H. Chu, R. Fedkiw, S. Haruna, A. Arzvedo, N. Vatin, Long-term durability properties of geopolymer concrete: an in-depth review, Case Stud. Constr. Mater. 15 (2021), e00661.
[11] Q. Meng, C. Wu, H. Hao, J. Li, P. Wu, Y. Yang, Z. Wang, Steel fibre reinforced alkali-activated geopolymer concrete slabs subjected to natural gas explosion in buried utility tunnel, Constr. Build. Mater. 246 (2020), 118447.
[12] N.A. Farhan, M.N. Sheikh, M.N.S. Hadi, Investigation of engineering properties of normal and high strength fly ash based geopolymer and alkali-activated slag concrete compared to ordinary Portland cement concrete, Constr. Build. Mater. 196 (2019) 26–42.
[13] A. Wardhono, C. Gunasekara, D.W. Law, S. Setungue, Comparison of long term performance between alkali activated slag and fly ash geopolymer concretes, Constr. Build. Mater. 143 (2017) 272–279.
[14] Q. Meng, C. Wu, H. Hao, J. Li, P. Wu, Y. Yang, Z. Wang, Steel fibre reinforced alkali-activated geopolymer concrete slabs subjected to natural gas explosion in buried utility tunnel, Constr. Build. Mater. 246 (2020), 118447.
[15] A. Castel, S.J. Foster, Bond strength between blended slag and Class F Fly ash geopolymer concrete with steel reinforcement, Cement Concr. Res. 72 (2015) 48–53.
[16] P.K. Sarker, Bond strength of reinforcing steel embedded in fly ash-based geopolymer concrete, Mater. Struct. 44 (2015) 1021–1030.
[17] A. Abidah, A. Altheeb, F. Alrshoudi, A. Abadel, H. Abbas, Y. Al-Salloum, Bond performance of GFRP and steel rebars embedded in metakaolin based geopolymer concrete, Structures 27 (2020) 1582–1593.
[18] P. Ramadass, Structure Concrete Combined effect of silica fume and steel fiber on the splitting tensile strength of high-strength concrete, Int. J. Civil. Eng. (2019) 96–103.
[19] B. Nematzadeh, J. Sanjayan, J. Qiu, E.H. Yang, High ductile behavior of a polyethylene fiber-reinforced one-part geopolymer composite: a micromechanics-based investigation, Arch. Civ. Mech. Eng. 17 (2017) 555–563.
[20] Keflaye Zerfu, Januarti Jaya Ekaputri, Review on alkali-activated fly ash based geopolymer concrete, Mater. Sci. Forum 841 (2016) 162–169.
[21] Biruk Haile Teka, Klaus Holschmacher, Alkali activated cement mixture at ambient curing: strength, workability, and setting time, Struct. Concr. J. Fib. (2021) 1-14.
[22] G. Fang, W.K. Ho, W. Tu, M. Zhang, Workability and mechanical properties of alkali-activated fly ash slag concrete cured at ambient temperature, Constr. Build. Mater. 172 (2018) 476–487.
[23] M. Kheradmand, Z. Abbodlalnejad, F. Fachecho-Torgal, Alkali-activated cement-based binder mortars containing phase change materials (PCMs): mechanical properties and cost analysis, Eur. J. Environ. Civ. Eng. 24 (2020) 1068–1090.
[24] K. Pasupathy, M. Berndt, J. Sanjayan, P. Rajeve, Durability of low - calcium fly ash based geopolymer concrete culvert in a saline environment, Cement Concr. Res. 100 (2017) 297–310.
[39] G. Arslan, E. Cihanli, Curvature ductility prediction of reinforced High-strength concrete beam sections, J. Civ. Eng. Manag. 16 (2010) 462–470.

[40] M.A. El Zareef, M.E. El Madawy, Effect of glass fiber rods on the ductile behaviour of reinforced concrete beams, Alex. Eng. J. 57 (2018) 4071–4079.

[41] B. Qin, A. Zhou, D. Lau, Effect of reinforcement ratio on the flexural performance of hybrid FRP reinforced concrete beams, Compos. B Eng. 108 (2017) 200–209.

[42] H.R. Chaboki, M. Ghalehnoori, A. Kartimipour, J. de Brito, Experimental study on the flexural behaviour and ductility ratio of steel fibres coarse recycled aggregate concrete beams, Construct. Build. Mater. 186 (2018) 400–422.

[43] S.A. Ashour, Effect of compressive strength and tensile reinforcement ratio on flexural behavior of high-strength concrete beams, Eng. Struct. 22 (2000) 413–423.

[44] M. Gümiş, A. Arslan, Effect of fiber type and content on the flexural behavior of high strength concrete beams with low reinforcement ratios, Structures 20 (2019) 1–10.

[45] ASTM standards (ASTM-C-618-19), Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use, 2019.

[46] ASTM Committee-C09, ASTM C.39/C.39M-01: Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens, ASTM Stand., 2014, pp. 3–9.

[47] ASTM Committee-C09, Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens, ASTM Stand., 2008, 545-545-3.

[48] C.C. Test, T. Drilled, C. Concrete, Standard test method for flexural strength of concrete (using simple beam with third-point loading) 1, in: ASTM C78/C78M-18, 2010, pp. 1–4.

[49] Jan Gerrit Rots, Computational Modelling of concrete Fracture, Delft University of Technology, 1988.

[50] B. Alfarah, F. Lopez-Almanna, S. Oliver, New methodology for calculating damage variables evolution in Plastic Damage Model for RC structures, Eng. Struct. 132 (2017) 70–86.

[51] K. Zerfu, J.J. Ekaputri, Nonlinear finite element study on element size effects in alkali-activated fly ash based reinforced geopolymer concrete beam, Case Stud. Constr. Mater. 15 (2021), e00765.

[52] Thomas Telford, CEB-FIP Mode Code-2010, 2010.

[53] W.B. Krautzig, R. Polling, An elasto-plastic damage model for reinforced concrete with minimum number of material parameters, Comput. Struct. 82 (2004) 1201–1215.

[54] V. Birtel, P. Mark, Parameterised finite element modelling of RC beam shear failure, Abaqus User’s Conf. (2006) 95–108.

[55] J. Oliver, A consistent characteristic length for smeared cracking models, Int. J. Numer. Methods Eng. 28 (1989) 461–474.

[56] K.M. Mosalam, G.H. Paulino, Evolutionary characteristic length method for smeared cracking finite element models, Finite Elem. Anal. Des. 27 (1997) 99–108.

[57] A.S. Genikomou, M.A. Polak, Finite element analysis of punching shear of concrete slabs using damaged plasticity model in ABAQUS, Eng. Struct. 98 (2015) 38–48.

[58] ACI Committee 318, Building Code Requirements for Structural Concrete (ACI 318M-99) and Commentary (ACI 318RM-99) Building Code Requirements and Commentary (ACI 318RM-99), 1999.

[59] British Standards Institution, Eurocode 2: Design of concrete Structures -Part 1-1: General Rules and Rules for Buildings, British Standards Institution, 2004.

[60] C.E. Orozco, Strain limits vs. reinforcement ratio limits - a collection of new and old formulas for the design of reinforced concrete sections, Case Stud. Struct. Eng. 4 (2015) 1–13.

[61] S. Sharma, S.K. Sharma, Gaurav Sharma, Crack Classification in Steel-RC and GFRP-RC Beams with Varying Reinforcement Ratio Using AE Parameters, Intech, 2012, p. 13.