Chapter 10
Experimental Study on Fly Ash and Ground Granulated Blast Slag-Based Geopolymer Corbels

B. Sumanth Kumar, V. Ramana Kollipara, and D. Rama Seshu

Abstract Concrete is the second most consumed material after water, due to phenomenal growth in infrastructure, trasportation, irrigation, and leads to huge consumption of cement as building material. Geopolymer concrete in this concern may prove to be a game-changing solution for the concrete industry as it has the potential to replace conventional cement in concrete. It also stresses on the utilization use of industrial wastes like fly ash and ground granulated blast slag (GGBS) to be used as binders due to their chemical action with alkaline solutions to produce inorganic molecule. Fifteen reinforced geopolymer concrete corbels with M20 grade of concrete and with different percentages of secondary reinforcement were cast and tested. The experimental shear strength at the interface of reinforced geopolymer concrete corbels obtained is compared with available analytical models and design codes applicable to the conventional concrete. The test results showed that the shear capacity of geopolymer concrete underrates based on OPC concrete models of corbels. The most analytical models are conservative in predicting shear capacity of GPC corbels. However, Hagberg (PCI: Precast. Prestressed Concrete Institute, 1983) and Eurocode 2 (Design of concrete structures – Part 1-1: General rules and rules for buildings. European Committee for Standardization, Avenue Marnix 17, B-1000 Brussels, Belgium. 225 p. (with corrigendum dated of 16 January 2008), 2004) predict better shear capacity of GPC corbels.

Keywords Geopolymer concrete corbels · Shear strength · Empirical approach · Shear friction · STM

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10.1 Introduction

Cement consumption amount will be increased manifold within a span of 10 years. But the raw material (limestone) which is used for the making of ordinary Portland cement will be in an acute shortage for the next 25–50 years. Besides, huge quantities of fly ash are being generated from thermal power plants for disposal, which has become a matter of concern. To overcome these issues, the employment of fly ash is increased as a supplementary cementing material along with GGBS and silica fume to develop substitute binders for conventional Portland cement [1, 2]. In this respect, geopolymer concrete with a relatively lower environmental impact holds great promise as a suitable alternative in the concrete industry [3–5].

Geopolymer concrete is designed by the alkali instigation of silicon (Si) and the aluminium (Al) in the source material such as fly ash, GGBS and metakaolin to produce binders. The process of polymerization leads to a 3D tetrahedral-bonded chain and ring structure consisting of Si-O-Al-O bonds; the size of the structure depends on the ratio of Si to Al. Generally, for the process of polymerization, the preferred alkalis are hydroxides of sodium or potassium, silicates of sodium or calcium, etc. [6, 7].

Even though the studies on mix design and physical properties of geopolymer concrete are more, the research on its application in structures is still scarce. And it is mostly constrained to precast construction only. In precast construction, connective distress is witnessed at the regions of well-defined planes called shear interfaces where longitudinal shear stresses may lead to sliding failure instead of diagonal tension failure along the well-defined plane, especially in structures like corbel, bearing shoe, etc. Therefore, the study of the behaviour and shear capacity of geopolymer concrete corbel section at column-corbel interface is vital in these circumstances.

10.2 Research Significance

The review of the literature indicates that in the case of corbels, load transfer is mainly through strut action rather than by simple flexure, that is, the basic assumption of plane section remaining plane before bending and after bending is not valid, and in order to establish the strut action, the shear span to effective depth ratio should be preferably equal to or less than 0.6. The non-linear stress behaviour of the corbel is influenced by the shear deformation in the elastic range. Hence, for the design concern, the shear strength at the interface of the section is becoming an important parameter. In conventional concrete, the ultimate strength of a corbel is calculated based on its dimensions, reinforcement ratio (ρ), strength of concrete (f_{ck})

and ratio of shear span to effective depth (a_v/d). The ultimate strength of corbel increases with the increase of reinforcement ratio and decreases with the increase of a_v/d ratio [8]. From the analysis of test results of corbel, it is observed that the horizontal stirrups that are provided in the corbel act as a tension reinforcement in resisting vertical loads.

In the case of corbels without shear reinforcement, diagonal tension cracks are formed, and a very sudden failure mostly occurs. This type of diagonal tension failure is generally referred as "diagonal splitting failure" [9]. In the case of corbels with horizontal stirrups, additional inclined cracks may occur usually beginning as a flexure crack at the horizontal face of the corbel and later on propagating as diagonal tension cracks towards the column-corbel interface.

A double corbel is a structural member in which brackets are projected from both the sides of a column. From structural point of view, double corbels are short cantilevers whose shear span to effective depth ratio is less than 1.0. The purpose of using double corbel for shear capacity study instead of single corbel is to reduce the moment coming at the interface section due to the application of vertical load with shear span and hence to ensure that the interface fails mostly in shear and not by any other mode (Fig. 10.1).

However, there is no significant literature on the behaviour of geopolymer concrete reinforced corbel section.

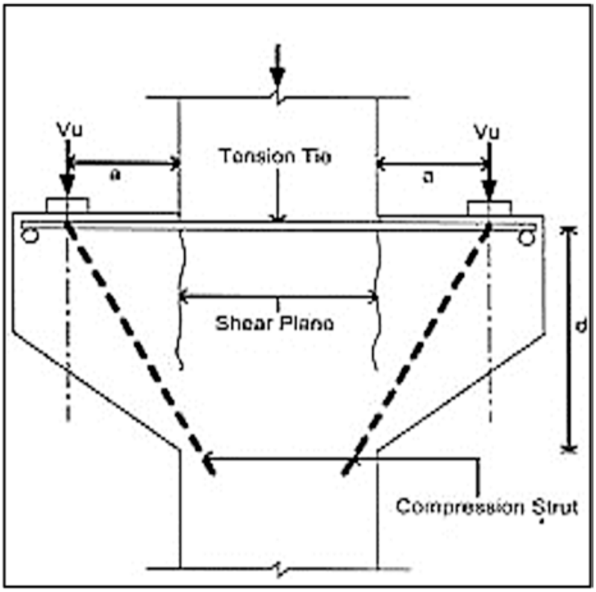


Fig. 10.1 Structural action of a column with double corbel

10.3 Experimental Programme

10.3.1 Materials Used

Fly ash: In this study, fly ash having a specific gravity of 2.25 and confirmed to IS: 3812 [10] is used for the experimental programme.

Ground granulated blast furnace slag (GGBS): The specific gravity of GGBS used is 2.90 and confirmed to IS: 12089 [11] (Table 10.1).

Alkaline solution: For the preparation of geopolymer concrete, alkaline solution of NaOH is used with the concentration of 8 moles/L. A constant ratio of silicate to hydroxide solution, which is equal to 2.5, is adopted, and the mixed solution is stored at room temperature (25 ± 2 °C) for 24 h before it is used for casting. Because of the dissolution of NaOH in water, heat is evolved, which can influence the concrete behaviour.

Water: In the experimental study, potable water is used for the preparation of alkaline solution.

Superplasticizer: Superplasticizer is used in the experiment based on sulphonated naphthalene polymers (Conplast SP430 Fosroc Make).

Aggregates: Crushed and angular aggregate of nominal size 20 mm is used as coarse aggregate with a specific gravity of 2.8, fineness modulus of 7.3 and water absorption capacity of 0.9%.

For fine aggregate, natural river sand of Zone II and confirming to IS: 383 [12] is adopted with a specific gravity of 2.65, fineness modulus of 3.35 and water absorption of 2%.

10.3.2 Mix Proportions

For the preparation of GPC corbel samples, the mix proportion is calculated based on the procedure given by G. Mallikarjuna Rao et al. [13], and the quantities are given in Table 10.2.

Table 10.1 Chemical composition of fly ash and GGBS (percent by mass)

Binder material	SiO_2	Al_2O_3	Fe_2O_3	SO_3	CaO	MgO	Na_2O	LOI
Fly ash	60.11	26.53	4.25	0.35	4.00	1.25	0.22	0.88
GGBS	37.73	14.42	1.11	0.39	37.34	8.71	–	1.41

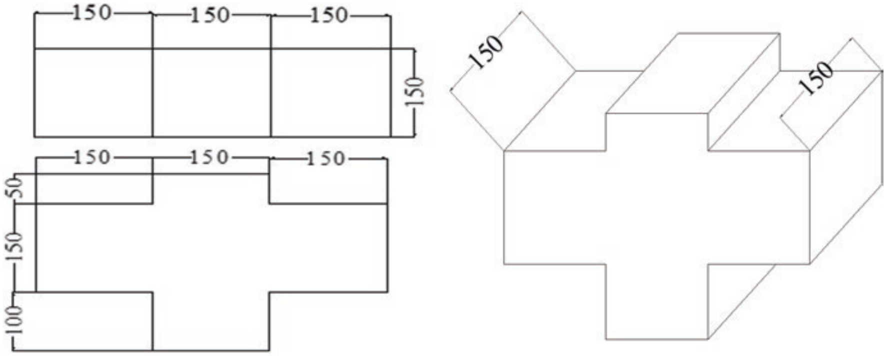
Table 10.2 Materials used in GPC (per m^3)

Grade of GPC	Materials						
	Coarse aggregate (kg)	Fine aggregate (kg)	Fly ash (kg)	GGBS (kg)	NaOH solution 8 molarity kg)	Sodium silicate (kg)	SP (kg)
A20	965	812	294	126	66	165	4.2

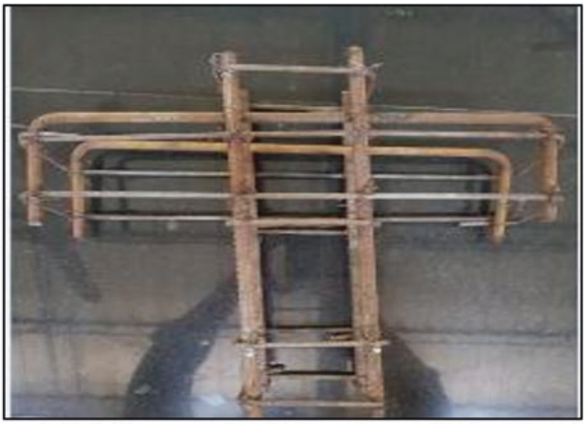
10.3.3 Mixing, Casting, Compacting and Curing of Double Corbel Samples

Fifteen (15) corbel samples had been cast of A20 grade of concrete. The sample consisted of a column of length 400 mm with two symmetric corbels projecting from its either side. Longitudinal reinforcement of diameter 10 mm with yield strength of 500 MPa and lateral ties of diameter 6 mm with yield strength of 250 MPa have been provided and spaced equally throughout the length of the column to ensure adequate reinforcements in compression. In the corbels, the main tension reinforcement of diameter 10 mm comprised parallel straight bars bent from the free end of one of the corbels passing through the longitudinal reinforcement of column and extending up to the free end of the other symmetric corbel.

Six (6) and three (3) samples were provided with interfacial shear reinforcement of mild steel with yield strength of 250 MPa with 0.53% (2nos–6 mm dia.) and 0.80% (3nos–6 mm dia.) in the form of closed rings enclosing the entire sample with columns and corbels, while the other six (6) samples did not contain any shear reinforcement across the shear plane. The detailing of reinforcement in corbel with shear stirrups is displayed in Fig. 10.2.



a. Corbel Specimen Details



b. Reinforcement details for column with double corbel

Fig. 10.2 (**a**) Corbel specimen details (**b**) Reinforcement details for column with double corbel

After 24 h, the samples are demoulded and allowed to air curing for a period of 28 days, subjected to a room temperature of 35 ± 2 °C. Before testing, V-groves of 4 mm deep were made on either side of the corbel samples along the shear plane.

The experimental study is limited to the M20 grade and also to 0–0.83% of shear reinforcement crossing the interface.

10.3.4 Testing of Double Corbel Samples

The test set-up of corbel samples is shown in Fig. 10.3. For convenience, the corbel samples had been tested in an inverted position. The corbels were supported on plain bearing free rollers resting on the top of the legs of the supporting wedge. The shear span to effective depth ratio was maintained as 0.46 for all the samples. The samples were loaded axially till failure which was confirmed by the emergence and development of a crack at the interface of column and corbel. The typical failure in the corbel samples is displayed in Figs. 10.4 and 10.5. The shear strength is calculated from the axial loads at failure, and Table 10.3 shows the values of shear strength and failure loads.

Fig. 10.3 Test set-up for column with double corbel

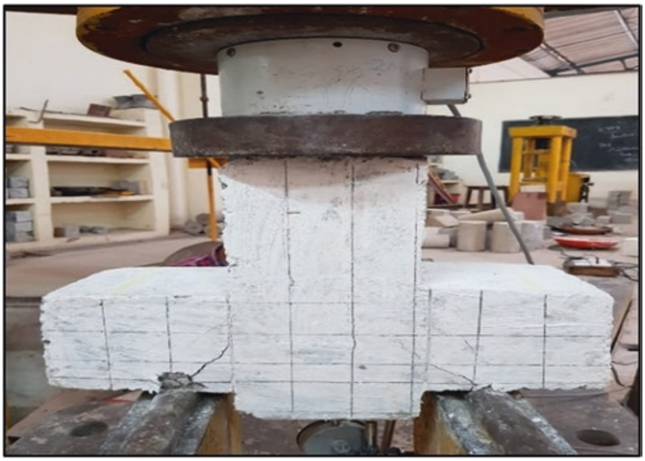


Fig. 10.4 Failure pattern for corbel without shear reinforcement

Fig. 10.5 Failure pattern for corbel with shear reinforcement

10.4 Results and Discussion

10.4.1 Failure Pattern of Unreinforced and Reinforced Samples

During the test, visible cracks were observed near the re-entrant corner of the column interface. With an increase in the load, several more inclined (shear) cracks were formed within the shear span and a little further from the interface. Failure was characterized by the expansion of one or more shear cracks associated with crushing

Table 10.3 The maximum shear force at the interface and corresponding shear stress at the interface of GPC corbels

S. No	Corbel ID	Compressive strength of GPC mix (f_{gpc}) (N/mm^2)	Closed loop stirrups (A_h) Details	A_h%	Maximum shear at interface (kN)	Shear stress (υ) (N/mm^2)	υ/f_{gpc}
1.	GCAS1-1	25.94	NA	0%	87.11	4.09	0.16
2.	GCAS1-2	26.07	NA	0%	84.91	3.99	0.15
3.	GCAS1-3	26.07	NA	0%	87.28	4.10	0.16
4.	GCAS1-4	26.07	NA	0%	85.71	4.02	0.15
5.	GCAS1-5	26.21	NA	0%	84.81	3.98	0.15
6.	GCAS1-6	26.56	NA	0%	88.37	4.15	0.16
7.	GCAS2-1	25.62	2nos.–6 mm dia.	0.53%	131.89	6.19	0.24
8.	GCAS2-2	25.62	2nos.–6 mm dia.	0.53%	134.88	6.33	0.25
9.	GCAS2-3	25.62	2nos.–6 mm dia.	0.53%	130.70	6.14	0.24
10.	GCAS2-4	25.94	2nos.–6 mm dia.	0.53%	133.83	6.28	0.24
11.	GCAS2-5	26.21	2nos.–6 mm dia.	0.53%	132.59	6.23	0.24
12.	GCAS2-6	26.56	2nos.–6 mm dia.	0.53%	134.88	6.33	0.24
13.	GCAS3-1	25.94	3nos.–6 mm dia.	0.80%	183.07	8.59	0.33
14.	GCAS3-2	26.21	3nos.–6 mm dia.	0.80%	185.81	8.72	0.33
15.	GCAS3-3	26.56	3nos.–6 mm dia.	0.80%	189.26	8.89	0.33

concrete near the intersection of the corbel and the column. In the absence of horizontal stirrups, the formation of cracks was sudden and resulted in wider diagonal cracks. However, the provision of horizontal stirrups made the diagonal cracks propagate slowly towards the column-corbel interface. Further, the width of diagonal cracks in stirrup reinforced corbels was small compared to that of corbels with no stirrup reinforcement. Testing of specimen was stopped at the point where load could no longer be sustained. There were no signs of cracks/crushing in the column portion observed. Similar failure pattern was observed by the previous investigators (Mattock et al. [8], Kriz and Raths [9]) on RC corbels.

While in corbels reinforced with principal tension and secondary shear reinforcements, numerous cracks could be witnessed, and failure was almost beam shear failure, which was characterized by the opening of one or more diagonal tension cracks while the flexure cracks remained fine followed by shear failure in the compressed zone of the strut at the column-corbel interface. It had also been noticed that in the case of non-reinforced samples, the cracks from the supporting areas had begun to emerge at about 65% of their ultimate capacity while in the case of reinforced samples, these cracks have grown at about 45% of their ultimate capacity. Hence, it may be concluded that the reinforced samples could sustain more loads and horizontal stirrups were sufficient to prevent premature diagonal tension failure and allow the main tension reinforcement to take its potential strength before ultimate failure had been reached.

10.4.2 Comparison of Experimental Shear Capacity with Theoretical Capacity

Load-carrying capacity of reinforced concrete corbels can be evaluated by several theories like shear friction theory, truss analogy (strut-and-tie method), geometrical method of force distribution and theory of plasticity. Few design codes considered shear friction theory for evaluating shear capacity of reinforced corbels along with strut-and-tie methodology. Numerous investigations proposed strut-and-tie methodology in calculating shear capacity of corbels. Table 10.4 presents the load-carrying capacity of reinforced corbels as per different investigators/codes of practice on ordinary Portland cement concrete.

To compare the shear transfer capacity of GPC corbel with conventional concrete, the theoretical shear capacity of the corbel section has been obtained from the shear strength expressions as mentioned in Table 10.4. Comparison of experimental shear capacity of GPC reinforced corbels with the shear strength predicted by the design codes/equations is presented in Table 10.5.

From Table 10.5, it may be observed that the interface shear capacity of conventional concrete, when used for GPC, underestimates the shear capacity and same was observed during experimental study on corbels on GPC. The comparison shows that the shear capacity obtained from different theories and codes are varying from 44% to 87% more than the experimental shear strength of geopolymer reinforced corbels. In general, the shear strength obtained based on strut-and-tie models is less conservative than the shear strength obtained from shear friction models. Hagberg [15] and Eurocode 2 seem to give better prediction of shear strength of GPC corbels.

Table 10.4 Load-carrying capacity of reinforced corbels as per different investigators/codes of practice on conventional concrete

Reference	Shear strength expression	Remarks
Kriz and Rath [9]	$V_u = \varphi bd\sqrt{f_c} F_1 F_2$ $F_1 = 6.5\left(1 - 0.5^{\frac{d}{a}}\right)$ $F_2 = \frac{(1000\rho)^{\left(\frac{1}{3}+\frac{0.4H}{V}\right)}}{10^{\frac{0.8H}{V}}}$	Empirical approach based on the experimental work
ACI 318 [14] Cl. 16.5	Shear friction strength – $V_u = \varnothing \mu A_{vf} f_y$ Flexural strength – $V_u = \frac{M_u}{a}$ $M_u = \varnothing \mu f_y A_{sm}\left(d - \frac{a}{2}\right)$ $a = \frac{A_f f_y}{0.85 f_c' b}$ Maximum or permissible shear strength – $V_u = 0.2 f_c' bd$ or 5.5bd or $(3.31 + 0.08\ f_c')bd$	Based on shear friction methodology. Strut-and-tie model was discussed in Chap. 23 which is similar to *PCI Handbook 2010*

Reference	Shear strength expression	Remarks
CSA A23.3 [18] Cl. 11.5	$\upsilon_u = \phi_c \lambda(c + \mu\sigma) + \Phi_s \rho_v f_y \cos \alpha_f$ $c = 1$; $\mu = 1.4$ for monolithic concrete. $\lambda = 1$ for normal density concrete; $\lambda(c+\mu\sigma) \leq 0.25 f_c'$, $\rho_{vmin} = 0.06\sqrt{\frac{f_c'}{f_y}}$	Based on shear friction methodology
	$\upsilon_u = \lambda k \sqrt{\sigma f_c'} + \rho_v f_y \cos\alpha_f$ $\lambda k \sqrt{\sigma f_c'} \leq 0.25 f_c'$ $k = 0.6$ for concrete placed monolithically	
PCI [16] 7th edition Cl. 5.9.4	Deriving from Fig. 5.9.4 of PCI Handbook [16] and Araújo et al. (2016) $V_d = \left(\sqrt{(1.7\gamma\beta b f_c a)^2 + 6.8 A_s f_y d \gamma^2 \beta b f_c} - 1.7\gamma\beta b f_c a\right)/2$	Based on strut-and-tie model. Shear friction methodology is similar to ACI 318 [14] γ = strength reduction factor = 0.75 β = 0.6 for no stirrups else 0.75
Hagberg [15]	$V_{max} = f_c \cos\beta$ $\left[1 - \frac{2 f_c b d}{F_s}\right]\tan^2\beta + \left[\frac{2 f_c b d}{F_s}\right]\tan\beta + 1 = 0$ $F_s = F_{s1} + F_{s2}$ $F_{s1} = A_f f_y$, $F_{s2} = A_h f_y$ $d = \frac{d_1 F_{s1} + d_2 F_{s2}}{F_s}$	Based on strut-and-tie model
EN 1992 -1-1 [17] Section J 3	Deriving from figure J 5 of Eurocode 2 [17] and Araújo et al. (2016) $V_d = (\sqrt{\left(abk_1\left(1 - \frac{f_c}{250\times10^6}\right)\frac{f_c}{\gamma}\right)^2 + 1.6 bdA_s f_y k_1\left(1 - \frac{f_c}{250\times10^6}\right)\frac{f_c}{\gamma}} - abk_1\left(1 - \frac{f_c}{250\times10^6}\right)\frac{f_c}{\gamma}$	Based on strut-and-tie model. K1 = 1.18, γ = strength reduction factor = 0.75

Table 10.5 Comparison of experimental shear capacity of GPC reinforced corbels with the shear strength predicted by the design codes/equations

Spec. ID	f_{gpc} N/mm^2	Vup kN	Ref a Vu1 kN	Vup/Vu1	Ref b Vu2 N	Vup/Vu2	Ref c Vu3 kN	Vup/Vu3	Ref d Vu4 kN	Vup/Vu4	Ref e Vu5 kN	Vup/Vu5	Ref f Vu6 kN	Vup/Vu6
GCAS1-1	25.94	87.11	67.63	1.29	82.47	1.06	85.32	1.02	86.07	1.01	83.49	1.04	94.58	0.92
GCAS1-2	26.07	84.91	67.80	1.25	82.47	1.03	85.32	1.00	86.19	0.99	83.60	1.02	94.68	0.90
GCAS1-3	26.07	87.28	67.80	1.29	82.47	1.06	85.32	1.02	86.19	1.01	83.60	1.04	94.68	0.92
GCAS1-4	26.07	85.71	67.80	1.26	82.47	1.04	85.32	1.00	86.19	0.99	83.60	1.03	94.68	0.91
GCAS1-5	26.21	84.81	67.98	1.25	82.47	1.03	85.32	0.99	86.31	0.98	83.73	1.01	94.79	0.89
GCAS1-6	26.56	88.37	68.43	1.29	82.47	1.07	85.32	1.04	86.61	1.02	84.03	1.05	95.06	0.93
GCAS2-1	25.62	131.89	80.52	1.64	109.14	1.21	88.68	1.49	112.00	1.18	123.06	1.07	116.39	1.13
GCAS2-2	25.62	134.88	80.52	1.68	109.14	1.24	88.68	1.52	112.00	1.20	123.06	1.10	116.39	1.16
GCAS2-3	25.62	130.70	80.52	1.62	109.14	1.20	88.68	1.47	112.00	1.17	123.06	1.06	116.39	1.12
GCAS2-4	25.94	133.83	81.03	1.65	110.50	1.21	89.78	1.49	112.27	1.19	123.57	1.08	116.61	1.15
GCAS2-5	26.21	132.59	81.45	1.63	111.65	1.19	90.72	1.46	112.50	1.18	123.99	1.07	116.80	1.14
GCAS2-6	26.56	134.88	81.99	1.65	112.15	1.20	91.93	1.47	112.79	1.20	124.53	1.08	117.03	1.15
GCAS3-1	25.94	183.07	86.32	2.12	110.50	1.66	89.78	2.04	122.87	1.49	144.70	1.27	127.63	1.43
GCAS3-2	26.21	185.81	86.77	2.14	111.65	1.66	90.72	2.05	123.10	1.51	145.23	1.28	127.80	1.45
GCAS3-3	26.56	189.26	87.35	2.17	113.15	1.67	91.93	2.06	123.39	1.53	145.90	1.30	128.02	1.48
Average			1.09		1.87		1.77		1.80		1.58		1.44	

Notations:

f_{gpc} – average compressive strength (N/mm^2), V_{up} – experimental shear strength of corbel (kN), V_u – ultimate shear strength (kN)
a. Kriz and Rath [9], b. ACI 318 [14], c. CSA A23.3-04, d. PCI [16], e. Hagberg (2013), f. EN 1992-1-1 [17]

10.5 Conclusions

The following are the conclusions arrived after the study of shear capacity of GPC corbels:

1. The ultimate load capacity of corbels increased with the increase in the compressive strength of GPC.
2. The ultimate load of corbels was increased by the increase in percentage of closed loop stirrups (secondary reinforcement).
3. The shear capacity as obtained from different codes and theories are underestimating the interface shear capacity of reinforced GPC corbels.
4. Hagberg [15] and Eurocode 2 [17] predict better shear capacity of geopolymer concrete corbels.

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