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ARTICLE

An experimental study on the interface shear strength of reinforced geopolymer concrete corbels

Sumanth Kumar B[a] and Rama Seshu D[b]

[a]Department of Civil Engineering, Kakatiya Institute of Technology and Science, Warangal, India; [b]Department of Civil Engineering, National Institute of Technology Warangal, Warangal, India

ABSTRACT
Geopolymer concrete is gradually assuming significance from the point of sustainability in the concrete industry. The reinforced concrete corbels are used for supporting the precast beams. This paper presents an experimental investigation on the interface shear strength of reinforced geopolymer concrete corbels. A total of forty-five symmetric double cantilever reinforced GPC corbels were cast and tested. The parameters of the study include the compressive strength of GPC and the percentage of secondary reinforcement i.e. closed loop ties crossing the interface. 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 results of the study indicated that the interface shear capacity of geopolymer concrete was evaluated based on conventional concrete analytical models that underestimates the shear capacity of GPC corbels. Further the experimental shear strengths of corbel are about 9% higher than the predicted interface shear strength of GPC corbels using the analytical expression proposed in this paper.

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KEYWORDS
Geopolymer concrete corbels; shear strength; empirical approach; shear friction; STM

1. Introduction

The phenomenal growth in the infrastructure coupled with the depleting natural resources has forced the concrete industry to look for sustainable alternative to the conventional concrete. In this regard, the Geopolymer concrete (GPC), is seen as one of the sustainable alternatives that help in reducing partially the huge consumption of conventional concrete. GPC is produced using the locally available industrial wastes/by-products that are rich in silicon and alumina.

In the recent past, several investigations reported various parameters affecting the strength of GPC. These parameters include the quantity of source material, activator to binder ratio, molarities of activator solution (Hardjito et al. (2005), Zende and Mamatha (2015) and Mallikarjuna Rao and Rao (2015)). The focus of these studies has been mainly based on material characterisation, physical and chemical properties and the associated polymerisation reaction, mix-proportioning of geopolymer concrete etc. There are limited studies on the structural performance of GPC in structural members where stress is concentrated in interfacial or connection zones, such as areas associated with corbels, near column-beam joints, beam-to-floor joints, etc. Corbels in general are the structural elements used to support primary beams and girders and are most commonly used in precast concrete connections. Extensive research have been done in various aspects of corbelsmade using normal concrete, high strength concrete (Kriz and Raths (1965), Mattock (1976), Ahmed, Diab, and Drar (2012), Mehdi Rezaei, Osman, and Shanmugam (2012)). Based on the published investigations, it can be understood that the behaviour of corbels depends on the type and direction of loads, shear span to depth ratio (a/d) ratio (Figure 1), strength of concrete, shape and dimensions of corbels, grade and arrangement of longitudinal and transverse steel reinforcement. Due to significant shear deformation, the corbels experienced ifferent failure modes, which include flexural tension, flexural compression, diagonal splitting and shear failure based on shear span to depth ratio (Kriz and Raths (1965), Mattock (1976)). The shear capacity prediction models of reinforced concrete corbels that are proposed in the literature were based on shear friction theory, purely empirical approach and strut and tie (STM) model (Yassin and Hasan (2015), Dawood, Kadhum, and Abdul-Razzaq (2018). Kriz and Raths (1965) adopted empirical approach based on a statistical fit, which are functions of parameters strength of concrete and a/d ratio. However, this approach does not include the strength of reinforcement steel. Mast (1968) applied shear friction hypothesis to the experimental data of Kriz and Raths (1965). Figure 2 illustrates the analogy of shear friction hypothesis. The external shear force is resisted by the frictional and cohesive forces along the shear crack.

CONTACT Rama Seshu D drseshu@nitw.ac.in Department of Civil Engineering, National Institute of Technology Warangal, Warangal, India

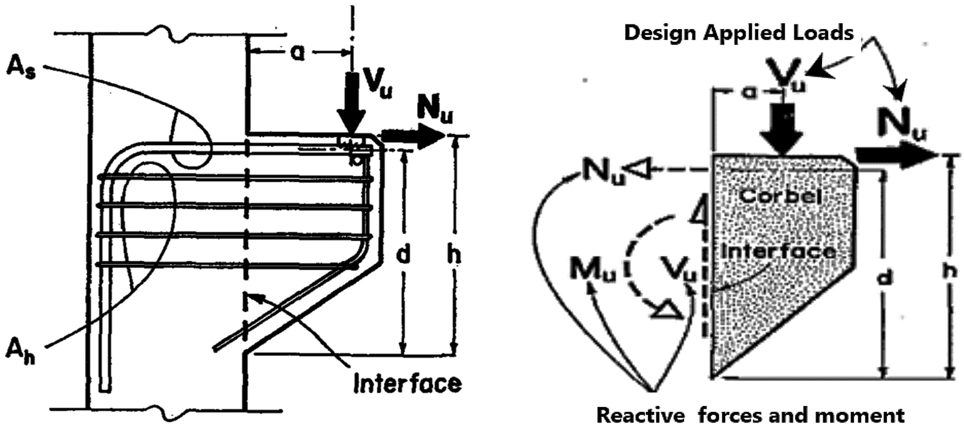


Figure 1. Typical corbel and free body force diagram (Mattock 1976).

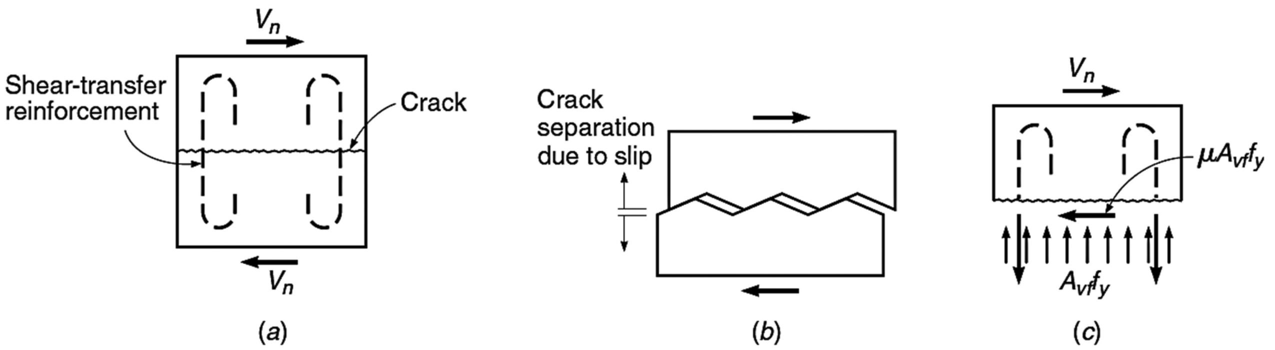


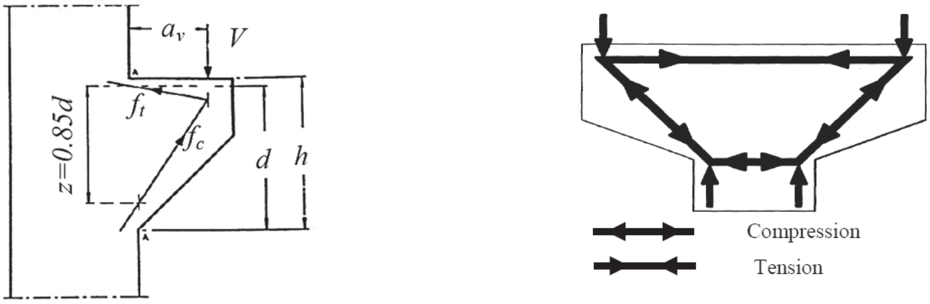
Figure 2. Shear-Friction design method: a) Applied shear b) Enlarged representation of crack surface c) Free-Body sketch of concrete above crack (Nilson, Darwin, and Dalon 2004).

The slippage and subsequent movement of the concrete along the crack will increase the tension in the reinforcement crossing the shear crack. The resulting tensile force creates equal and opposite pressure between concrete surfaces on either side of the crack. This can be seen from the free body sketch in Figure 2 (c) that the maximum value of this interfacial pressure is equal to $A_{vf}f_y$, where A_{vf} is the total area steel crossing the crack, and f_y is its yield strength.

Later Mattock (1976) estimated by static equilibrium of vertical and horizontal forces (flexural model) along with shear friction approach i.e. the reactive forces V_u and N_u must be equal to the design vertical and horizontal loads, V_u and N_u, respectively. In addition, the reactive moment M_u must be equal to (from Figure 1) $V_u a + N_u(h - d)$.

Hagberg (1963) proposed a simplified design of corbels using the strut and tie method (also known as Truss analogy). In this approach, tension and compression zones form in corbels, which act as a truss. Steel ties are placed in tension zones to resist tensile forces and compressive struts are represented by the concrete in between the cracks. Figure 3 presents mechanism of force transfer considered in the STM. For corbels with short span-to-depth ratios, a large portion of the applied vertical shear force is directly transferred to the supporting columns or walls through inclined strut, with the formation of a full-length horizontal tie to balance the thrust of the inclined struts. Design shear strengths of corbels are evaluated by considering yielding of steel at tension zone and cracking resistance of concrete in compressive struts.

Despite the advantages like better strengths, durability etc., for geopolymer concrete when compared to conventional concrete, the use of geopolymer concrete in practice is significantly limited. This is mainly due to a lack of research in terms of behaviour of structural elements made using GPC and the related design and applications. This paper presents the use of Geopolymer concrete in corbels and the application of shear friction concept in predicting the shear capacity of GPC corbels.



Truss Analogy (Franz and Niedenhoff, 1963) **Refined Strut and Tie Model (Siao, 1994)**

Figure 3. Typical STM or truss analogy diagram for the design of concrete corbels.

2. Review of analysis approaches for the design of corbels

The review of literature indicated the requirement of studies on shear strength and shear transfer characteristics of GPC since the research work was carried out on conventional concrete. Particularly, the suitability of different theories such as shear-friction theory in the prediction of shear transfer characteristics of GPC. Shear capacity at interface in general depends on parameters like roughness of interface, amount of reinforcement crossing the interface or shear reinforcement and strength of concrete. The authors in their earlier experimental investigation on GPC push off specimens have presented the expression for the shear strength of GPC that takes in to account the influence of three shear load carrying mechanisms(Randl 1997)i.e. Cohesion (due to interlocking between aggregates), friction (due to slip among different concrete layers and is effected by normal stress and roughness at the interface) and the dowel resistance of steel connectors i.e. dowel action (due to presence of reinforcement crossing the interface) (Kumar 2021). The expression for the shear strength of monolithic GPC interface (V_u) was proposed as follows:

$$V_u = V_c + V_f + V_d \tag{1}$$

Where,

V_c	-	Shear strength of unreinforced GPC due to cohesion, $V_c = c*(f_{gpc})^{(1/3)}*bh$, where For $f_{gpc} \leq 40$, $c = 0.031\ f_{gpc}+0.06$, For $f_{gpc}>40$, $c = 0.0054f_{gpc}+1.0809$
V_f	-	Shear strength of reinforced GPC due to friction, $V_f = \mu[\sigma_n+\rho k f_y]*bh$, where $k = 0.5$ and For $f_{gpc} \geq 20$MPa, $\mu = 0.8$, For $f_{gpc} \geq 35$MPa, $\mu = 1.0$, $\rho = \rho_{main} + \rho_{Stirrups}$ (Randl (1997))
V_d	-	Shear strength due to dowel contribution, $V_d = \alpha\rho\sqrt{f_y f_{gpc}}bh$, $\rho = \rho_{Stirrups}$ where $\alpha = 6.338\rho\sqrt{f_y f_{gpc}}$

V_u- Ultimate longitudinal shear stress at the interface;

c – Coefficient of cohesion;

μ – coefficient of friction;

ρ_{main} – reinforcement ratio provided for flexural design across the interface;

$\rho_{stirrups}$- reinforcement ratio provided for shear design across the interface;

k – coefficient of efficiency for shear reinforcement to transmit the tensile force;

f_{gpc}- characteristic value of geopolymer concrete compressive strength;

f_y – characteristic value of yield strength of the reinforcement;

σ_n- normal stress at the interface due to external loading;

α – coefficient for dowel action (flexural resistance of reinforcement);

bh – shear area at the interface.

In the present study, the shear carrying capacity of geopolymer concrete corbels is investigated with the following objectives:

(1) To study the behaviour and the failure pattern of reinforced geopolymer concrete corbels.
(2) To study the shear carrying capacity of geopolymer concrete corbels and.
(3) To compare the experimental shear strength of corbels with the proposed shear equation for fly ash and GGBS based geopolymer concrete.

Also, the experimentally determined shear strength of corbels is compared with the shear capacity evaluated using the shear strength expressions of conventional concrete, as mentioned in Table 1.

The parameters of experimental investigation are:

(a) Compressive strength of GPC – Three different strengths- B (20–25 MPa) – 15 corbels, C (40–45 MPa) – 15 corbels, D (50–55 MPa) – 15 corbels.
(b) Three different percentages of secondary reinforcement (A_h) crossing the monolithic interface of GPC corbel, in the form of horizontal stirrups – 0% – 18 corbels, 0.53% – 18 corbels and 0.80% – 9 corbels of the cross-sectional area at the monolithic interface.

Table 1. Load carrying capacity of reinforced corbels as per different investigators/codes of practice on conventional concrete.

Reference	Shear strength expression	Remarks/Design Approach
Kriz and Raths (1965)	$V_u = \varphi \mathrm{bd}\sqrt{f_c}F_1F_2F_1 = 6.5\left(1-0.5^{\frac{d}{a}}\right)F_2 = \frac{(1000\rho^{\left(\frac{1}{3}+\frac{0.4H}{V}\right)}}{10^{\frac{0.8H}{V}}}$	Empirical approach based on the experimental work.
ACI	Committee 318 (2014)Cl. 16.5	Based on shear friction strength – $V_u = \emptyset \mu A_v f_y$Based on flexural strength –
	$V_u = \frac{M_u}{a} M_u = \emptyset \mu f_y A_{sm}\left(d-\frac{a}{2}\right) a = \frac{A_f f_y}{0.85 f_c' b}$Maximum or permissible shear strength – $V_u =$ $0.2f_c' bd$ or 5.5bd or $(3.31 + 0.08f_c')bd$For STM – Refer PCI Handbook	Based on shear friction methodology. Strut and Tie model was discussed in chapter 23 which is similar to PCI Handbook 2010.
CSA A23.3 (2014)Cl. 11.5	$\upsilon_u = {}_c\lambda(c+\mu\sigma) + \Phi_s \rho f_y \cos a_f c = 1$; μ = 1.4 for monolithic concrete.λ = 1 for normal density concrete;$\lambda(c+\mu\sigma) \leq 0.25f_c'$, $\rho_{vmin} = 0.06\sqrt{\frac{f_c'}{f_y}}$	Based on shear friction methodology.
	$\upsilon_u = \lambda k\sqrt{\sigma f_c'} + \rho_v f_y \cos a_f \lambda k\sqrt{\sigma f_c'} \leq 0.25 f_c' k = 0.6$ for concrete placed monolithically.	
PCI (2010), 7th	editionCl. 5.9.4	For shear friction model and flexural strength – Refer ACI 318Deriving from figure 5.9.4 of PCI handbook – 2010 and Araújo, D.L et al
	(2016)$V_d = \left(\sqrt{(1.7\gamma\beta b f_c a)^2 + 6.8A_s f_y d\gamma^2\beta b f_c} - 1.7\gamma\beta b f_c a\right)/2$	Based on Strut and Tie Model.Shear Friction methodology is similar to ACI Committee 318 (2014) γ = strength reduction factor = 0.75β = 0.6 for no Stirrups else 0.75
Hagberg (19863)	$V_{max} = f_c \cos\beta\left[1-\frac{2f_c bd}{F_s}\right]\tan^2\beta + \left[\frac{2f_c bd}{F_s}\right]\tan\beta + 1 = 0 F_s = F_{s1} + F_{s2} F_{s1} = A_s f_y,$ $F_{s2} = A_h f_y d = \frac{d_1F_{s1}+d_2F_{s2}}{F_s}$	Based on Strut and Tie Model.
EN	1992 – 1-1 (2004)Section J 3	Deriving from figure J 5 of Euro code 2–2004 and Araújo et al.
	(2016)$V_d = (\sqrt{\left(abk_1\left(1-\frac{f_c}{250\times10^6}\right)\frac{f_c}{\gamma}\right)^2 + 1.6bdA_s f_y k_1\left(1-\frac{f_c}{250\times10^6}\right)\frac{f_c}{\gamma}} - abk_1\left(1-\frac{f_c}{250\times10^6}\right)\frac{f_c}{\gamma}$	Based on Strut and Tie Model.K_1 = 1.18, γ = strength reduction factor = 0.75

For each variation, there are 3 (or) 6 identical specimens and numbered as … 1,2,3,4,5,6. Hence, each GPC corbel is designated by its mix strength, percent of stirrups and the identical specimen number. For example, in the corbel specimen designation GCBS1-3: GC indicates GPC Corbel, B indicates the corbel was cast using B type mix, S1 indicates the corbel is provided with 0% percent of secondary reinforcement (A_h) in the form ofclosed loop Stirrups crossing the interface and 3-indicates the identical specimen number.

In this investigation, the primary reinforcement (Main tension flexural tension reinforcement (A_s)) and shear span to depth (a/d) are kept constant in all the tested corbels so as to avoid crushing of column and failure in flexure.

3. Experimental program

3.1. Materials used

Fly ash and GGBS are used as binders from NTPC power plant, Ramagundam, India and JSW Pvt ltd, Bilakalagudur, India, respectively. The specific gravity is 2.90 and 2.17 for GGBS and fly ash, respectively. Table 2 shows the details of chemical compositions.

Fine aggregate of river sand conforming to Zone-2 of IS: 383 (2016) was used as fine aggregate. The specific gravity and bulk density of sand are 2.65 & 1.45 g/cm^3, respectively. Well-graded aggregate coarse aggregate conforming to IS: 383 (2016) with 20 mm nominal size of granite is used as coarse aggregate.2.80 & 1.50 g/cm^{3}are specific gravity and bulk density, respectively.

Potable water was used in the experimental work. Alkaline solution consists of sodium silicate solution to sodium hydroxide solution (8 Molarity) with 2.5:1. The alkaline solution is stored at room temperature (25 ± 2°C) and relative humidity of 65% for 24 h before using it in the casting of GPC Corbel specimens.

3.2. Mix proportions

The GPC Mix proportions considered were based on work done by Mallikarjuna Rao and Rao (2015). The materials used per cubic metre of GPC are given in Table 3.

3.3. Casting and curing of GPC corbels

The dimensions of the Symmetrical Double Corbel (SDC) specimens used for testing are shown in Figure 4. A total of 45 numbers of corbels were cast and tested. Corbels have been cast with three different

Table 2. Chemical composition of fly ash and GGBS (% 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 3. Details of GPC mix.

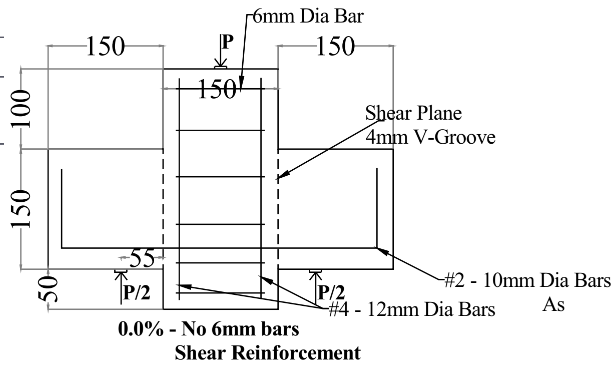


Figure 5. (a) SDC specimen reinforcement with 0.00% shear reinforcement (b) SDC specimen reinforcement with 0.53% shear reinforcement (c) SDC specimen reinforcement with 0.80% shear reinforcement (d) load scheme adopted (e) Corbel mould and reinforcement configuration.

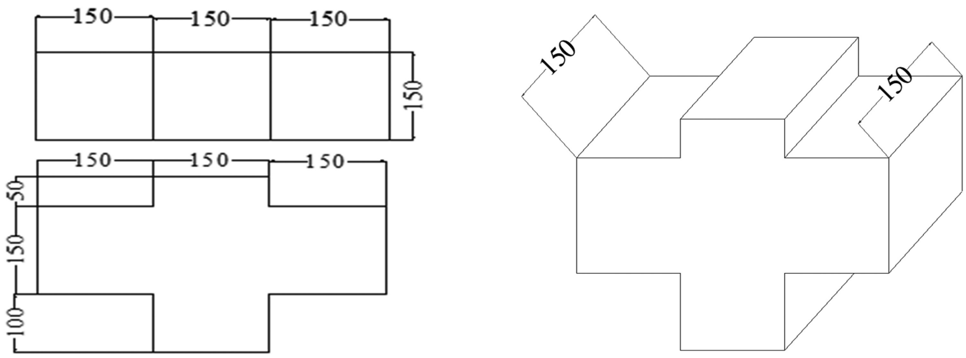
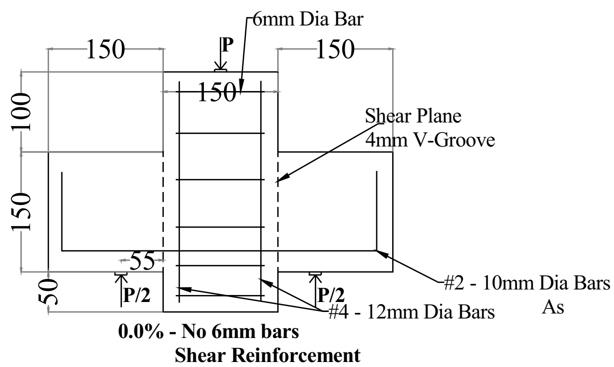


Figure 4. SDC corbel specimen geometry.

GPC strengths and different percentage of shear or horizontal stirrups i.e. shear reinforcement in the form of closed loop stirrups has been provided across the shear interface in GPC corbels. The longitudinal reinforcementof the column consisted of four 12 mm diameter bars with a yield strength of 500 MPa (Figure 5(e)). Horizontal lateral ties have been provided in the column of diameter 6 mm spaced at 75 mm c/c along the length of the column. In the corbels, the primary tension reinforcement consisted of 2–10 mm having a yield strength of 500 MPa conforming to IS: 1786–2008. Shear reinforcement (A_h) in the form of horizontal reinforcement consisted of 2-legged 6 mm dia. closed loop stirrups placed across the shear plane. The number of closed loop stirrups is varied to change the percent of shear reinforcement across the interface. The details of reinforcement of corbel specimen cast are shown in Figure 5(e). 4 mm deep V grooves were made along the vertical direction at the corbel – column junction for ensuring the location and the direction of shear crack.

A 100 kg capacity rotating drum type pan mixer was used for the proper mixing of materials. Initially, binder materials (fly ash and GGBS) along with fine and coarse aggregates are mixed followed by the addition of alkaline activator solution and super plasticiser. The mixing time was 5 to 7 min once all the ingredients have been added. After mixing, the fresh mix was poured into moulds and compacted. Specimens were demoulded after 24 h and air cured at ambient conditions (temperature – 35 ± 2°C and relative humidity – 75%) for 28 days.



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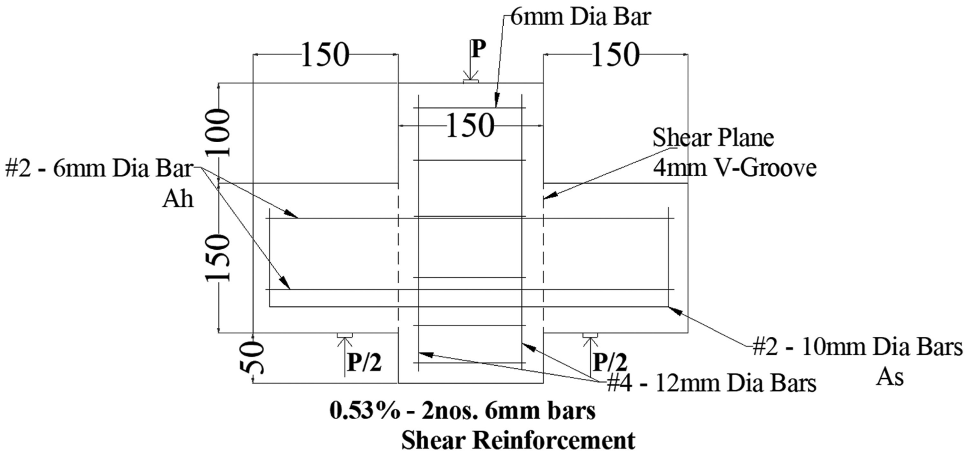

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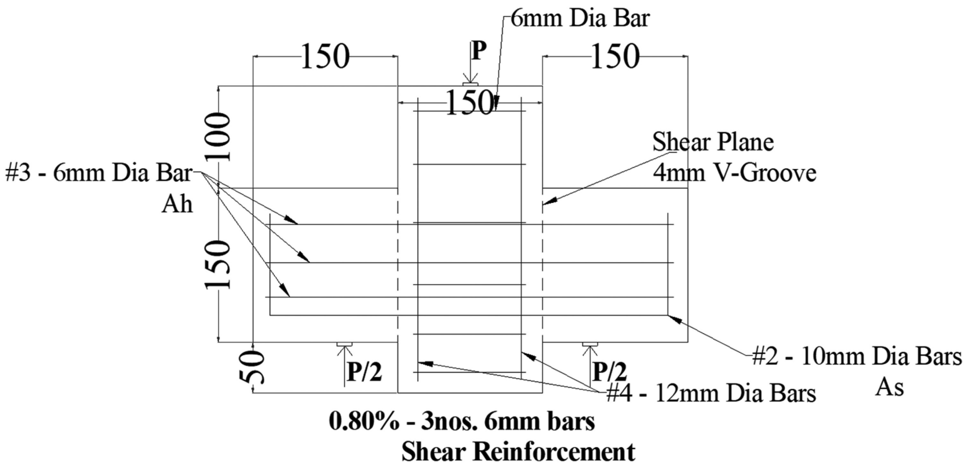


Figure 5. Continued.

3.4. Testing of GPC corbels

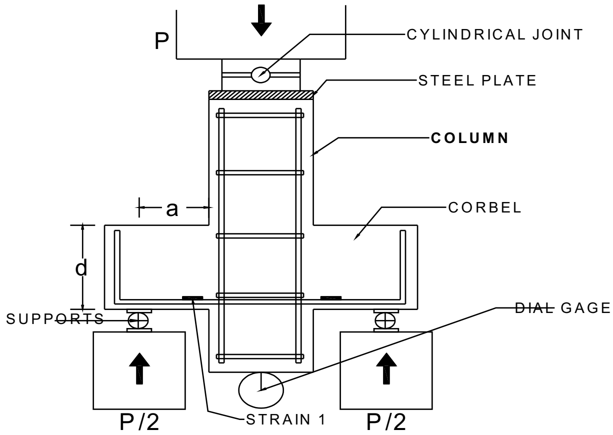
The test set up for the corbel is shown in Figure 5(d). For convenience, the corbel specimens have been tested in an inverted position as shown in Figure 6. The corbels have been supported on plain bearing free rollers resting on top of legs of the supporting wedge at a distance 'a = 55 mm' from the face of column. The vertical load on the column section have been applied by 2000kN capacity Tinius Olsen Testing (T.O.T) machine located concentrically on top of the column. This setup was assumed to impart only vertical load and no horizontal load is developed.

The corbel sample are subjected to incremental loads until the failure. The failure was characterised by the appearance of cracks near the interface. Since the tested corbel is a symmetric double cantilever type, the shear load at each interface is considered as 50% of the maximum load at failure and the same are shown in Table 4. Table 5 presents the maximum observed deflection. The failure patterns of the tested corbel specimens are shown in Figure 7. The failure in general was characterised by the appearance of cracks near the interface.

The deflection of the corbels is measured using the dial gauge placed under the column portion at the centre of SDC specimen. The load deflection plots are shown in Figure 8.

4. Results and discussions

All GPC Corbels were tested till the failure. The load-deflection curve obtained for the GPC corbels has three different phases such as uncracked, cracked and ultimate. The uncracked phase ends upon the appearance of fine and visible cracks. In the uncracked phase, the deflection increased linearly in all the corbels with load. After the first crack, a noticeable decrease in stiffness i.e. change in slope of load – deflection curve was observed. The uncracked phase ended approximately at about 30 to 35% of the observed ultimate load. In cracked phase, the load deflection curve varied non-linearly characterised by the decrease in the slope of the load – deflection curve with increasing load. On reaching the ultimate the load started decreasing



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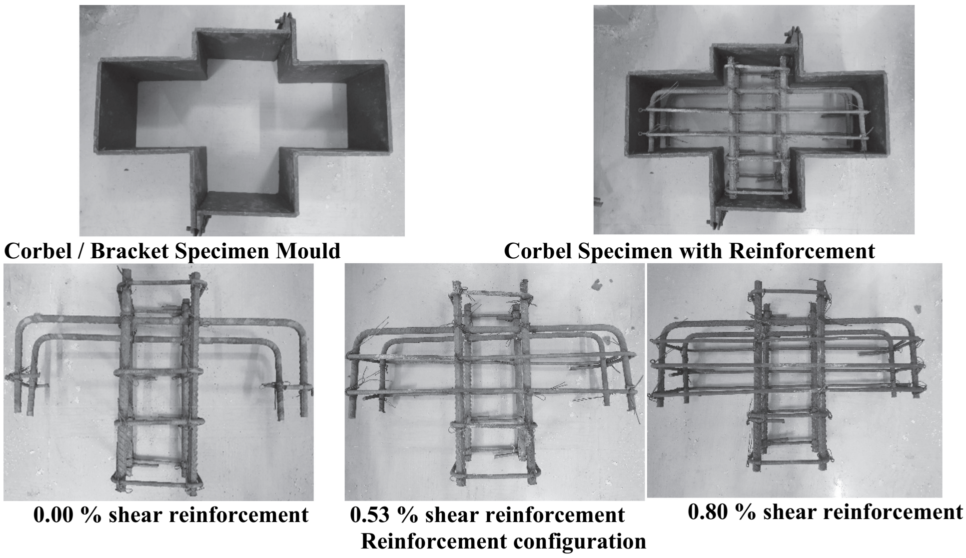


Figure 5. Continued.

Figure 6. Test setup.

characterised by the increase in deflection. Figure 8 shows the load-deflection curves of the tested GPC corbels

From Table 5, it can be observed that the minimum increase in the shear load observed is about 52% and 94% for the 0.53% and 0.80% of transverse reinforcement respectively at the interface of corbel. Similarly the minimum increase in the deflection at ultimate observed is about 32% and 51% for the 0.53% and 0.80% of transverse reinforcement respectively at the interface of corbel. Also, it can be observed from Table 5, that the average ultimate shear strength increased from 87.59 kN to 156.76 kNi.e. the increase is about 79% as the compressive strength of GPC

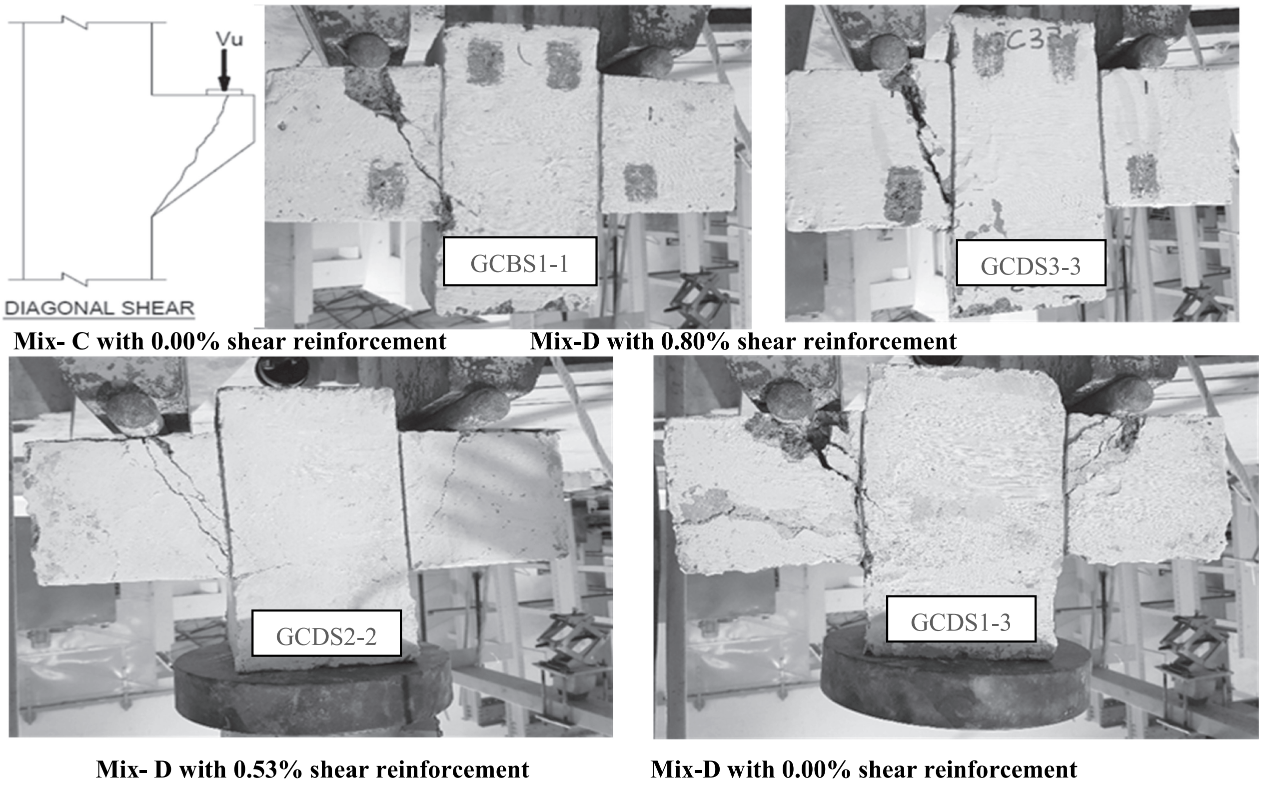


Figure 7. Failure pattern of corbel/bracket.

Table 5. Ultimate shear and corresponding deflection of corbel specimens.

Description	Ultimate Load kN	% of the increase in Load	Deflection mm	% of the increase in Deflection
GCBS1 – 1	87.11	-	4.10	-
GCBS1 – 3	87.28		4.36	
GCBS1 – 6	88.37		4.61	
Average	**87.59**		**4.36**	

(*Continued*)

Table 4. The Max. Shear force at the interface and corresponding Shear stress at the interface of GPC Corbels.

increased from 20MPa to 50MPa. The shape and area under the load deflection curves are often used as indicators of ductility and toughness, respectively. The load deflection diagrams (Figure 8) shows that additional secondary reinforcement resulted in an increase in load carrying capacity as well as in ductility of corbels.

During the testing, visible cracks were observed near the re-entrant corner of the column corbel interface. With an increase in the load,a few more inclined (shear) cracks were formed well within the shear span and slightly away from the interface. The failure was characterised by the widening of one or more shear cracks associated with concrete crushing 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 were 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.

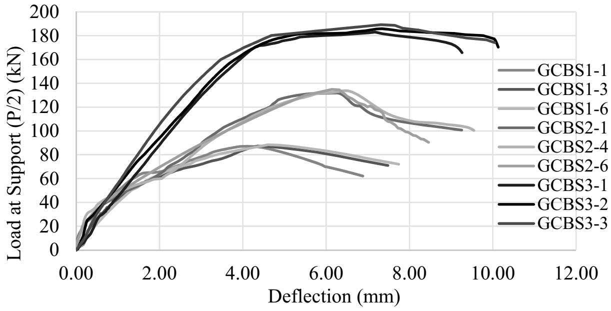
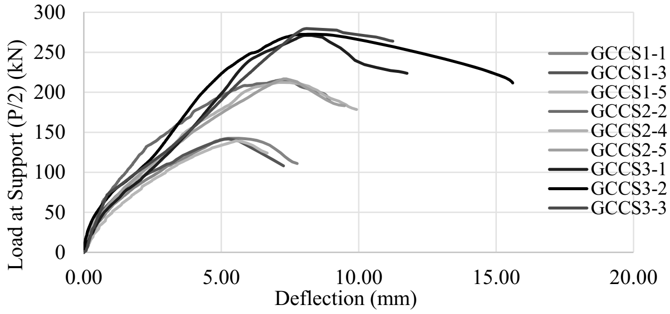


Figure 8. Load deflection curves for corbels.



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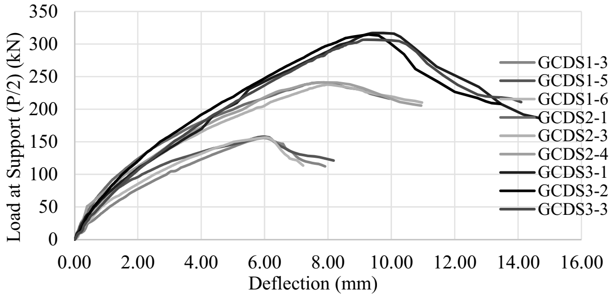

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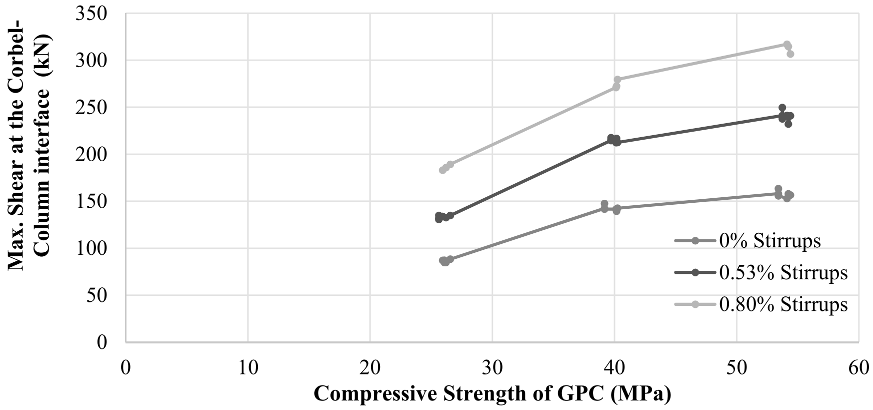

Figure 9. Shear Strength vs. Compressive Strength of GPC Corbels.

There were no signs of cracks/crushing in the column portion was observed. Similar failure pattern was observed by the previous investigators (Kriz and Raths (1965), Mattock (1976)) on RC corbels.

The variation of shear strength of GPC corbels with the corresponding GPC compressive strength is shown in Figure 9. From the variation it is observed that the shear strength has increased with an increase in compressive strength of GPC. Also, the rate of increase of shear strength has slightly decreased for a compressive strength of GPC more than 40MPa.

5. Validation of the proposed analytical expression for shear strength

In order to predict the shear strength of monolithic GPC interface, the equation proposed by the authors (Kumar 2021), based on experimental tests on push off specimens was used. Total percentage of the steel consist of both primary (main tension steel) and secondary (closed loop stirrups) reinforcement was used in the calculation of the shear strength contribution (V_f) of reinforced GPC due to friction. As the failure of GPC corbels was characterised by the diagonal shear cracks, for the calculation of shear strength contribution due to dowel action, only closed loop stirrups crossing the interface only was considered. The shear capacity calculated from the predicted equation has been compared with that of the experimental shear capacity obtained from the tests of corbel samples. The results of comparison are given in Table 6. From Table 6, it may be observed that there is about 9% variation in the predicted results compared to experimental shear strength results. Hence, it may be concluded that the results of the experimental shear capacity of corbel are well in agreement with the model proposed to predict the interface shear capacity of monolithic fly ash and GGBS-based geopolymer concrete. Figure 10 shows the correlation of experimental and predicted shear capacity of the tested corbels. The coefficient of correlation between the experimental and predicted shear capacity of the tested corbels is 0.99.

Table 7 presents the comparison of experimental shear capacity of GPC-reinforced corbels with the shear strength of corbels predicted using different Design Codes/equations applicable to the conventional concrete available in the literature. The comparative study shown in Figure 10, indicates that the available normal concrete shear capacity of corbels prediction models is highly conservative in estimating the shear capacity of GPC Corbels. The comparison shows that the shear capacity obtained from different theories and codes are varying from 44% to 87% less than the experimental shear strength of geopolymer-reinforced Corbels.

6. Conclusions

The following are the conclusions arrived at after the experimental and comparative study of shear capacity of GPC corbels

(1) The ultimate load capacity of corbels increased with increase in the compressive strength of GPC.
(2) The rate of increase of shear strength has slightly decreased for compressive strength of GPC approximately more than 40 MPa.
(3) The ultimate load of corbels was increased by an increase in the percentage of closed loop stirrups (secondary reinforcement).
(4) The experimental shear strengths of corbel are about 9% higher than the predicted interface shear strength of GPC corbels using the proposed analytical expression.
(5) The shear capacity as obtained from different codes and theories are underestimating the interface shear capacity of reinforced GPC corbels by about 44 to 87%.

Disclosure statement

On behalf of all authors, the corresponding author states that there is no conflict of interest.

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Table 7. Comparison of experimental shear capacity of GPC-reinforced corbels with the shear strength predicted by the design codes/equations.

			Ref [13]		Ref [12]		Ref [2]		Ref [20]		Ref [18]		Ref [8]		Ref [6]	
Spec. ID	f_{gpc}N/mm^2	V_{ue}kN	V_{u1}kN	V_{ue}/V_{u1}	V_{u2}kN	V_{ue}/V_{u2}	V_{u3}kN	V_{ue}/V_{u3}	V_{u4}kN	V_{ue}/V_{u4}	V_{u5}kN	V_{ue}/V_{u5}	V_{u6}kN	V_{ue}/V_{u6}	V_{u7}kN	V_{ue}/V_{u7}
Notation:			f_{gpc} = Ave Comp.St of GPC (MPa)Vue = Expt. Shear Strength of Corbel (kN)Vu1 = Predicted Shear Strength of Corbel (kN) using the proposed equation [Ref.13] Vu2 = Predicted Shear Strength of Corbel (kN) [Ref.12] Vu3 = Predicted Shear Strength of Corbel (kN) [Ref.2]								Vu4 = Predicted Shear Strength of Corbel (kN) [Ref.20] Vu5 = Predicted Shear Strength of Corbel (kN) [Ref.18] Vu6 = Predicted Shear Strength of Corbel (kN) [Ref.8] Vu7 = Predicted Shear Strength of Corbel (kN) [Ref.6]					
GCBS1-1	25.94	87.11	85.90	1.01	67.63	1.29	82.47	1.06	85.32	1.02	86.07	1.01	83.49	1.04	94.58	0.92
GCBS1-2	26.07	84.91	86.25	0.98	67.80	1.25	82.47	1.03	85.32	1.00	86.19	0.99	83.60	1.02	94.68	0.90
GCBS1-3	26.07	87.28	86.25	1.01	67.80	1.29	82.47	1.06	85.32	1.02	86.19	1.01	83.60	1.04	94.68	0.92
GCBS1-4	26.07	85.71	86.25	0.99	67.80	1.26	82.47	1.04	85.32	1.00	86.19	0.99	83.60	1.03	94.68	0.91
GCBS1-5	26.21	84.81	86.62	0.98	67.98	1.25	82.47	1.03	85.32	0.99	86.31	0.98	83.73	1.01	94.79	0.89

(*Continued*)

Table 6. Validation of the proposed analytical expression for shear strength at the monolithic interface of corbel.

Corbel Specimen	f_{gpc}	A_{Main}	$A_{Stirrups}$	Vue	Predicted Shear strength based on Monolithic Interface of GPC Vc	Vf	Vd	Vup	Vue/Vup
GCBS1-1	25.94	0.74	0.00	87.11	54.49	31.42	0.00	85.90	1.01
GCBS1-2	26.07	0.74	0.00	84.91	54.83	31.42	0.00	86.25	0.98
GCBS1-3	26.07	0.74	0.00	87.28	54.83	31.42	0.00	86.25	1.01
GCBS1-4	26.07	0.74	0.00	85.71	54.83	31.42	0.00	86.25	0.99
GCBS1-5	26.21	0.74	0.00	84.81	55.20	31.42	0.00	86.62	0.98
GCBS1-6	26.56	0.74	0.00	88.37	56.14	31.42	0.00	87.55	1.01
GCBS2-1	25.62	0.74	0.53	131.89	53.64	42.72	24.37	120.73	1.09
GCBS2-2	25.62	0.74	0.53	134.88	53.64	42.72	24.37	120.73	1.12
GCBS2-3	25.62	0.74	0.53	130.70	53.64	42.72	24.37	120.73	1.08
GCBS2-4	25.94	0.74	0.53	133.83	54.49	42.72	24.67	121.89	1.10
GCBS2-5	26.21	0.74	0.53	132.59	55.20	42.72	24.93	122.86	1.08
GCBS2-6	26.56	0.74	0.53	134.88	56.14	42.72	25.26	124.13	1.09
GCBS3-1	25.94	0.74	0.80	183.07	54.49	48.38	55.52	158.38	1.16
GCBS3-2	26.21	0.74	0.80	185.81	55.20	48.38	56.10	159.68	1.16
GCBS3-3	26.56	0.74	0.80	189.26	56.14	48.38	56.85	161.36	1.17
GCCS1-1	39.18	0.74	0.00	142.46	92.21	39.27	0.00	131.48	1.08
GCCS1-2	39.18	0.74	0.00	147.54	92.21	39.27	0.00	131.48	1.12
GCCS1-3	39.18	0.74	0.00	141.65	92.21	39.27	0.00	131.48	1.08
GCCS1-4	40.12	0.74	0.00	141.62	94.61	39.27	0.00	133.88	1.06
GCCS1-5	40.16	0.74	0.00	139.43	94.66	39.27	0.00	133.93	1.04
GCCS1-6	40.24	0.74	0.00	142.60	94.76	39.27	0.00	134.03	1.06
GCCS2-1	39.71	0.74	0.53	214.62	93.82	53.41	37.77	184.99	1.16
GCCS2-2	39.71	0.74	0.53	215.46	93.82	53.41	37.77	184.99	1.16
GCCS2-3	39.71	0.74	0.53	217.66	93.82	53.41	37.77	184.99	1.18
GCCS2-4	40.12	0.74	0.53	212.33	94.61	53.41	38.16	186.18	1.14
GCCS2-5	40.16	0.74	0.53	216.78	94.66	53.41	38.20	186.27	1.16
GCCS2-6	40.24	0.74	0.53	212.58	94.76	53.41	38.28	186.44	1.14
GCCS3-1	40.12	0.74	0.80	271.10	94.61	60.47	85.87	240.95	1.13
GCCS3-2	40.16	0.74	0.80	272.46	94.66	60.47	85.95	241.09	1.13
GCCS3-3	40.24	0.74	0.80	279.60	94.76	60.47	86.12	241.35	1.16
GCDS1-1	53.41	0.74	0.00	158.07	109.84	39.27	0.00	149.11	1.06
GCDS1-2	53.41	0.74	0.00	163.43	109.84	39.27	0.00	149.11	1.10
GCDS1-3	53.41	0.74	0.00	155.84	109.84	39.27	0.00	149.11	1.05
GCDS1-4	54.11	0.74	0.00	152.89	110.62	39.27	0.00	149.89	1.02
GCDS1-5	54.22	0.74	0.00	157.89	110.74	39.27	0.00	150.01	1.05
GCDS1-6	54.39	0.74	0.00	156.56	110.93	39.27	0.00	150.20	1.04
GCDS2-1	53.73	0.74	0.53	241.13	110.20	53.41	51.11	214.71	1.12
GCDS2-2	53.73	0.74	0.53	249.69	110.20	53.41	51.11	214.71	1.16
GCDS2-3	53.73	0.74	0.53	237.72	110.20	53.41	51.11	214.71	1.11
GCDS2-4	54.11	0.74	0.53	241.23	110.62	53.41	51.47	215.50	1.12

(*Continued*)

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