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*Full Length Research Paper*

# Utilization of natural and industrial mineral admixtures as cement substitutes for concrete production in Jordan

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**Several materials such as tuff and Tripoli which is naturally occurring and industrial by-products wastes as high calcium ash and slag were investigated as cement substitutes in this paper. Compressive strength of various standard mortar samples have been tested at 7 and 28 days. The obtained results show that these materials have improved the properties Ordinary Portland Cement (OPC) concrete. It was found that an increased compressive strength of 22% were attained at an optimum of 10% addition of ash. Meanwhile, the compressive strength increases by 30% when the content of Tripoli is of 10%. The slag and Tuff have adverse effect on the strength. The reason may be attributed to the mode and original of crystals. The finding of this research work also show that using these materials will reduce the use of OPC. This reduction will reduce some of the adverse environmental effect due to OPC production and consequently, reduces the consumption of energy in Jordan.**

**Key words:** Ordinary Portland Cement (OPC), cement substitutes, tuff, Tripoli, ash, environmental.

## INTRODUCTION

Mineral admixtures are materials used as an ingredient of concrete or mortar and added to the batch immediately before or during mixing mainly to improve or modify several properties of Portland cement concrete PCC. Mineral admixtures are used in conjunction with Portland or blended cements as a supplementary cementing material (SCM) through hydraulic or pozzolanic activity or both. PCC mixtures are multiphased, particle-reinforced composites that consist of irregularly shaped and randomly oriented aggregate particles embedded in an inelastic matrix. PCC mixtures generally exhibit complicated mechanical behavior and multiple modes of damage. Although precise identification and prediction of the inelastic damage modes of the PCC is extremely difficult, it is important to seek out simpler approaches of predicting mechanical behavior including damage characteristics of the mixture in place of expensive and time-consuming laboratory experiments where possible.

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**Table 1.** Classification of class C and F compared with Karak ash (Analysis was done using XRF technique).

| Oxide (%) | Ferguson et al. (1999) | | Ordinary Portland Cement* | Karak ash |
|---|---|---|---|---|
| | Class C ash | Class F ash | | |
| $SiO_2$ | 54.9 | 39.9 | 19.94 | 23.82 |
| $Al_2O_3$ | 25.8 | 16.7 | 5.37 | 5.34 |
| $Fe_2O_3$ | 6.9 | 5.8 | 3.18 | 1.94 |
| CaO | 8.7 | 24.3 | 63.65 | 52.84 |
| MgO | 1.8 | 4.6 | 2.59 | 0.9 |
| $SO_3$ | 0.6 | 3.3 | 2.88 | 8.71 |

*Neville (1995).

Recently, the use of reinforced concrete in multi store structure and industrial plants in the vicinity of the capital Amman and surrounding towns is continuously increasing in the last decade. There is shortage in production and huge demand on OPC which the local cement factories cannot satisfy the local market needs. This has led to an increase in the demand energy. On the other hand, there is a negative environmental impact of cement production. For instance the production of one ton of OPC releases one ton of $CO_2$ and these issues are considered the most important challenges facing construction sector.

Some of the local cement factories in Jordan have investigated the use of bituminous limestone ash as a source of raw material after direct combustion and reducing its energy in the kiln system. This may be considered as a good approach for energy saving, but it will lead to complicated environmental problems due to the evolved $SO_x$ and $CO_x$ gases.

To achieve sustainability of the OPC, changes in working practices are required. This means that low energy, low carbon, and low waste techniques have to be developed to replace more intensive techniques. For example, *in situ* remediation of contaminated land may become a more attractive than a ‘dig-and-dump’ approach, where the material is removed and disposed of to landfill and then replaced with new material. These techniques are not only high in energy consumption and produce large amounts of contaminated waste, but also highly expensive. Alternative methods however, have been recently developed to reuse several types of industrial waste materials in civil engineering construction by many researchers such as Tay (1987), Churchill (1994), Perez et al. (1996), Stefanov (1986), Pereira et al. (2000), Pavlova (1996) and Maaitah (2012).

In this paper, some materials such as Tripoli, ground tuff, high calcium ash and air cooled slag will be added to the mortar. These materials have been investigated as cement substitutes. The aim of this work is to investigate the enhancement of mechanical properties of the PCC by the addition of mineral admixtures to the concrete mixtures as cement substitutes. The optimum percent or fixation value for each additive will be investigated. The weight of cement content decreases but the strength is increased that is required to produce a certain class of reinforced concrete with improved quality and durability through adding self cementitious materials as mineral admixtures. This may reduce the consumption of OPC which in turn may help reduce all the adverse effect to the cement production.

## EXPERIMENTAL PROCEDURE

### Materials

The naturally occurring material such as ash and Tripoli are abundant in Jordan. Bituminous limestone as a source of ash and Tripoli are available as millions of tones in the vicinity of Al-Karak city about 120 km to the south of the capital Amman. Slag is available at many steel factories located at and around Amman. The material that produces waste as high calcium ash and slag were used as cement substitutes.

In the present work the Karak ash (from various outcrops from El-Lajjun) was obtained by direct combustion of Karak bituminous limestone at a temperature of (900 to 1000)°C. The sample was allowed to cool down to the ambient temperature which can be considered as fast cooling. This means that the crystal is micro crystalline. Then, the sample was ground under dry conditions to obtain the possible minimum grain size. Small ball mills and Los Angles machine were used. The sample was crushed using a jaw crusher to obtain bituminous limestone aggregates of 9 mm nominal size particles.

Ash could be the solid waste product of possible utilization of the Karak bituminous limestone. The ash used in this study is a high calcium ash that has been produced by direct combustion of the bituminous limestone at 950°C (Maaitah, 2012). The chemical properties of Karak ash are summarized in Table 1. The combusted bituminous rocks in Karak/Jordan have indicated the presence of two groups of minerals; high temperature which is equivalent to clinker cement (Khoury, 1993; Al-Hamaiedh, 2010) and low temperature which is similar to the hydrated cement products (Khoury and Nasser, 1982). The low temperature mineral group has a similar composition to the hydrated cement products and has been precipitated from high alkaline circulating water (pH > 12.5). This naturally occurring alkaline water is analogous to the cement percolating water (Khoury, 1993).

Tuff can be obtained from the northern province of Jordan. Tuff is a natural volcanic material that is characterized by very high porosity, low density, rich in $SiO_2$, and has a considerable content of $Al_2O_3$ in addition to $Fe_2O_3$. Huge quantities of this material are available in the eastern and north eastern provinces of the Kingdom; these natural recourses are utilized in various

**Table 2.** Chemical composition of slag, tuff, and Tripoli by using XRF technique. (Tests were carried out at the Natural resources authority (NRA) in Amman).

| Oxide Wt. % | Tuff | Ash | Slag |
|---|---|---|---|
| $SiO_2$ | 50.6 | 25.30 | 16.55 |
| $Al_2O_3$ | 15.2 | 2.35 | 7.8 |
| $Fe_2O_3$ | 11.2 | 1.37 | 18,33 |
| CaO | 9.0 | 45.21 | 7.12 |
| MgO | 5.8 | 1.63 | 7.83 |
| $P_2O_5$ | 4.6 | 5.47 | ------ |
| $Na_2O$ | ---- | 0.85 | 2.83 |
| $TiO_2$ | --- | 0.14 | ------ |
| MnO | 2.5 | 0.02 | 4.52 |

**Table 3.** Chemical composition and physical properties of Tripoli.

| Area in Karak | $SiO_2$ | CaO | $Al_2O_3$ | $Na_2O$ | $Fe_2O_3$ | MgO | L.O.I | Specific gravity |
|---|---|---|---|---|---|---|---|---|
| Shahabiyeh* | 94.50 | 0.44 | 0.33 | 0.45 | 0.16 | 0.23 | 3.10 | 2.43 |
| Adnanieh* | 90.64 | 2.16 | 0.11 | 0.94 | 0.13 | 0.37 | 4.77 | 2.6 |

* both village in west of Karak.

engineering and agricultural aspects.

About 20 kg of reddish crushed sand size was sampled from Amani quarry at Tall Hassan in the vicinity of Al-Azraq area. The sample was ground to get the maximum possible passing #200 sieve fraction, the Loss Angles abrasion machine was used for this purpose to produce a bulk fine tuff sample that will be used later to investigate the cement-tuff mortars. The chemical composition of each material was determined using the XRF technique as shown in Table 2.

Slag can be obtained from the United Iron and Steel MFG. Co., 30 km to the south of Amman. Huge quantities of blackish, tough aggregation stockpiles of iron slag wastes are available as by-product of iron production at different steel factories in the vicinity of Amman and surrounding areas. The sample was obtained from Al Manaseer Steel Factory about 30 km south of Amman. The sample was ground to fine powder and passing sieve No. 200 was used.

The chemical composition of each material was determined using the XRF technique as shown in Table 2. Conplast $C_3O$ as a super plasticizer was used in constant dosage in all trials and with Specific Gravity of 3.3. The unit weight is 1750 kg/m$^3$ and the Absorption is up to 2.32. The slag crystal is not compatible, some crystal is glassy because of the cooling is very fast in the surface and some fine. The fine crystal is constituted because it is cooled slowly somehow at the down of the stack. Therefore, the production is out of control and the composition varies from time to time. It is difficult to find similar sample among the slag stack after manufacturing.

Tripoli is microcrystalline silica. It is a form of silica, earthy, light colored, light weight, friable, very fine grained chalcedonic and opaline silica of cryptocrystalline, and it is commercially classified as "Soft Silica" or "Amorphous Silica", having a 90 to 94% Silica content with high purity and high melting point. The Tripoli can be used as filler material in paints and rubber, plastic industries, mild abrasive, insecticides. The occurrences are found in Karak District, 168 km Southwest of Amman, in the following areas:

i) Around thirteen million metric ton in El-Adnanieh (8 km South of Karak);
ii) Around seven million metric ton in El-Shahabiyeh, 4 km Southwest of Karak;
iii) Undetermined huge millions metic tonnage in Wadi Rakin and Wadi Ben Hamad 4.5 km Northwest of Karak;
iv) Undetermined millions of metic tonnage in Wadi Falqa, 5.5 km Southwest of Karak;
v) Undetermined huge millions metic tonnage in Ainun, 5.5 km South-Southwest of Karak along the west bank of Wadi Daba.

The chemical composition of Tripoli was determined using the XRF technique as shown in Table 3. The Adnanieh Tripoli were used in this work.

## Approach

The combusted limestone ash, slag, tuff and Tripoli were ground using Loss Angles machine for one and half an hour each, followed by sieving the ground material on No. 200 sieve. The fraction passing #200 sieve was collected in tight plastic bags. The silica sand was sieved to prepare standard sand for mortar preparation. A reference mortar mix composed of 1500 gr of the standard silica sand was mixed with 500 gr of OPC at a w/c ratio of 0.485. The mortar was mixed well and cast into 5x5x5 cm cubes, two layers into each cube and 10 blows on each layer using a standard rod with 2x5 cm cross sectional area. Six cubes were prepared and de-molded after 24 h. The samples were cured in water until testing at 7 and 28 days. Compressive strengths presented are the averages for 3 cubes at each of the (7 and 28 days). The same procedures were repeated for the other mortars- admixtures with substituting cement by various percents of that admixture. ASTM C109 was followed strictly for the whole procedure. The compressive strength of various standard mortar samples have been tested at 7 and 28 days.

## Mortar composition

The proportions of materials for the standard mortar shall be one part of cement to 2.75 parts of graded standard sand by weight.

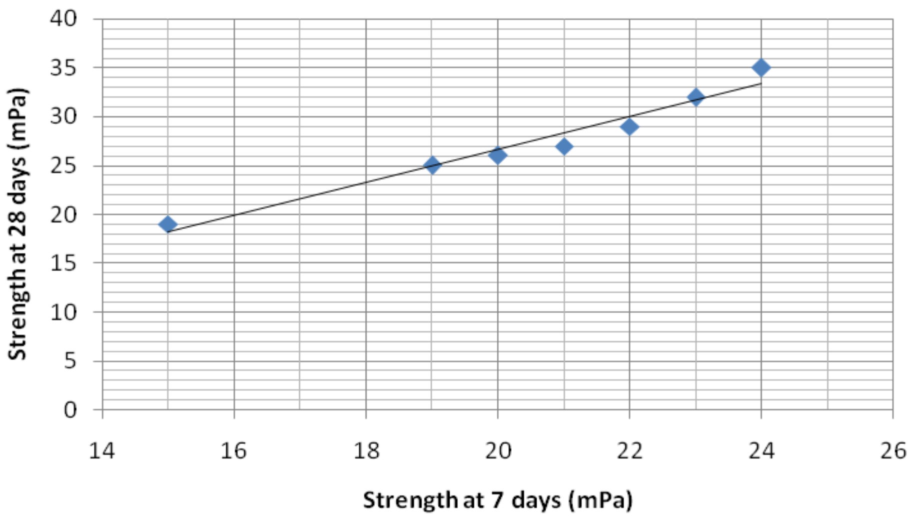


**Figure 1.** Comparison between strength at 7 and 28 days for Tripoli.

The water/cement ratio of 0.485 was used for all Portland cements. The water/cement ratio for other than Portland and air entraining Portland cements shall be such as to produce a flow of 110 +/- 5.

The specimen mold preparation was conducted using Mortar Mixing Procedure in accordance with (ASTM C305). This was performed by applying a thin coating of mold release to the interior surfaces of the molds and base plates. Surfaces were wiped with a cloth to remove any excess, with dry paddle and bowl placed in the mixing position of the mixer. The strength for native sample (without any additives) after 7 days is 21 kg/cm$^2$ and after 28 days is 27 kg/cm$^2$. An increase in strength of 28.5%, possibly due to curing time.

## RESULTS AND DISCUSSION

The bond strength between cement substitutes and concrete materials is related to the interface properties, interface fracture mechanics and most likely to the crystal. The mode of crystal (that is, phanertic, aphanertic and glassy) play a significant role in determining the success of the substitutes. The loadings will impart both tensile and compressive normal and shear stresses at the interface, and thus failure will be under multi-component stresses.

The development of the self cementaceous properties and strength are controlled by the additive content and the curing period. The additive is similar to a great extent to the Portland cement. The results are obtained through the standard mixtures of each sample (Table 4). The mixtures are prepared and cured under the same standard procedures and conditions. Figure 1 show the effect of curing time on Tripoli which improve that the Tripoli reacts with OPC. This is because it has micro crystalline original and a high content of $SiO_2$ and CaO reacts with cement (Table 5).

The strength of mortar increases as the Tripoli content increases up to specific value (that is, 20%) and then it has no effect on strength up to 40% content. The content of Tripoli more than 40% has adverse effect because the strength decreases as shown in Figure 2. At fixation point (10% of Tripoli content) the strength improved 30% from native sample after 28 days. On the other hand, the strength increases by 14% after 7 days for the same Tripoli content. This is an improvement on the fact that the Tripoli reacts with time. The well known reaction between the elite ($C_3S$) and water is

$$3CaO.SiO_2 + H_2O \rightarrow 2CaO.H2O + Ca(OH)_2 + 120\ cal/gr$$

The extra Portlandite $Ca(OH)_2$ will react with $SiO_2$ from Tripoli to produce Belit which will cause an increase in the strength by 30% as shown in Figure 2.

$$Ca(OH)_2 + ((SiO_2 + CaO)\ (from\ Tripoli)) \rightarrow 2CaOSiO_2\ (Belit)$$

The extra Belit (*$2CaOSiO_2$*) which is due to the addtion of Tripoli is appearantly resposible for the increase in compressive strength.

From the obtained results in Figure 3, it is clear that the mortar strength has increased by 22% of ash fixation point after 28 days, whilst, at 7 days an increase of 14% were observed. The fixation point is at 10% ash content.

The ash and Portland cement are essentially composed of lime (CaO). Silica ($SiO_2$) and alumina ($Al_2O_3$) are present at higher concentrations in the Portland cement and react with CaO at about 1425°C to form alite. Heat treatment of the bituminous rocks and Portland cement raw material involves dehydration, thermal decomposition of clay minerals (300 to 650°C), decomposition of calcite (greater than 800°C), the formation of belite ($C_2S$), tricalcium aluminate ($C_3A$), and tetracalcium alumina ferrite ($C_4AF$). The liquid phase and

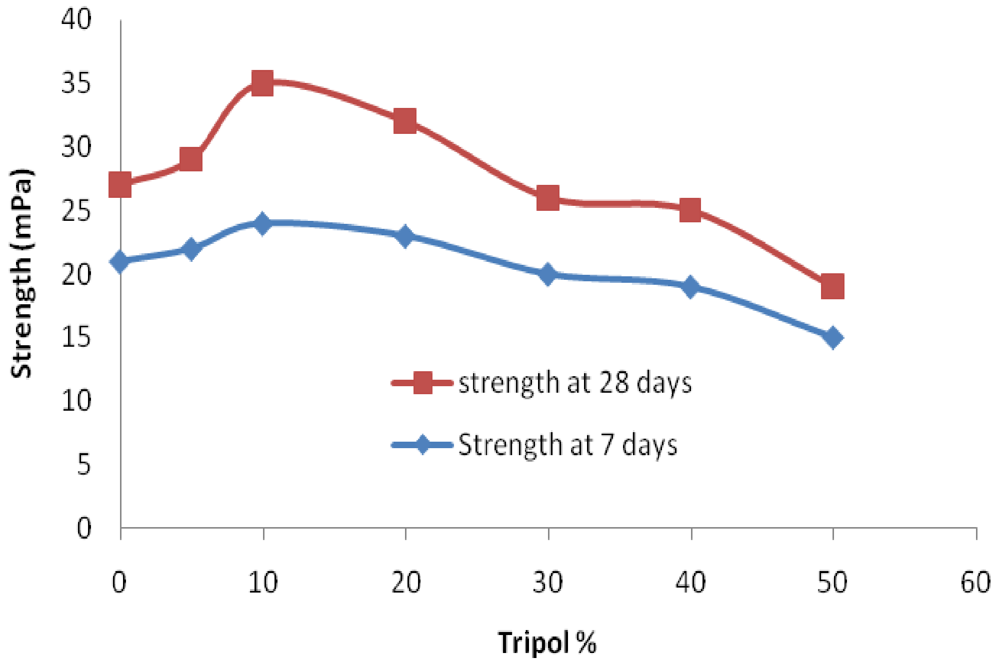


**Figure 2.** Mortar strength versus Tripoli content.

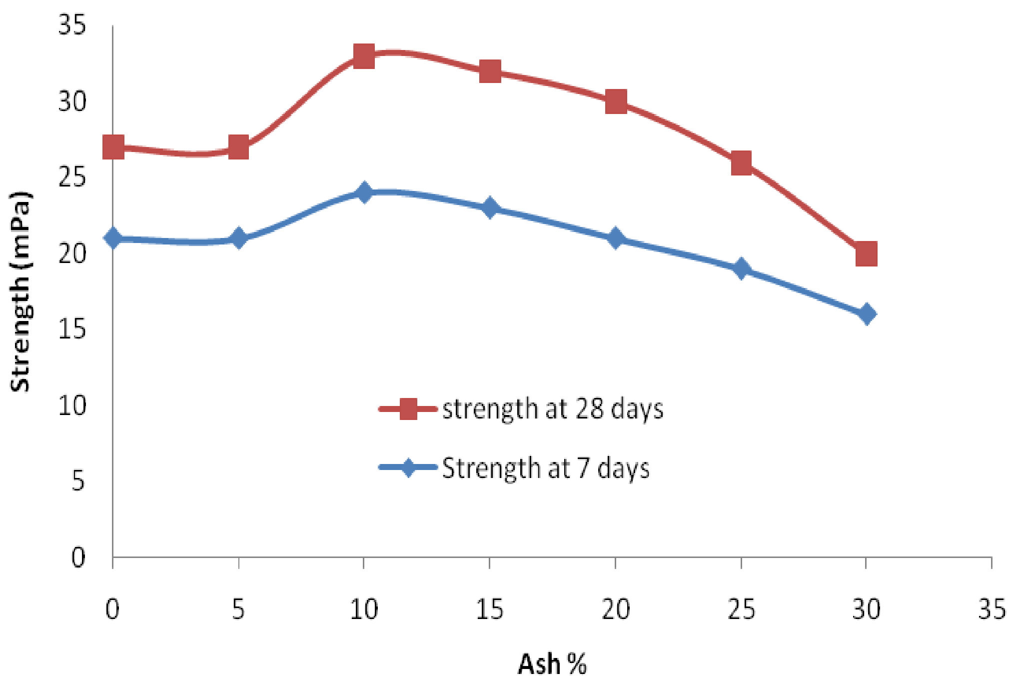


**Figure 3.** Mortar strength versus Ash content.

sintering at about 1425°C form alite ($C_3S$) which is responsible for the strength of concrete (Khoury, 1993).

The variable content of $SO_3$, the alkali CaO, and pozzolanic content ($SiO_2$ +$Al_2O_3$+ $Fe_2O_3$), are found in both the ash samples and OPC raw material. The strength build up in all the samples is related to the setting reactions of lime (CaO) with the pozzolanic constituents to produce calcium silicate hydrate (CSH) and calcium aluminate hydrate (CAH). High pH solution is highly reactive with amorphous Al-Si rich phases at

**Table 4.** The following batch component, which is sufficient for 6 samples.

| Material | Amount (g) | w/c |
|---|---|---|
| Portland Cement | 500 | |
| Silica Sand | 1375 | |
| Distilled Water | 242 | Portland (w/c=0.485) |

**Table 5.** Summary of the results.

| Additives | % of additives | %Strength at 7 days/strength for native sample | %Strength at 28 days/strength for native sample |
|---|---|---|---|
| Tripoli | 20 | 14 | 30 |
| | 50 | -28.5 | -30 |
| Ash | 10 | 14 | 22 |
| | 20 | 1 | 11 |
| Slag | 5 | -10 | -4 |
| | 50 | -76 | -70 |
| Tuff | 10 | -5 | -8 |
| | 30 | -23 | -24 |

normal room temperature. Sulfate minerals as ettringite are expected to form because of the availability content of $SO_3$. The CaO content plays an important role in the alkali–pozzolanic reaction.

The reaction products leave the kiln as a clinker. The clinker leaves the kiln here to be cooled, mixed with gypsum, and then ground into a fine powder (cement). The setting of cement involves a number of stages at different rates. It is generally known that a complex series of reactions do take place as the cement reacts with water. Setting of $C_2S$ involves slow hydration reactions and the formation of Portlandite Ca $(OH_2)$ and calcium silicate hydrate.

Portlandite Ca $(OH)_2$ plays an important role in the setting reaction. Portlandite reacts with silicates and aluminum rich phases to form insoluble compounds which contribute to the strength formation (pozzolanic reactions). Excess portlandite reacts with atmospheric $CO_2$ to precipitate calcium carbonate that helps in strengthening the product after aging.

The combusted bituminous rocks in central Jordan have indicated the presence of two groups of minerals; high temperature which is equivalent to clinker cement (Khoury, 1993) and low temperature that is similar to the hydrated cement products (Khoury and Nasser, 1982). The low temperature mineral group has a similar composition to the hydrated cement products and has been precipitated from high alkaline circulating water (pH > 12.5). This naturally occurring alkaline water is analogous to the cement percolating water (Khoury, 1993).

Figure 4 and Table 5 show that the tuff has an adverse effect on strength. This is possibly, because the Tuff is poured rock and the powder has high surface area. The Tuff powder absorbs extra water, in turn affecting the water/cement ratio for reaction. This could be related to the kind of crystal of tuff that shows no reaction with OPC. Figure 4 and Table 5, also, show that the slag has an adverse effect on strength. The slag production is out of control. The slag contains strange materials and impurities. The properties of slag vary from sample to sample. The slag durability may be low due to the rust.

The effect of curing time can be seen Figure 5 and Table 5. Curing period of 28 days and more has influenced the compressive strength results. High compressive strength values obtained for intact samples indicate no disintegration features under fully saturated conditions. All hydrated samples have shown a similar behavior to the hydrated OPC products but with lower compressive strength.

## Conclusion

Table 5 summaries the results of this research and illustrates that an addition of 20 and 10% Tripoli and ash, respectively, lead to an improvement of the strength with respect to native sample by 19 and 22%. It is also, apparent that any increases in the additive content more than the fixation point will cause a reduction in the compressive strength. The obtained results suggest that the presence of Tuff and slag additives have determined

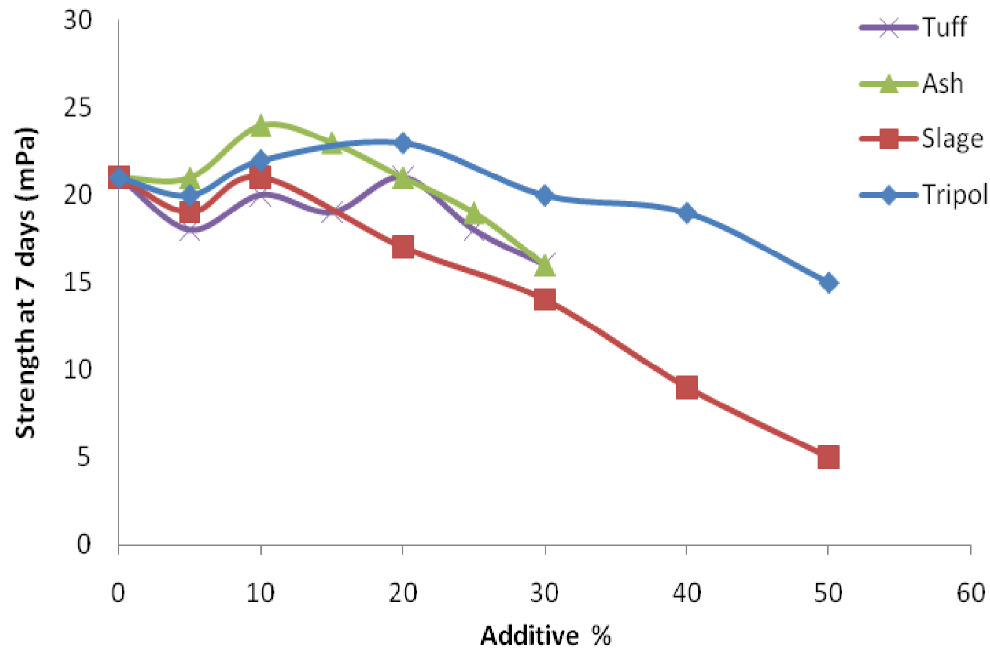


**Figure 4.** Strength versus percent of additives after 7 days.

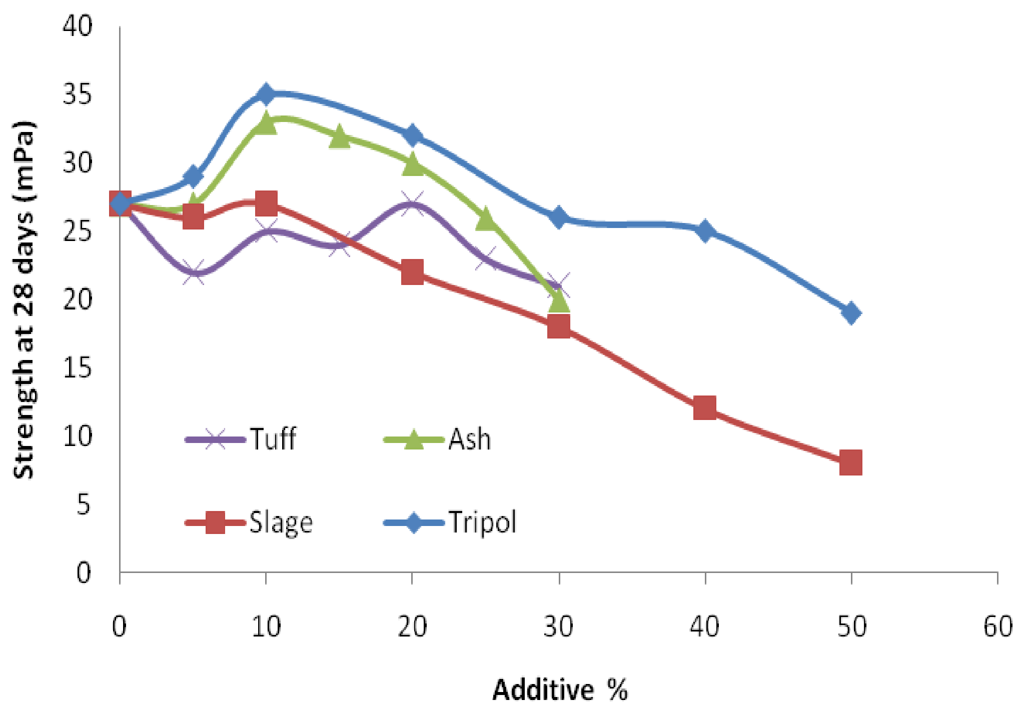


**Figure 5.** Strength versus percent of additives after 28 days.

effect on the compressive strength, which can be explained as a result of the mode and original of crystal (that is, phanertic, aphanertic and glassy). The crystalline plays a significant role in determining the success of the substitutes. Curing period of 28 days and more has a good influence on the compressive strength results.

## Conflict of Interest

The authors have not declared any conflict of interests.

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*Full Length Research Paper*

# Code compliant behaviour of palm kernel shell reinforced concrete (RC) beams in shear

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**This paper evaluates the shear strength of simply supported palm kernel shell (PKS) concrete and normal weight concrete (NWC) beams subjected to four-point loading. The primary variable of the investigation was evaluation of shear capacity of reinforced PKS concrete and NWC beams with and without shear reinforcement. Four pairs of reinforced concrete (RC) beams with similar geometrical properties were designed and tested with two replicate beams per design. Measured shear strengths at failure were compared with theoretical predictions calculated using BS 8110, ACI 318-05 and EC2. All but one specimen failed as a result of diagonal-tension. PKS beams exhibited lower ultimate shear capacities, smaller crack widths but higher displacements at failure. The performance of the EC 2, BS 8110 and ACI 318-08 code equations in predicting the shear resistance of PKSC and NWC beams is also presented. Based on the test results, it is concluded that shear capacity provisions of ACI give a conservative estimate of shear capacity of PKS lightweight concrete beams while the EC 2 gives a very close prediction of the shear capacity of PKS RC beams. The results of this study increases knowledge on the structural properties of PKS concrete, the efficient use of PKS and reduce the indiscriminate disposal of the PKS as a waste material.**

**Key words:** Palm kernel shells, normal weight concrete (NWC), Shear strength, BS 8110, ACI 318, EC 2.

## INTRODUCTION

The construction industry depends largely on conventional materials which are cement, granite and sand for concrete production. The increasing demand for concrete in the construction industry using normal weight aggregates such as gravel and granite has led to the depletion of the natural granite deposits at an alarming rate (Emiero and Oyedepo, 2012; Alengaram et al., 2008), while efforts to maintain ecological balance has also been a major challenge. However, efforts to substitute granite as coarse aggregate has been a major challenge (Adom-Asamoah, and Afrifa, 2010).

Lightweight aggregate concrete (LWAC) has successfully been investigated and used for structural purposes for many years. This is due to the added benefits to the construction industry in its application. Its use results in higher strength to weight ratio, higher tensile strain capacity, superior heat and sound insulation characteristic, and lower dead loads with smaller sections for structural members (Nahhas, 2013; Yasar, 2003). This and other associated benefits make the use of lightweight concrete (LWC) one of the important strategies in reducing the cost of construction in

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developing and underdeveloped countries (Shafigh et al., 2010; Ndoke, 2006). Lightweight aggregates may be either artificial or natural aggregates that are produced from a wide variety of raw materials and production procedures. Depending on the production techniques and the source of origin, lightweight aggregates vary within broad limits (Chi et al., 2003). The use of different materials, ranging from industrial wastes to agricultural waste, and other naturally occurring lightweight aggregates for concrete production have been reported in literature. Palm kernel shells (PKS) are agricultural solid waste obtained from cracking the fruits of the palm tree. PKS are hard, stony, light and naturally sized, flaky and of irregular shapes depending on the cracking pattern of the nut. Large quantities of PKS are produced by the palm oil mill but only a fraction is used as fuel for steam boilers at palm oil mills. The greater proportion of this by-product is therefore disposed off in stockpiles in open fields which results in negative impact on the environment. Exploiting the use of PKS in construction not only maximise the use of oil palm, but also helps to preserve natural resources and maintain ecological balance. Studies (Okpala, 1990; Mannan and Ganapathy, 2002) have shown that the use of PKS as aggregates can produce structural concrete of compressive strength of about 20 $N/mm^2$ at 28 days with a density in the range of 1800-1900 $kg/m^3$. The structural behaviour in relation to flexure and bonding has been reported in addition to the mechanical properties of PKS concrete (Teo et al., 2006; Alengaram et al., 2010). Teo et al. (2006) reported that ultimate moments predicted using BS 8110 provides a conservative estimate for PKS concrete beams up to a reinforcement ratio of 3.14%. Deflections and crack widths at service loads were all reported to be within maximum allowable values stipulated by BS110. Teo et al. (2007) investigated the structural bond and durability properties of PKS aggregate concrete. The authors concluded that the experimental bond strength of PKS concrete was much higher than theoretical bond strength as stipulated in BS 8110. Alengaram et al. (2010) concluded that the bond between PKS and cement matrix in tension was weaker compared to the bond between crushed granite aggregates and cement matrix.

Shear behavior and design of reinforced lightweight aggregate concrete (LWAC) beams in shear is one of the most topical issues in the structural behavior of reinforced concrete due to its brittle nature (Russo et al., 2004). It is generally assumed that lightweight concrete members are weak in shear due to their lower tensile strength. The shear failure of reinforced concrete beams without web reinforcement is a distinctive case of failure which depends on various parameters such as shear span to effective depth ratio (*a*/*d*), tension steel ratio (ρ), type of aggregate, strength of concrete, type of loading, and support conditions (Sudheer et al., 2011). The amount of shear reinforcement has a direct relation on the behavior of structural RC members. This is because it is possible for the structure to fail in a brittle manner without any warning if the shear stress exceeds the shear carrying capacity (Jasim, 2009). Structural design codes have therefore made provisions for the determination of safe shear capacities of concrete and have become more stringent on prevention of sudden failure (Mansour et al., 2004; Yang et al., 2003). Jumaat et al. (2009) investigated the shear strength of four reinforced oil palm shell foamed concrete (OPSFC) beams with a target density of 1600 $kg/m^3$ and a 28-day compressive strength of about 20$N/mm^2$. It was concluded that OPSFC with shear reinforcement failed in flexure mode while beams without shear reinforcement failed in shear modes. Additionally, it was reported that OPSFC beams had 50% higher deflections compared to corresponding NWC beams at ultimate stage. The spacing of cracks in OPSFC concrete beams were all reported to be closer than those found in NWC beams. Setiawan and Saptono (2012) reported on the shear capacity of reinforced concrete beam with different cross section types of lateral reinforcement on minimum ratio. It was concluded that the various types of shear reinforcement cross sections had no significant differences for shear capacity of the concrete.

To fully understand the behaviour of PKS concrete in shear, it is of importance that the effect of shear reinforcement on the behaviour of PKS RC beams in shear be investigated. Knowledge of the post-peak deformation characteristics of shear-critical reinforced concrete members of PKS lightweight concrete is important to better understand the contribution of the shear (web) reinforcement and the failure mechanisms for structural applications. This study is therefore aimed at studying the effect of the amount of shear reinforcement on the shear strength of PKS and NWC RC beams. To this end, the code compliant behaviour of PKS RC beams in shear was investigated using BS 8110, ACI 318 and EC 2 to determine their suitability as shear models for LWC.

## Prediction of shear capacity of RC beams

The behaviour of RC elements in flexure has been well understood such that their flexural strengths can be predicted with reasonable accuracy. Contrary to the flexural behaviour of structural members, sophisticated approaches have been proposed based on fracture mechanics, physical models of structural failure, and finite element analyses (Song et al., 2010; Oreta, 2004) to predict the behaviour of beams in shear. That notwithstanding, no single theory is available for estimating the precise shear strength of reinforced concrete elements (Bentz et al., 2006). This problem has been attributed to the complex nature associated with the shear transfer mechanisms especially after cracks are initiated (Song et al., 2010). As a result, the design and

**Table 1.** Different approaches to Shear design.

| Approach | Shear Strength, Vc | Shear Strength, Vs |
|---|---|---|
| BS 8110 - 97 | $V_c = \frac{0.79}{\gamma}\left[\left(\frac{100A_s}{b_v d}\right)^{1/3} \times \left(\frac{400}{d}\right)^{\frac{1}{4}} \times \left(\frac{f_{cu}}{25}\right)^{1/3}\right] bd$ | $V_S = \frac{A_{sv}}{S_v} \times 0.95 f_y d$ |
| EC 2 - 2004 | $V_{RD,\,C} = \left[0.12\left(1 + \sqrt{200/d}\right)(100\rho_1 f_{ck})^{1/3} + \frac{0.1N_{ED}}{A_c}\right] bd$ | $V_{RD,S} = \frac{A_{sw}}{S} Z f_{ywd}\, Cot\theta$ |
| ACI 318 - 08 | $V_c = \left[0.16\lambda\sqrt{f_c'} + 17\rho_w \frac{V_u d}{M_u}\right] b_w d$ | $V_S = \frac{A_V f_{yt}\, d}{S}$ |

$V_c$: Shear strength provided by concrete; $f_c'$, $f_{ck}$, $f_{cu}$: Concrete compressive strength; $b_w$: Web width; d: Effective depth; $V_u$: Shear force; $M_u$: External moment; $\rho_1$, $\rho_w$: Longitudinal reinforcement ratio; $N_{ED}$: Axial force; $A_c$: Cross sectional area of concrete; $\lambda$: modification factor reflecting the reduced mechanical properties of lightweight concrete; Z: Lever arm; $f_{yt}$: tensile strength of longitudinal reinforcement; $V_s$: shear strength contributed by shear reinforcement.

behavior of structural concrete to shear is an important and ongoing area of research in structural concrete. However, research has revealed that the resistance of reinforced concrete members to shear is the summation of several internal shear transfer mechanisms (NCHRP, 2005). Some (Hassan et al., 2008; Taylor, 1974) suggest that transfer mechanisms include shear in the uncracked compression zone of the beam (ranging between 20% and 40%), aggregate interlock mechanism or interface shear transfer (ranging between 35% and 50%), dowel action of the longitudinal reinforcement (ranging between 15% to 25%), whilst others Jung and Kim (2008) attribute the transfer to other factors such as residual tensile stresses across the crack, and the presence of shear reinforcement. It is therefore believed that a major component of the shear transfer in the fractured interface is generated from the frictional forces that develop across the diagonal shear cracks due to "aggregate interlock" which provides resistance against slip (Hassan et al., 2008). Generally, tensile stresses due to external loading will exceed the tensile capacity of the concrete, leading to flexure cracks. Once these cracks form, the beam no longer responds to loads in an elastics manner. Shear is then transferred through the intact concrete in the compression zone above the flexure cracks. If the shear span-to-depth ratio is small, arching action and a compression strut may also develop to carry part of the shear.

Most design codes however use empirical equations for simplicity. Some design codes like the ACI limits the shear capacity of a member to the onset of diagonal cracking of the beam, regardless of the ultimate shear load which may usually be higher than the cracking load. This is because the extent to which a member continues to resist shear after the onset of diagonal cracking is unpredictable and depends largely on the loading configurations and the material properties (Juan, 2011). Considering the complex nature of shear resistance, design codes consider the shear capacity of concrete members from two mechanisms: design shear capacity of the concrete without shear reinforcement, $V_c$ and the contribution of shear reinforcement, $V_s$ as shown in Table 1.

For lightweight aggregate concrete (LWAC) members, (BS 8110-2, 1985) adopts the same design parameters as that of normal weight concrete members for concrete grades greater than 25 MPa. Meanwhile the concrete's design stress ($V_c$) is taken as 0.8 times the values for normal weight concrete if the grade of concrete is greater than 25 MPa. This factor is also imposed on the maximum limit of shear stress that a section can be subjected to. That is $0.63f_{cu}$ or 4 MPa whichever is lower.

The design for shear using the (ACI 318, 2008) is based on the computation of the shear strength, $V_c$, of the concrete beam cross-section and the maximum shear, $V_u$, that the beam will resist. A reduction factor of 0.85 is adopted for sand-lightweight aggregate concrete while a factor of 0.75 is assumed for all-lightweight aggregate concrete.

Contrary to the design provisions of ACI 318, the provisions of the EC2 consider a semi-empirical equation for members which do not require shear reinforcement. This equation considers size effect of beams instead of the shear span to depth ratios. Values of $C_{rdc}$ for lightweight concrete is also reduced from $0.18/\gamma_c$, for normal weight concrete to $C_{rdc} = 0.15/\gamma_c$ for lightweight concrete. Another reduction factor which depends on the density of the lightweight aggregate concrete used is also imposed on lightweight concretes. The coefficient is taken as 60% of the normal weight concrete by the ratio of the upper limit of the appropriate density class to the

**Table 2.** Physical properties of aggregates.

| Properties | PKS (LWA) | Granite (NWA) | Limits |
|---|---|---|---|
| Maximum aggregate size, mm | 14 | 14 | 0 |
| Shell thickness, mm | 1 – 5.9 | - | 0 |
| Specific gravity, saturated surface dry | 1.35 | 2.65 | < 2.4 - 2.8 |
| Aggregate impact value (AIV), % | 3.01 | 13.5 | ≤ 25 |
| Aggregate crushing value (ACV), % | 5.30 | 25.7 | ≤ 30 |
| Los Angeles Abrasion Value (AAV), % | 4.73 | 19.6 | ≤ 30 and ≤ 50 |
| 24-hour water absorption, % | 18 | 0.68 | 25% |
| Flakiness Index (%) | 63.2 | 31 | 40 – 50 |
| Elongation Index (%) | 16.6 | 22 | 40 – 50 |

**Table 3.** Mix proportions of concrete.

| Mix designation | Mix ratio | | | |
|---|---|---|---|---|
| | Cement content, Kg/m$^3$ | w/c | Cement/sand/aggregates | Superplasticizer (%) |
| PKSC, P | 550 | 0.40 | 1:1.3:0.70 | 1 |
| NWC, N | 440 | 0.50 | 1:1.7:2.5 | 0 |

**Table 4.** Properties of beam specimens and concrete strengths.

| Beam designation | Beam size B x D (mm) | Effective depth d (mm) | Age at testing (days) | Compressive strength, $f_c$ (N/mm$^2$) | Flexural strength, $f_r$ (N/mm$^2$) | Spacing of stirrups (mm) |
|---|---|---|---|---|---|---|
| P0 | 110 × 225 | 193 | 28 | 23.6 | 3.60 | - |
| P200 | 110 × 225 | 193 | 28 | 22.6 | 3.70 | 200 |
| P250 | 110 × 225 | 193 | 28 | 23.4 | 3.71 | 250 |
| P300 | 110 × 225 | 193 | 28 | 23.1 | 3.45 | 300 |
| N0 | 110 × 225 | 193 | 28 | 28.7 | 4.1 | - |
| N200 | 110 × 225 | 193 | 28 | 29.5 | 4.1 | 200 |
| N250 | 110 × 225 | 193 | 28 | 30.3 | 4.3 | 250 |
| N300 | 110 × 225 | 193 | 28 | 29.8 | 4.3 | 300 |

density of normal weight aggregate concrete (2200 kg/m$^3$). The reduction factor becomes $\eta_1 = 0.40 + {}^{0.60\rho}/_{2200}$.

## EXPERIMENTAL PROCEDURE

### Materials and beam identification

Ordinary Portland cement with a 28-day compressive strength of 42.5N was used in the study. The fine aggregate used in the study was river sand. The coarse aggregates were crushed palm kernel shells (PKS) with a maximum size of 12.5 mm (for PKS concrete) and granite (for the normal weight concrete). The shells were flushed with water to remove dust and other impurities. The aggregates were oven dried and the physical properties were determined (Table 2). Due to the high water absorption properties of the PKS aggregates (Table 2), they were pre-soaked for 24 hours and subsequently air dried prior to its use. Mix proportions of the PKS concrete was obtained from trial mixes due to the comparatively different nature of the PKS aggregates (Table 2), to obtain a target design strength of 25 N/mm$^2$. Meanwhile, the granite concrete was designed for a target strength of 25 N/mm$^2$ using the UK's Department of Environment (1998) mix design method. Sika viscocrete, high-range water reducing admixture (superplasticizer) was used to improve the workability of the PKSC mix since the water/cement (w/c) was kept low to obtain the target strength. The mix proportions of both PKSC and NWC are presented in Table 3.

Eight reinforced concrete beams (four PKS and four granite reinforced concrete beams) were cast and tested in the Civil Engineering Concrete Laboratory of KNUST. Four of these beams were made of PKS concrete (P0, P200, P250 and P300) and the other four, normal granite concrete (N0, N200, N250 and N300). All the beams were of the same length of 2000 mm with the same cross-sectional dimension of 110mm (width) and 225 mm (depth). Each beam is designated by a letter and a number. The letters (P or N) denotes the type of concrete whilst the number (0, 200, 250 or 300) denotes the spacing of shear stirrups (Table 4). Each concrete

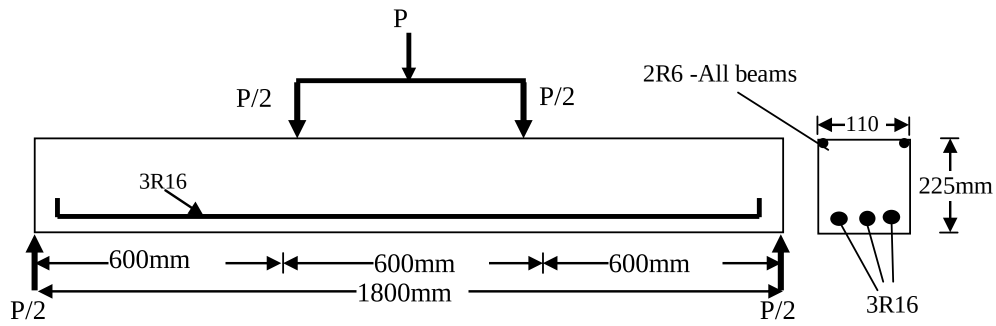


**Figure 1.** Beams without shear reinforcement, P0 and N0.

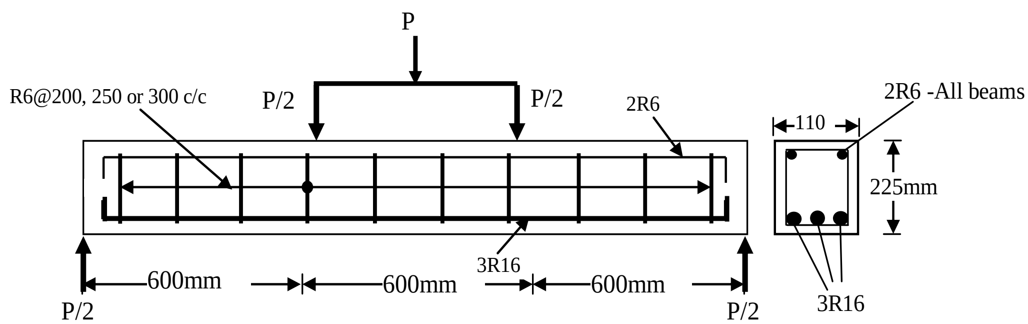


**Figure 2.** Beams with shear reinforcement (P200, P250, P300 & N200, N250, N300).

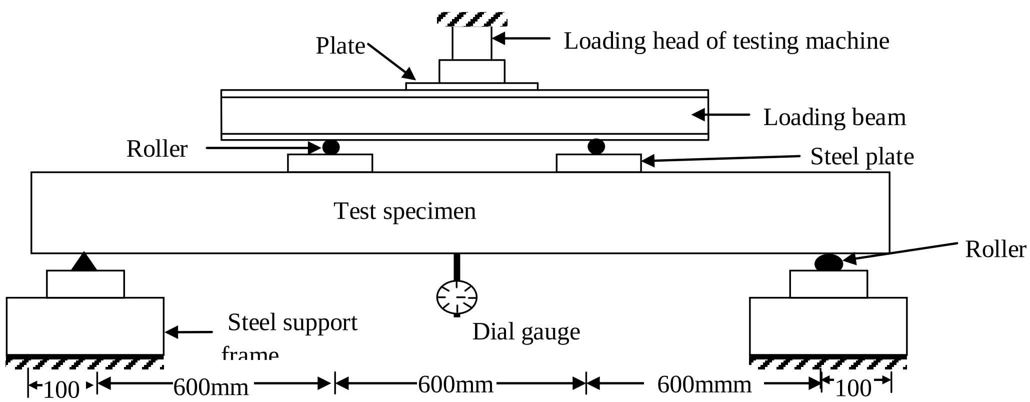


**Figure 3.** Schematic sketch of experimental set-up.

type had one beam (P0 and N0) without web reinforcement. The other three beams of each type of concrete (P200, P250, P300, N200, N250 and N300) had web reinforcement in the spacing of 200, 250 and 300 mm, respectively. Both categories of beams were provided with nominal hanger bars of 2R6. Deformed mild steel bars of mean yield strength 271.2 N/mm$^2$ were used for the longitudinal reinforcement and shear reinforcement (stirrups). The geometrical properties and reinforcement details of the beams are shown in Figures 1 and 2. Companion concrete specimens of 150 × 150 × 150 mm and 100 × 100 × 500 mm were cast to study the compressive and flexural strengths respectively. Curing was done using hessian mat spread on the beams in the open temperature with regular watering until 28-days.

**Testing of beams**

The beams were simply supported on a stiff steel frame in the Civil Engineering Laboratory of the KNUST, Kumasi. A hydraulic actuator under crosshead displacement control was used to apply loads through a steel spreader beam. A four point loading configuration (Figure 3) was used for the test. The spreader beam had sufficient bending capacity and stiffness to avoid excessive deformation and yielding before failure of the test specimens.

Beam deflections at mid-span for a steady loading rate of 0.2 kN/s were measured with the aid of a dial gauge with a 0.001 mm accuracy fixed at the bottom of each beam. Cracks were marked on the sides of the specimens as they developed, in order to assess

**Table 5.** Test results of cracking loads and service load crack widths.

| Beam no. | Cracking loads, kN | | | | Service Loads, $V_{sl}$ (kN) | Crack width at service loads, (mm) | Crack width at failure, (mm) | No. of cracks at failure | Ultimate deflection, $\delta_c$ (mm) | Failure modes |
|---|---|---|---|---|---|---|---|---|---|---|
| | First flexural crack, $V_{fc}$ | First shear crack, $V_{cr}$ | Ultimate shear force, $V_u$ | Post diagonal crack resistance | | | | | | |
| P0 | 10 | 28 | 50 | 22 | 33.33 | 0.22 | 0.29 | 13 | 6.86 | FS/DT |
| P200 | 12 | 42 | 72 | 30 | 48.00 | 0.24 | 0.32 | 24 | 12.46 | FS/CC |
| P250 | 10 | 38 | 68 | 30 | 45.33 | 0.215 | 0.34 | 23 | 11.60 | FS/DT |
| P300 | 8 | 30 | 62 | 32 | 41.33 | 0.235 | 0.36 | 21 | 9.55 | FS/DT |
| N0 | 14 | 40 | 60 | 20 | 40.00 | 0.245 | 0.31 | 22 | 6.22 | FS/DT |
| N200 | 16 | 50 | 74 | 24 | 49.33 | 0.255 | 0.33 | 19 | 13.48 | FS/DT |
| N250 | 14 | 48 | 72 | 24 | 48.00 | 0.24 | 0.45 | 18 | 8.05 | FS/DT |
| N300 | 12 | 42 | 66 | 24 | 44.00 | 0.32 | 0.72 | 16 | 7.95 | FS/DT |

FS: Flexural shear; DT: Diagonal tension; CC: Concrete crushing.

the first flexural and shear cracks, and crack widths at tension steel levels. Observation of cracking was performed visually while the crack propagation and crack pattern were marked by hand. Selected crack widths were measured using a crack microscope of optical magnification X10 and reading to 0.02 mm. Initiation and propagation of both flexural crack and shear cracks were closely observed and recorded against corresponding applied loads.

## RESULTS AND DISCUSSION

### Properties of aggregates

The results of the physical properties of both PKS and crushed granite aggregates are presented in Table 2. The physical properties of the aggregates conformed to the minimum requirements of (BS 882, 1992), however, the PKS aggregates appear stronger than the granite aggregates used. The results of the densities and mechanical properties such as compressive and flexural strength properties of the PKS concrete and normal weight concrete beams tested at the age of 28 days are presented in Table 4. The results show that the compressive strength of PKS concretes are higher than the minimum required strength of 17 N/mm$^2$ for structural lightweight concrete (ASTM C330, 1999). Considering the physical properties, the lower compressive strength could imply that the compressive strength of the PKS ultimately depends on the bond between the aggregates and the cement matrix. Thus, improving the bond between the PKS aggregates will result in increased compressive strength of the PKS concrete.

### General behaviour of beams

Table 5 shows the values for cracking loads, ultimate loads and the maximum deflections. In all beam specimens, flexural tension cracks propagated first in the pure bending zone followed by shear cracks in the shear zone. Flexural cracks initiated at 10 kN which represent 20% of ultimate load for the palm kernel shell concrete beams without shear reinforcement (P0) compared to 14 kN (23% of ultimate load) for the normal weight concrete beams without shear reinforcement (N0). First flexural cracks varied from 12.9 to 16.7% of the ultimate for PKS concrete beams with shear reinforcement (P200, P250 and P300). First flexural cracks for the NWC beams with shear reinforcement (N200, N250 and N300) varied from 18.2 to 21.6% of the ultimate load. It is reported that the type of aggregate in concrete can influence the characteristics of the reinforced concrete members in shear (Adom-Asamoah et al., 2012). This was evident as failure loads of NWC beams were comparatively greater than those of PKS concrete beams (Table 4). In all beams, it was seen that increasing the amount of shear reinforcement (stirrups) led to an enhancement of cracking strength of the beams as observed by others researched on other non-conventional aggregates for concrete (Adom-Asamoah and Afrifa, 2013).

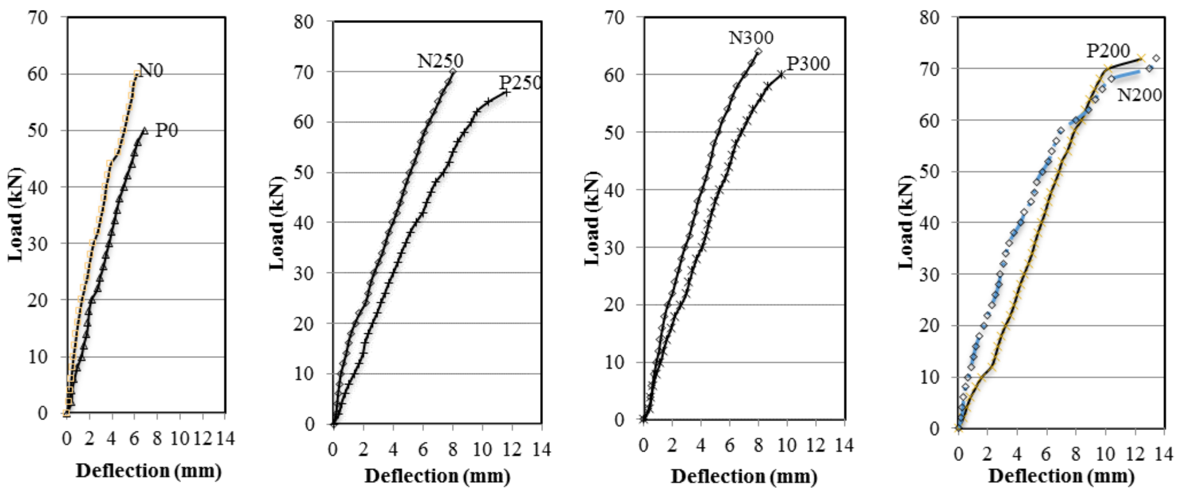


**Figure 4.** Load-deflection curves for PKSC and NWC beams.

## Load-deflection, cracking and failure behaviour

First flexural cracks were observed at lower loads among the PKS concrete beams when compared to the NWC beams. This indicates a lower bending strength of PKS concrete beams as shown by the results of the modulus of rupture test (Table 4). In all instances, the ultimate deflections for PKS concrete beams were higher than corresponding NWC concrete beams (Table 5). This may be attributed to the good energy absorbing quality of the PKS aggregates derived from the low AIV and ACV typical of PKS compared to NWC (Teo et al., 2007). The initial straight portion of the curve, before first crack, indicated that the pre-cracking stiffness of the beams were always higher than those of PKS independent of the type of beam and amount of shear reinforcement. At this stage, the beams behaved elastically until the onset of cracks, after which shear is transmitted through the concrete section above the flexural cracks. The onset of cracking in the concrete is thus controlled by the bending strength of the material itself and the compressive strength of the concrete. The beams without stirrups (P0 and N0) failed at a much lower load and had significantly less deflection than the beams with shear reinforcement (Table 5). Shear transfer through the shear reinforcement was activated once the concrete section was no longer able to resist the applied stresses. Consequently, the ultimate deflections were proportional to the amount of shear reinforcement (Figure 4).

The cracking loads and corresponding service loads and crack widths are summarized in Table 5. Generally, all beams failed in shear mode except P200 which failed as a result of flexural shear. Cracking of PKS beams were along the convex surfaces of the shells which shows a weak bond between the PKS aggregates and the cement matrix. The PKS concrete beams exhibited early flexural cracks compared to the companion NWC beams. This could be attributed to the lower modulus of rupture of the PKS beams (Table 4). Diagonal shear loads varied from 48.4 to 58.3% of ultimate loads for PKS concrete beams and 63.6 to 67.6% of ultimate loads for NWC for beams with shear reinforcement. The post-diagonal cracking shear resistance of PKS concrete beams without shear reinforcement were comparatively higher than that of corresponding NWC beams (Table 5). Diagonal shear cracks were observed at about 56% of the ultimate load for the PKS concrete as compared to 66.7% of ultimate loads for corresponding NWC beams. At the post-diagonal cracking, beams without shear reinforcement derive their shear resistance from aggregate interlock mechanisms, and the dowel action between the aggregates and the longitudinal reinforcement. The results show good aggregate interlock in the PKS beams compared to the NWC beams, thereby contributing to the higher post-diagonal cracking shear resistance. More so, it could be deduced from Table 2, the palm kernel shell concrete were able to absorb more loads even after cracking as compared to the normal weight concrete probably due to the lower ACV of 5.3%. In addition, after the post-diagonal cracking, the good bonding between concrete and the longitudinal reinforcement resulted in the transfer of stresses to the steel; leading to the utilization of the tensile strength of the steel to achieve equilibrium in the composite section (Nejadi and Gilbert, 2004).

The number of cracks in PKS concrete beam without shear reinforcement (P0) was 13 compared with 22 cracks in NWC (N0). The average number of cracks in the PKS beams with shear reinforcement (P200, P250 and P300) were greater than those of corresponding NWC beams. The amount of shear reinforcement was directly proportional to the number of cracks in both types of concrete specimens at failure. As observed by (Lim et al., 2006; Hassan et al., 2008) a higher number of cracks

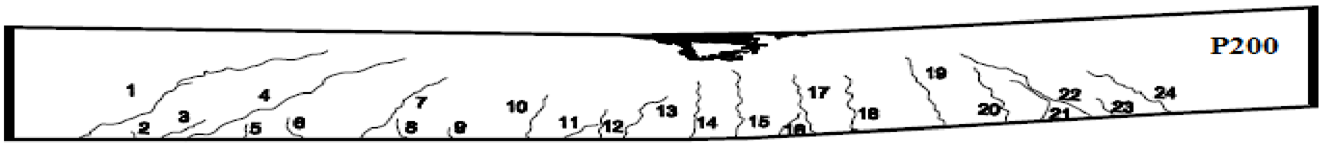


**Figure 5.** Crack patterns and flexural shear failure of Specimen P200.

resulted in smaller crack widths for both types of concrete beams. It is also seen from Table 5 that PKS concrete beams developed smaller crack widths compared to that of corresponding NWC beams. The wider cracks in NWC can be attributed to physical characteristics of the normal weight aggregate as aggregate interlock plays a significant role in shear transfer across a diagonal crack (Adom-Asamoah and Afrifa, 2013). It was expected that the higher deflections in the PKS beams coupled with the smooth convex surfaces would result in wider crack openings along the convex surfaces. However, the required friction developed by the smooth crack surface of the PKS aggregate concrete for shear transfer, is lost at a lower crack width compared to normal weight concrete. Since the developed friction is low, the crack slips before yielding of the shear reinforcement (Duthinh, 1997), thereby leading to a brittle failure. This could have contributed to the higher post-diagonal cracking resistance but lower ultimate failure loads. Cracks occurring within concrete do not only reduce the stiffness of structural members (Duthinh, 1997) but also make it weak with age due to exposure of structural steel to the environment (Parghi et al., 2008). Therefore, crack widths have to be controlled to acceptable limits in reinforced concrete members for a variety of reasons, such as, control of deflection, corrosion protection, impermeability, maintenance of integrity and appearance of the structure.

## Failure modes

Table 5 presents various modes of failure for all specimens. The mode of failure of the test specimens was brittle in nature with virtually no warning. With the exception of P200 which failed as result of concrete crushing, all beam specimens failed as a result of diagonal tension/flexural-shear. Ahmed et al. (1995) noted that the diagonal tension (shear) failure of concrete is brittle in nature and fails with little or no warning especially the failure of beams reinforced only with flexural (tensile) reinforcement. Typically, the concrete rupture completely with wide diagonal cracks which are widest in the flexural tensile zone of the beam. Ultimate failure of all beams occurred with the material rupturing along a fresh diagonal crack parallel and independent from earlier formed cracks. The beams remained relatively stable after the formation of the diagonal crack, which run between the inner edges of the supports (Sagaseta and Vollum, 2010). Diagonal cracks were far from the mid-span of the beam, and splitting the bond of the longitudinal reinforcement all the way to the point of loading. It was observed that initial diagonal tension cracking occurred at a low load in the PKS beams which is related to the flexural strength of the concrete (Juan, 2011). That notwithstanding, PKS concrete beams were able to develop sufficient shear capacity from other mechanisms to continue resisting increasing loads. In addition to the diagonal shear failure, the NWC showed bond failure at the tension side of the beams. This could be attributed to the high stress concentration near the support. It is reported that for shear span/effective depth ($a_s$/d) ratio greater than 1 but less than 2.5, the diagonal crack often forms independently and not as a result of flexural crack development (Sagaseta and Vollum, 2010; Kong and Evans, 1994).

Beam specimen P200 failed in a ductile manner and as result of concrete crushing/flexural shear (Figure 5). Vertical flexural cracks propagated from the tension side of the beam to the compression zone. Crushing of the concrete occurred with associated disintegration of the concrete cover in the compression zone. Once the concrete crushed, the beam continued to resist shear stresses through the shear reinforcement and the dowel action of the longitudinal reinforcement until the longitudinal reinforcement yielded. Specimen P200 contained the greatest amount of shear reinforcement for the PKS beams in this study. Given the low compressive strength (Table 3), P200 failed as a result of flexural shear. This implies that the increased shear capacity (resulting from high shear reinforcement ratio) of the PKS beams led to the flexural shear before reaching its full shear capacity. Jumaat et al. (2009) observed that oil palm shell foamed concrete (OPSFC) beams with 150 mm centre to centre web reinforcement failed in flexure compared to OPSFC beams without shear reinforcement which failed in shear modes. Thus increasing shear reinforcement in reinforced PKS concrete beams results in flexural failure mode instead of shear mode.

## Shear resistance characteristics of PKS and NWC beams

To analyze and compare the shear strength of beams, the ultimate shear load ($V_u$) is normalized to account for the difference in compressive strength between SCC and

**Table 6.** Experimental results and code predictions.

| Beam ID | Exp. Shear force (kN), $V_{exp}$ | Code predictions – $V_{code}$ | | | Shear force ratios, $V_{code}/V_{exp}$ | | |
|---|---|---|---|---|---|---|---|
| | | BS 8110 | EC 2 | ACI 318 | BS 8110 (%) | EC 2 (%) | ACI 318 (%) |
| P0 | 50 | 36.48 | 38.95 | 36.04 | 73 | 78 | 72 |
| P200 | 72 | 62.40 | 63.51 | 48.05 | 87 | 88 | 67 |
| P250 | 68 | 57.22 | 58.60 | 45.65 | 84 | 86 | 67 |
| P300 | 62 | 53.76 | 55.32 | 44.04 | 87 | 89 | 71 |
| N0 | 60 | 45.60 | 49.05 | 41.90 | 76 | 82 | 70 |
| N200 | 74 | 71.52 | 73.60 | 53.90 | 97 | 99 | 73 |
| N250 | 72 | 66.34 | 68.69 | 51.50 | 92 | 95 | 72 |
| N300 | 66 | 53.76 | 63.28 | 49.90 | 81 | 96 | 76 |

NC. Since the shear strength is proportional to the square root of the compressive strength of concrete ($f_c$) the normalized shear load ($V_n$) was determined as follows:

$$V_n = \frac{V_u}{\sqrt{f_c}} \tag{1}$$

The normalized shear stress ($V_{ns}$) is then calculated as:

$$V_{ns} = \frac{V_n}{bD} \tag{2}$$

Normalized shear load and stress for all experimental PKSC/NWC beams are tabulated in Table 7.

In general, PKSC beams with shear reinforcement exhibited higher *Vns* compared to NWC beams with shear reinforcement. The normalized shear stress, *Vns* also decreased with increasing spacing of shear reinforcement for both NWC and PKSC beams. The normalized shear stress of NWC beams without shear reinforcement was however, found to be higher than corresponding PKSC beams with varying spacing of shear reinforcement (Figure 6).

## Comparison of test results and some existing code shear models

Table 6 compares the experimental results and theoretical loads calculated using equations in Table 1. The performance of BS 8110, EC 2 and ACI 318-08 in predicting the ultimate shear load of both PKSC and NWC beams with and without shear reinforcement have also been presented. It is seen that ACI 318-08 under predicted the ultimate shear capacity of both PKSC and NWC beams irrespective of the amount of shear reinforcement. The ratio of $V_{(ACI\ 318)}/V_{exp}$ ranges between 67 and 76% for PKSC and NWC beams with shear reinforcement.

The study shows that the conservativeness of the ACI increases with decreasing amount of shear reinforcement, and can be used to safely to predict the ultimate shear resistance of both PKSC and NWC beams. The ratio of BS 8110 prediction to experimental values ranges between 84 to 87% for PKSC beams with shear reinforcement and ranges between 81 and 97% for NWC beams with shear reinforcement. It is seen that, both ACI 318 and BS 8110 are conservative, especially for the palm kernel shell concrete beams. However, the ACI is found to be more conservative than the BS 8110. The ratio of predicted EC 2 values to the experimental values ranges from 86 to 89% for the PKSC concrete beams and ranges from 95 to 99% for the NWC beams. EC 2 is found to closely predict the ultimate shear capacity of the PKSC and NWC beams compared to that of the ACI and the BS 8110. For NWC beams without shear reinforcement, the predictions are 70%, 76% and 82% of the shear capacity of the NWC beams for the ACI, BS 8110 and EC 2 respectively. The predictions of PKS beams without shear reinforcement are 72, 73 and 80% of the shear capacity of the NWC beams for the ACI, BS 8110 and EC 2, respectively.

The shear stress ratios of experimental values to BS 8110 predicted values range from 1.15 to 1.19 for PKSC beams with shear reinforcement while the ratio range from 1.04 to 1.23 for NWC beams with shear reinforcement. The shear stress ratios of the experimental values to EC2 predicted values range from 0.98 to 1.11 for PKSC beams

**Table 7.** Shear stress prediction of various design codes.

| Beam ID | Normalized shear stress, $V_{ns}$ | | | | Shear stress ratios ($V_{exp}/V_{code}$) | | |
|---|---|---|---|---|---|---|---|
| | $V_{Exp}$ | $V_{BS8110}$ | $V_{ACI\ 318}$ | $V_{EC2}$ | $V_{BS8110}$ | $V_{ACI\ 318}$ | $V_{EC2}$ |
| P0 | 0.42 | 0.30 | 0.30 | 0.33 | 1.40 | 1.40 | 1.27 |
| P200 | 0.61 | 0.53 | 0.41 | 0.55 | 1.15 | 1.49 | 1.11 |
| P250 | 0.57 | 0.48 | 0.38 | 0.50 | 1.19 | 1.50 | 0.98 |
| P300 | 0.52 | 0.45 | 0.37 | 0.47 | 1.16 | 1.41 | 1.11 |
| N0 | 0.45 | 0.34 | 0.32 | 0.37 | 1.32 | 1.41 | 1.22 |
| N200 | 0.55 | 0.53 | 0.40 | 0.55 | 1.04 | 1.38 | 1.00 |
| N250 | 0.53 | 0.49 | 0.38 | 0.50 | 1.08 | 1.39 | 1.06 |
| N300 | 0.49 | 0.40 | 0.37 | 0.47 | 1.23 | 1.32 | 1.04 |

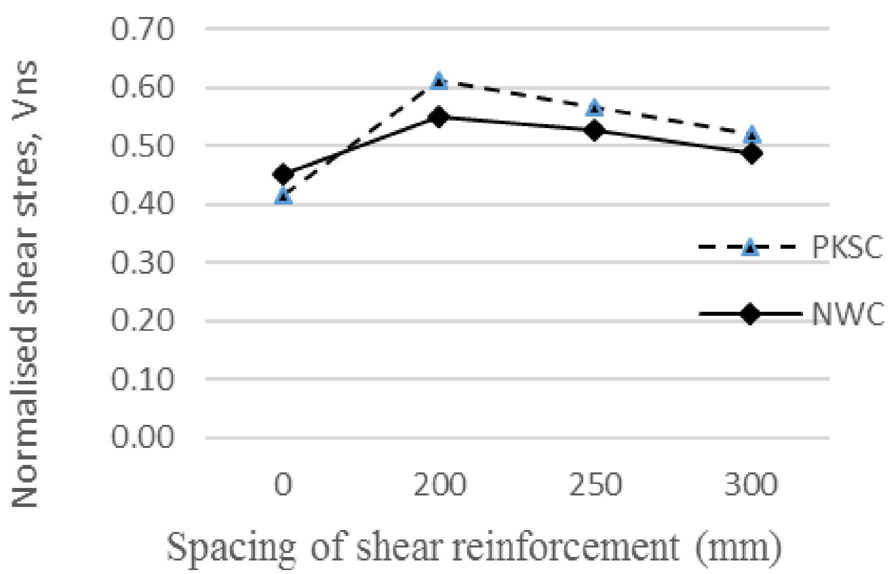


**Figure 6.** Effect of spacing of shear reinforcement on normalized shear stress (Vns).

without shear reinforcement. The variation of the shear stresses in relation to the amount of shear reinforcement was found to be non-proportional. That notwithstanding, the shear stresses decreased with decreasing amount of shear reinforcement. As seen from Figure 7, the experimental shear stress results were higher than the code predictions, indicating that the codes are more conservative with the design of reinforced PKSC beams.

## Conclusion

The shear resistances of NWC and PKS concrete beams with and without shear reinforcement were described based on the experimental results. The results of these tests are discussed with particular attention to cracking loads, maximum crack width, deflection, ultimate loads and the effect of shear reinforcement on both PKS concrete and NWC beams. The experimental failure loads are also compared with the predictions of the methods contained in the BS 8110, ACI 318-08 and the EC 2. From the study, the following conclusions are drawn on the behaviour of PKS concrete beams in shear.

The general behaviour of PKS concrete beams in shear is comparable to that of an equivalent NWC beam. The addition of shear reinforcement showed a higher enhancement of the shear capacity of the PKS beams compared to corresponding NWC beams. The failure modes of PKS concrete beams were brittle in nature as a result of diagonal tension. The post-diagonal shear resistance of PKSC beams are higher than that of corresponding NWC beams, especially beams with shear reinforcement. Nevertheless, the ultimate shear strength of NWC beams were higher than that of corresponding PKS concrete beams. EC 2 gives the closest prediction of the shear capacity of PKSC beams compared to the shear models of BS 8110 and ACI 318-08 which tends to under-estimate the shear capacity of PKS concrete

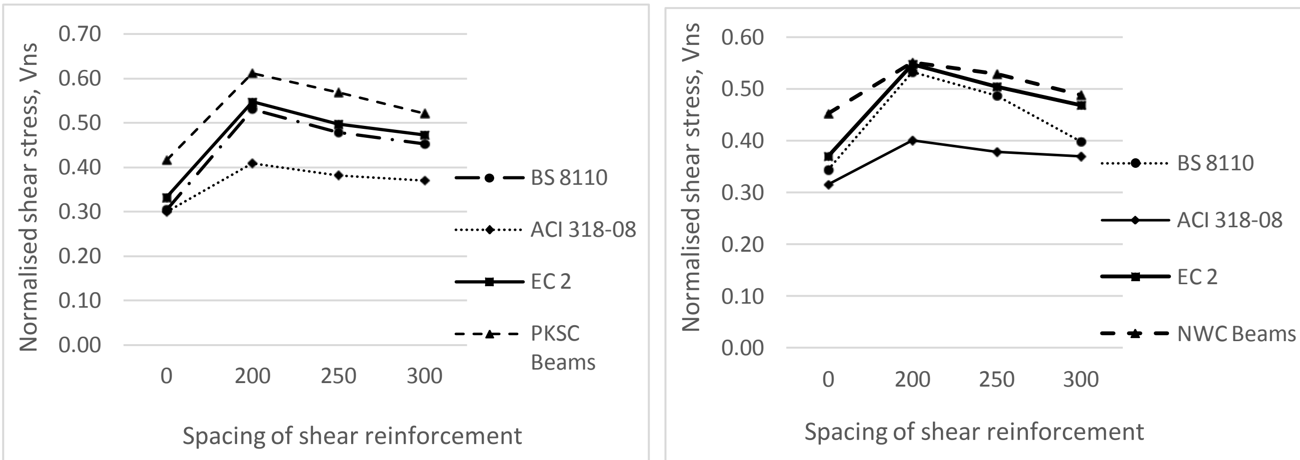


**Figure 7.** Comparison of shear stress (*Vns*) distributions with PKSC and NWC beams.

beams especially PKS beams without shear reinforcement. The results of the study show that PKS has good potential as a coarse aggregate for the production of structural lightweight concrete for structural application construction.

## Conflict of Interest

The author(s) have not declared any conflict of interests.

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