Materials and Design 53 (2014) 325–331

Contents lists available at SciVerse ScienceDirect

Materials and Design

journal homepage: www.elsevier.com/locate/matdes

Flexural strength of palm oil clinker concrete beams

Bashar S. Mohammed [a,*], W.L. Foo [b], M. Abdullahi [c]

[a] Department of Civil Engineering, Universiti Teknologi PETRONAS, Bandar Sri Iskandar, 31750 Tronoh, Perak Darul Ridzuan, Malaysia
[b] Civil Engineering Department, College of Engineering, Universiti Tenaga Nasional, km-7, Jalan Kajang-Puchong, 43009 Kajang, Selangor, Malaysia
[c] Department of Civil Engineering, Federal University of Technology, Minna, Nigeria

ARTICLE INFO

Article history:
Received 1 April 2013
Accepted 11 July 2013
Available online 22 July 2013

Keywords:
Palm oil clinker
Lightweight concrete
Reinforced concrete beams
Flexural behaviour

ABSTRACT

This paper presents an experimental program on the flexural behaviour of reinforced concrete beams produced from palm oil clinker (POC) aggregates. POC is obtained from by-product of palm oil milling. Utilisation of POC in concrete production not only solves the problem of disposal of this solid waste but also helps to conserve natural resources. An experimental work was conducted involving eight under-reinforced beams with varying reinforcement ratios (0.34–2.21%) which were fabricated and tested. The data presented include the deflection characteristics, cracking behaviour and ductility indices. It was found that although palm oil clinker concrete (POCC) has a low modulus of elasticity, the test results revealed that the deflection of singly reinforced POCC beams, with reinforcement ratio less than 0.524, under the design service load is acceptable as the span-deflection ratios range between 250 and 257 and these values are within the allowable limit provided by BS 8110. In addition, the results reported in this paper indicate that the BS8110 based design equations can be used for the prediction of the flexural capacity of POCC beams with reinforcement ratio up to 2.23%.

© 2013 Elsevier Ltd. All rights reserved.

1. Introduction

Increase in population has created greater demand on construction material which leads to a chronic shortage of building materials and thereby increases the construction cost [1]. To alleviate this problem, engineers are not only challenged with the future home-building in terms of construction cost control but also the need to convert the industrial wastes to useful construction and building materials [2]. One of such ways is to introduce industrial waste material into concrete. Such waste materials are fly ash, wood chipping, paper mill, crumb rubber, silica fume and palm oil clinker. The utilisation of these waste reduce the use of aggregate from natural sources and ensures sustainability.

Malaysia is one of the world largest producers of palm oil and generate significant amount of waste in the milling process [3]. The large amount of waste produced is one of the main contributors to the nation's pollution problem. Palm oil mill in Malaysia incinerate palm oil waste to produce steam needed for the milling process. The waste product of incineration is palm oil clinker (POC) [4]. Instead of dumping the POC into environment, a better waste management option is to crush POC into desired sizes (fine and coarse aggregate) and utilise it as aggregate to produce lightweight concrete [5]. The advantages of lightweight concrete have been identified by several researchers which include reduction in building cost, ease of construction, thermal and acoustic insulation, fire resistance, reduction in building weight, and as a mean of disposal of waste [6].

Extensive research works were carried out to produce lightweight concrete utilising POC aggregate as full replacement to conventional fine and coarse aggregates. The physical and mechanical properties of the palm oil clinker concrete (POCC) have been established [3]. The developed POCC easily attains compressive strength of more than 17 MPa, which is the requirement for structural lightweight concrete as stated in the ASTM:C330 [4]. Lightweight POCC is still a relatively new construction material and the structural performance of the concrete has not yet been investigated. Therefore, for structural applications, the flexural behaviour of POCC beams has to be closely examined and clearly established.

2. POC aggregate

The POC used in this study was obtained from local palm oil mill. Fig. 1 shows the flow diagram to process POC. It was crushed and sieved to the desired particle sizes. Particles less than 5 mm are considered as fine aggregate and particles in the range of 5–14 mm are considered as coarse aggregate.

The result of sieve analysis is shown in Fig. 2. For fine aggregate the percentage by mass passing sieves 1.18 mm (No. 16), 300 μm (No. 50), and 150 μm (No. 100) are 50.77%, 15.37% and 10.12%

* Corresponding author. Tel.: +60 53687305.
E-mail addresses: bashar.mohammed@petronas.com.my, bashar_sami@hotmail.com (B.S. Mohammed).

0261-3069/$ - see front matter © 2013 Elsevier Ltd. All rights reserved.
http://dx.doi.org/10.1016/j.matdes.2013.07.041

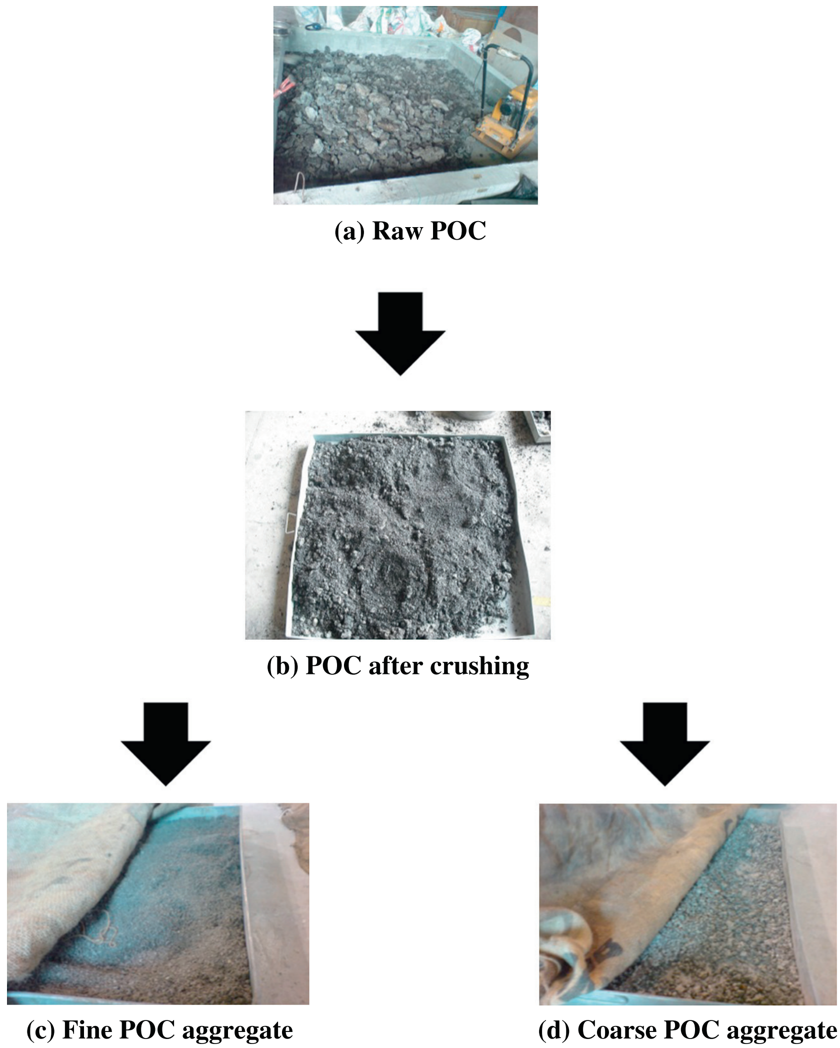

Fig. 1. The fine and coarse POC aggregate [3].

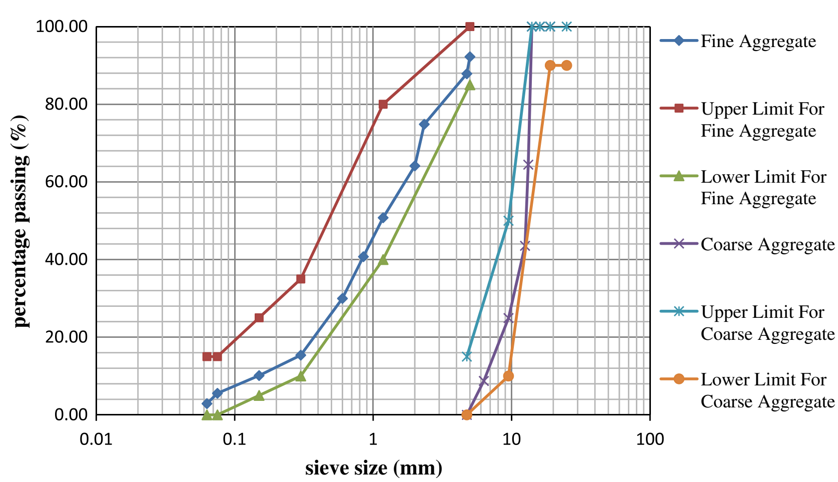

Fig. 2. Sieve analysis of fine and coarse POC [3].

respectively. For coarse aggregate the percentage by mass passing sieves 19.0 mm (3/4 in.), 9.5 mm (3/8 in.), and 4.75 mm (No. 4) are 100%, 24.93% and 0%, respectively. These experimental values are in accordance with grading requirements for lightweight aggregate for structural concrete as per ASTM:C330. The result shows that POC is well graded and suitable for use in concrete work. The pore space of the coarse aggregate will be filled by the fine aggregate and in turn the pore space of the fine aggregate will be filled by cement paste forming a strong matrix concrete. This also reduces the void space and lowers paste requirement [7].

Table 1
Properties of POC.

Properties	Fine	Coarse
Aggregate size (mm)	<5	5–14
Bulk density (kg/m^3)	1118.86	781.08
Specific gravity (SSD)	2.01	1.82
Moisture content	0.11	0.07
Water absorption (24 h)	26.45	4.35
Fineness modulus	3.31	6.75
Los Angeles abrasion value (%)	–	27.09
Aggregate impact value (AIV) (%)	–	25.36
Aggregate crushing value (ACV) (%)	–	18.08

Table 2
Factor setting using central composite design.

Factor	Axial point	Minimum	Centre	Maximum	Axial point
x_1	0.38	0.40	0.44	0.46	0.50
x_2	460.00	480.00	500.00	520.00	540.00

Aggregates having dry unit weights of less than 1200 kg/m^3 are classified as lightweight aggregate [8]. Due to the porous nature of POC aggregate, low bulk density and high water absorption were expected. From Table 1 it can be seen that POC fine and coarse aggregate has a unit weight of 1119 kg/m^3 and 781 kg/m^3, respectively. This is approximately 25% lighter than the conventional river fine sand [5] and 48% lighter than the crashed granite stone [9]. Consequently, the resulting concrete would be lightweight concrete. This reduces the overall dead load of a structure, with a subsequent significant amount of savings in the total construction cost.

In general, most lightweight aggregate have higher water absorption values compared to conventional aggregate. Although POC has a high water absorption, even though higher water absorption were reported for pumice aggregate which have a value of about 37% [10]. However the high water absorption of POC aggregate can be beneficial to the resulting hardened concrete. It has been reported that lightweight concretes with porous aggregate (high water absorption) are less sensitive to poor curing as compared to normal weight concrete especially in the early ages due to the internal water supply stored in the porous lightweight aggregate [11].

From Table 1, it can be seen that the aggregate impact value (AIV) and aggregate crushing value (ACV) of POC aggregates were higher compared to the conventional crushed stone aggregates [5]. More specifically the AIV and ACV were approximately 34% and 30%, respectively, higher than the granite aggregate. The higher ACV value for the POC aggregate might be caused by the particle shape of POC used in this study which is porous and angular. The aggregate with such shape and condition have the possibility to be crushed when load is applied on them.

3. Experimental program

3.1. Material

The materials used in this work are water, Portland cement and POC aggregate. The water used was a potable drinking water from tap which is suitable for concrete work [12]. Commercial cement meeting the requirements of the ASTM:C150 for Type I Portland cement was employed in this study. The aggregate used was POC which has been obtained from locally palm oil manufacturer in Malaysia. The clinkers were crushed and separated into desired size; fine aggregate (particles less than 5 mm) and coarse aggregate (particle between 5 and 14 mm).

3.2. Mix proportions

The mix proportioning has been carried out in accordance with the requirements of ACI Committee 211.2-98 [13]. The POCC mix design was carried out using equations which have been developed in previous research work [14]. These equations are capable of giving the material constituents of POCC for the first trail batch from given performance criteria [14]. Polynominal models made up of three parameters have been used to describe the effect of mix ingredients on slump, air density and strength of POCC [15]. However, the adjustment of mix proportions of POCC was carried out using a Matlab program for diagnosis and adjustment of mix proportions of structural lightweight concrete [16,17]. A central composite design (CCD) was employed for the factor setting after an extensive trail mixes were done in the laboratory. The factors used in this work are water–cement ratio (x_1) and cement content (x_2). Each factor has fixed the limit in accordance to the requirements of ACI Committee 211.2-98 [13] which are water–cement ratio and cement content have range 0.40–0.46 and 480–520 kg/m^3, respectively. Five mixtures proportions were chosen to establish the POCC properties. These mixtures were labelled as A1, A2, A3, A4 and A5. The factors settings are shown in Table 2.

The aggregate was considered in dry condition since POC easily loses moisture to the outside environment. The natural moisture content of POC was almost zero since the aggregates were kept for some days after preparation. Due to the high water absorption of POC, the aggregates were pre-soaked for 24 h in water before mixing. This is expected to prevent further absorption during mixing. The saturated surface dry (SSD) state of POC was achieved. Two stages mixing approach was employed to allow the cement paste to coat the aggregate permitting the absorbed water to be retained and preventing any water absorption or penetration of cement paste into the aggregate. For each of the mix proportions the air dry density was measured in accordance to the requirement of ASTM:C567. Nine cylinders (150 $\times$ 300 mm) and three prisms (100 $\times$ 100 $\times$ 500 mm) of concrete specimens, for each mixture, were cast and cured under water in accordance to the requirement of ASTM:C192. At age of 28 days, the specimens were tested for compressive strength, splitting tensile strength, flexural strength and modulus of elasticity in accordance to the requirement of ASTM:C39, ASTM:C496, ASTM:C293 and ASTM:C469, respectively. The hardened properties of POCC of the 5 mixtures are presented in Table 3.

The test results for compressive strength ranges from 25.5 to 42.56 N/mm^2. It is approximately 60% higher than the minimum required strength of 17 N/mm^2 for structural lightweight concrete recommended by ASTM:C330. Lightweight concrete normally has density less than 2000 kg/m^3 and the air dry density for POCC ranges from 1818.24 to 1845.62 kg/m^3, which is less than 2000 kg/m^3 and approximately 16% lighter than normal concrete (2200 kg/m^3) [9]. The test results show that the modulus of elasticity of POCC ranges from 9.73 to 26.94 GPa. The splitting tensile strength and modulus of rupture result ranges from 1.85 to

Table 3
Properties of POCC for five chosen mix proportions.

Mixture	Air dry density (kg/m^3)	Compressive strength (MPa)	Splitting tensile strength (MPa)	Modulus of rupture (MPa)	Elastic modulus (GPa)
A1	1845.62	42.56	2.72	4.64	26.94
A2	1835.79	32.08	2.51	4.38	19.35
A3	1832.95	27.15	2.26	4.01	16.87
A4	1820.53	26.52	1.90	3.64	12.61
A5	1818.24	25.50	1.85	3.46	9.73

Table 4
Properties of POCC.

Concrete properties	Average of three sample
Air dry density (kg/m^3)	1832.95
Compressive strength, 28 days (MPa)	27.15
Splitting tensile strength, 28 days (MPa)	2.26
Modulus of rupture, 28 days (MPa)	4.01
Elastic modulus, 28 days (GPa)	16.87

2.72 N/mm^2 and from 3.46 to 4.64 N/mm^2, respectively. The results show that splitting tensile strength and modulus of rupture increased when the cement content increased, and decreased when the water to cement ratio increased. The final mix proportion was chosen by refer to the result shown in Table 3. The centre mix proportion (A3) with compressive strength ranging from 25 to 30 N/mm^2 was selected to be used in the investigation of the flexural behaviour of reinforced POCC beams. The acceptable mixture contained 500 kg/m^3 of cement, 473 kg/m^3 of fine POC aggregate, 155 kg/m^3 of coarse POC aggregate and with a free water–cement ratio of 0.44. For lightweight concrete, the amount of cement content is in range of 285–510 kg/m^3 [18]. The properties of POCC used in cast the reinforced POCC beams are presented in Table 4.

3.3. Reinforced POCC beams

The beams were designed as under-reinforced beams according to the requirements of BS8110 [19]. Eight concrete beam specimens were designed and fabricated before being tested. Four beams were singly reinforced (labelled with S) and the other four beams were doubly reinforced (labelled with D).

All beams had rectangular cross-sectional area of 150×300 mm, with a total length of 2400 mm and an effective span of 2100 mm. The beams size and length were chosen to produce typical flexural mode of failure. The shear-span to depth ratio of 2.33 and the clear concrete cover of 30 mm were kept constant. The beam dimensions were also sufficiently large to simulate a real structural element. The beams details are shown in Table 5 and Fig. 3. The yield strength, (f_y) for the tension steel bars were 590, 575, 553 and 597 N/mm^2 for Y10, Y12, Y16 and Y20, respectively. The links for the shear reinforcement were used only in the shear span to ensure that the beams would fail in flexure.

3.4. Instrumentations and testing

The strains in both reinforcement and concrete were measured through using KYOWA strain gauges. The strain gauge model KFG-10-120-C1-11 has been used for reinforcing steel and the strain gauge model KFG-300120-C1-11 has been used for concrete. All the strains values were recorded using computerise data logger. A small part of the tension bars at the mid-span was ground smooth to facilitate the fixing of the strain gauges and then silicone gel was used to protect the strain gauges from accidental damage during pouring of concrete. The test set up and the instrumentation are shown in Fig. 3. The concrete beams were simply supported and tested under two-point loading. The load from the hydraulic jack was transferred to the beam by means of a spreader beam and the applied load was measured by using a load cell which connected to data logger. All the beams were loaded under two-point loads that were kept at 700 mm apart on a clear span of 2100 mm. Three linear voltage displacement transducers (LVDTs) were placed, one at centre of the beam, the other two under the loading points to measure the beam deflections. During testing, the beams were preloaded with a minimal force of 0.5 kN to allow initiation of the LVDTs and strain gauges. Then the load was applied incrementally with 5 kN for each increment and the load was kept constant for 5 min after each increment to allow the load distribute equally and stably to the beam. The crack widths at the level of tensile reinforcement were measured using hand held microscope with sensitivity of 0.02 mm. All strain, crack width and deflection measurements were measured at every load increment. The first crack load was recorded immediately after the formation and all the cracks were marked as and when they propagated in the beam.

4. Results and discussions

4.1. Failure mode

The failure modes of all the tested POCC beams were typical flexural mode, whereas the flexural cracks initiated first in constant moment zone. Fig. 4 shows the crack pattern and failure of the tested beam D2. The yielding of steel reinforcement took place first and this was followed by crushing of concrete in the compression zone. Since all the beams were designed as under-reinforced, the failure started by yielding of the tension steel bar before the compression failure of concrete as expected. More specifically, the failure of the beams was classified as a ductile failure because the strain in the tension reinforcing steel bar has reached the yielding value ($\varepsilon_y = 0.00287$) before the concrete crushing at the constant moment zone occurred. New flexural cracks formed in the shear spans and curved diagonally due to the effect of flexure and shear combination as the load increased. No bond or shear failure occurred during the test because the anchorage and shear link provided were adequate.

4.2. Bending moment

Table 6 shows a comparison between the experimental ultimate moment (M_{ult}) and the theoretical design moment. The theoretical design moment (M_{des}) of the beam was predicted by using the rectangular stress block analysis recommended by BS 8110 [19]. For beams with reinforcement ratios of 2.23% or less the ultimate mo-

Table 5
Beam details.

Beam ref.	Beam type	Steel bar reinforcement		Beam size $b \times d$ (mm)	Tensile steel area (mm^2)	$\rho = 100As/bd$ (%)
		Tension	Nominal/compression			
S1	Singly	2Y10	2R6	150×302	157	0.349
S2	Singly	2Y12		150×303	226	0.503
S3	Singly	3Y10		150×302	236	0.524
S4	Singly	3Y12		150×305	339	0.754
D1	Doubly	2Y16 + 2Y10	2Y10	150×301	559	1.243
D2	Doubly	2Y20	2Y12	150×298	628	1.396
D3	Doubly	2Y20 + 1Y12	3Y12	150×302	741	1.648
D4	Doubly	3Y16 + 2Y12	2Y16	150×300	1005	2.234

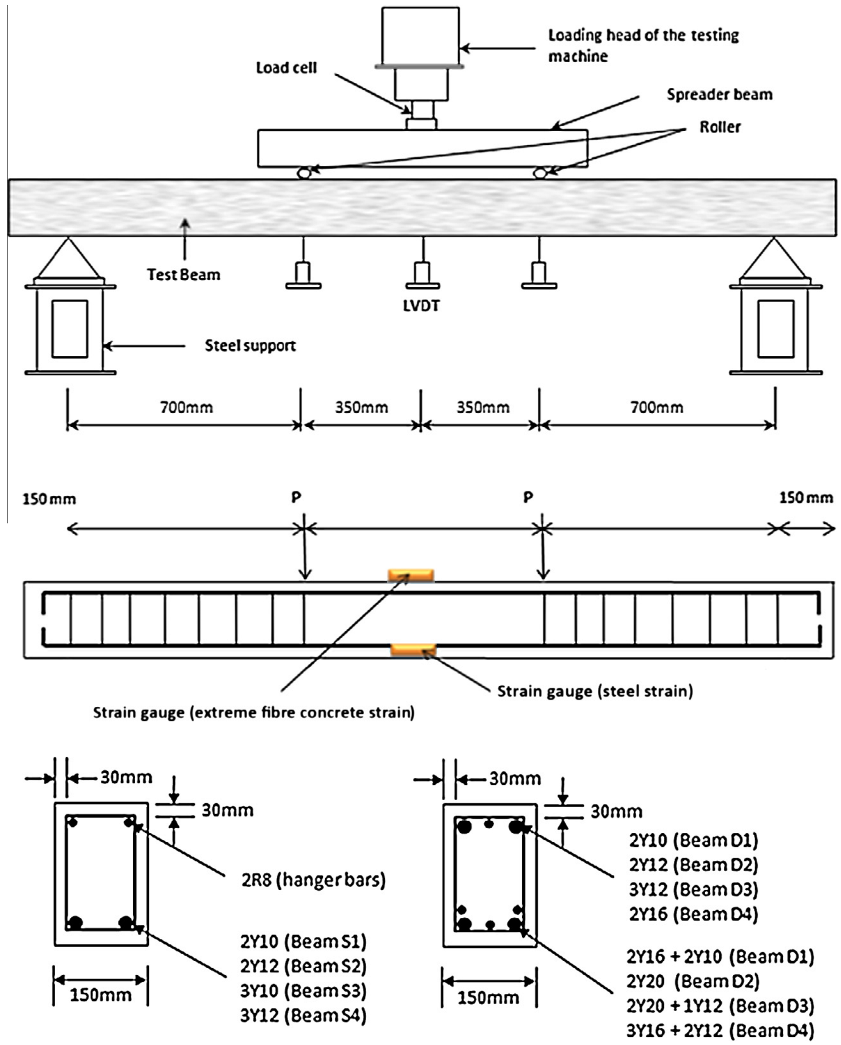

Fig. 3. POCC beams experimental set-up and details.

ment obtained experimentally was approximately 1–7% higher than the predicted value. From the results, it was observed that the BS 8110 based design equations can be used for the prediction of flexural capacity of POCC beams and also provide adequate factor against failure for reinforcement ratios up to 2.23% [19].

4.3. Deflection behaviour

Fig. 5 shows the deflections at mid-span for all the tested beams. It can be seen that at early loads, the slope of the load–deflection curve was steep and linear. Once the flexural cracking occurred, the slope of load–deflection curve changed to fairly linear until yielding of steel reinforcement took place.

A comparison between the predicted mid-span deflections under service moment with the experimental value is shown in Table 7. The predicted deflection is calculated from the beam curvatures according to the requirements of BS 8110 [19]. It was observed that the experimental values are lower than the predicted deflection values recommended by the BS 8110 [19]. More specifically, the experimental values are approximately 10–45% lower than the predicted values. From the POCC properties in Table 4, it can be seen that POCC has low value of modulus of elasticity. Although

(a) Typical flexural failure of beam **(b) Cracks at constant moment**

Fig. 4. Crack patterns and failure mode of POCC beam D2.

Table 6
Comparison between experimental and theoretical ultimate moment.

Beam ref.	Neutral axis at service moment, M_s (mm) [1]	Experimental design moment, M_{ult} (kNm) [2]	Theoretical design moment, M_{des} (kNm) [3]	Capacity ratio of POCC beams [4] = [2]/[3]
S1	44.04	28.56	26.82	1.07
S2	55.71	35.32	33.35	1.06
S3	66.04	40.39	38.76	1.04
S4	83.63	48.32	47.65	1.02
D1	97.61	70.40	69.58	1.01
D2	123.28	80.10	79.63	1.01
D3	125.63	84.25	83.23	1.01
D4	131.42	93.03	92.21	1.01

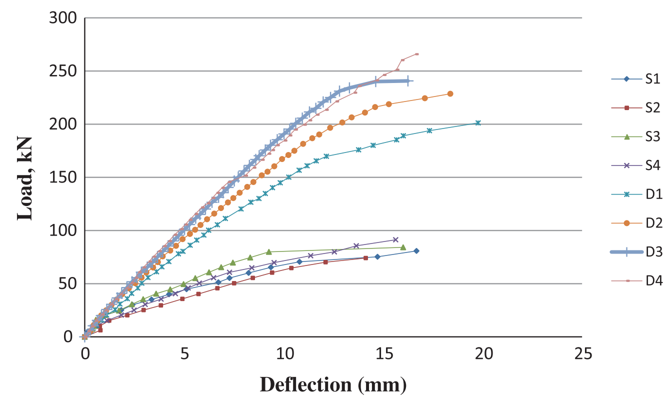

Fig. 5. Experimental load–deflection curves for singly and doubly reinforced POCC beams.

Table 7
Deflection of reinforced POCC beams at service moment.

Beam ref.	Theoretical service moment (kN m)	Experimental deflection Δ_{exp} (mm)	Theoretical deflection Δ_{theo}	$\Delta_{exp}/\Delta_{theo}$	Span/ Δ_{exp}
S1	16.81	8.165	9.755	0.84	257.20
S2	20.63	8.394	11.967	0.70	250.18
S3	24.19	8.239	14.034	0.59	254.89
S4	29.38	13.652	15.194	0.90	153.82
D1	43.13	18.104	22.121	0.82	116.00
D2	49.38	17.169	25.385	0.68	122.31
D3	57.50	16.439	29.739	0.55	127.74
D4	61.88	20.419	35.901	0.57	102.85

Table 8
Displacement ductility of POCC beams.

Beam ref.	Yield stage		Ultimate stage		Displacement ductility ratio, Δ_u/Δ_y
	Load	Deflection, Δ_y (mm)	Load	Deflection, Δ_u (mm)	
S1	52.36	6.87	81.60	18.638	2.71
S2	73.70	13.92	100.90	24.228	1.74
S3	78.44	9.00	115.40	22.757	2.53
S4	83.30	12.07	138.06	29.966	2.48
D1	169.13	12.07	201.20	19.778	1.64
D2	184.52	11.24	228.60	18.378	1.64
D3	210.74	11.84	240.70	16.248	1.37
D4	234.29	13.35	265.8	16.641	1.25

POCC has low modulus of elasticity, the deflection under the design service loads for the singly reinforced beams is acceptable as the span-deflection ratios ranged from 250 to 257 and it is within the allowable limit provided by BS 8110 [19]. BS 8110 recommends an upper limit of span/250 for the deflection in order to satisfy the appearance and safety criteria of a structure [19]. The span-deflection ratio for the other tested beams ranged from 102 to 153. Hence, for higher reinforcement ratios, it is recommended that larger beam depths should be employed.

4.4. Ductility behaviour

Table 8 shows the ductility of tested reinforced POCC beams. The displacement ductility ratio is taken in term of $\mu = \Delta_u/\Delta_y$, which is the ratio of ultimate to first yield deflection, where Δ_u is the deflection at ultimate moment and Δ_y is the deflection when steel yields. It can be seen that beam S1 displayed the most ductile behaviour, with a ductility index of 2.71. From this investigation, it was observed that the reinforced POCC beams exhibit less ductile behaviour with increasing of the reinforcement ratio. This is in agreement with the study of other researchers [20].

4.5. Cracking behaviour

The comparison between predicted crack width value according to ACI 318 [21] and BS 8110 [19] under service loads with experimental crack width value is shown in Table 9. It was observed that the crack width value obtained from experiment at the ultimate load compares reasonably well to the predicted crack width value recommended by ACI 318 and BS 8110. However, ACI 318 gives better accuracy predicted crack width value compare to BS 8110. (See Table 10).

ACI 318 and BS8110 permits crack widths up to 0.41 mm and 0.3 mm, respectively. It was observed that for POCC, the crack widths at service load were below the maximum allowable value.

4.6. Concrete and steel strains

The strain distribution for steel and concrete are shown in Fig. 6. The concrete and steel strains were measured at every 5 kN increment. The result shows that at the early loads, the slope of the

Table 9
Cracking characteristics of POCC beams.

Beam ref.	Experimental cracking moment $M_{cr\cdot(exp)}$ (kN m)	Theoretical cracking moment $M_{cr\cdot(theo)}$ (kN m)	Theoretical design service moment M_s (kN m)	Experimental crack width at failure (mm)	Average crack spacing (mm)	No. of cracks between loading points
S1	8.82	15.34	16.81	1.4	166.4	5
S2	7.105	16.09	20.63	0.55	161.2	4
S3	7.175	16.78	24.19	4.24	154.3	4
S4	8.785	18.15	29.38	3.2	136.6	5
D1	8.96	20.35	43.13	1.26	113.2	7
D2	14.04	20.35	49.38	2.8	133.4	6
D3	10.71	20.35	57.5	1.28	130.3	6
D4	10.68	20.56	61.88	1.46	120.4	5

Table 10
Comparison between predicted and experimental crack widths at service loads.

Beam ref.	[2] Experimental crack width (mm)	[3] Theoretical crack widths, BS8110 (mm)	[4] Theoretical crack widths, ACI (mm)	[2]/[3]	[2]/[4]
S1	0.28	0.26	0.28	1.08	1
S2	0.29	0.24	0.26	1.21	1.12
S3	0.27	0.27	0.29	1.00	0.93
S4	0.3	0.24	0.28	1.25	1.07
D1	0.24	0.23	0.25	1.04	0.96
D2	0.28	0.24	0.27	1.17	1.04
D3	0.24	0.28	0.28	0.86	0.86
D4	0.26	0.3	0.29	0.87	0.90

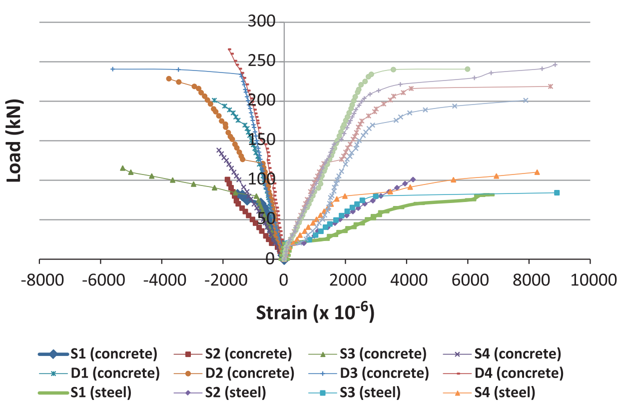

Fig. 6. Steel and concrete strains in POCC beams.

load–strain curve was steep and linear. The measured concrete and steel strain at failure varied from 1525 to 5603 $\times$ 10^{-6} and 4210 to 8911 $\times$ 10^{-6}, respectively. The strain readings were recorded approximately up to 95% of the failure load and therefore the actual strain values are much higher [5]. These results show that the POCC under flexural loading can achieve its full strain capacity.

5. Conclusions

The result from this study shows that the flexural behaviour of reinforced palm oil clinker concrete (POCC) beam is comparable to other types of lightweight concrete and confirms that palm oil clinker (POC) can be used as aggregate in the production of structural lightweight concrete. All the tested under-reinforced POCC beams showed typical structural ductile behaviour. This indicates that the POCC beam can provide ample warning to the imminence of failure. However, the ductility ratio of reinforced POCC beams decreases as the steel reinforcement ratio increasing. It also has been concluded that the BS8110 based design equations can be used for the prediction of flexural capacity of OPCC beams with reinforcement ratio up to 2.23%. In addition, the deflection under the design service loads for singly reinforced POCC beams with reinforcement ratio less than 0.5% were within the allowable limit provided by BS8110. Therefore, it has been suggested that for beams with higher reinforcement ratio, beam depths should be increased. Also, it was found that the crack width for POCC beams at service loads ranged from 0.24 mm to 0.3 mm and this was within the maximum allowable value as stipulated by BS8110 for durability requirement.

References

[1] Mannan MA, Ganapathy C. Concrete from an agricultural waste-oil palm shell (OPS). Build Environ 2004;39(4):441–8.
[2] Mannan MA, Ganapathy C. Engineering properties of concrete with oil palm shell as coarse aggregate. Constr Build Mater 2002;16(1):29–34.
[3] Mohammed BS, Foo WL, Hossain KMA, Abdullahi M. Shear strength of palm oil clinker concrete beams. Mater Des J 2013;46:270–6.
[4] Mohammed BS, Al-Ganad MA, Abdullahi M. Analytical and experimental studies on composite slabs utilising palm oil clinker concrete. Constr Build Mater 2011;25:3550–60.
[5] Delsye CL, Manna MA, Kurian JV. Flexural behaviour of reinforced lightweight concrete beams made with oil palm shell (OPS). J Adv Concr Technol 2006;4(3):1–10.
[6] Qiao XC, Ng BR, Tyrer M, Poon CS, Cheeseman CR. Production of lightweight concrete using incinerator bottom ash. Constr Build Mater 2008;22(4):473–80.
[7] Mindess S, Young JF, Darwin D. Concrete. 2nd ed. (NJ) USA: Pearson Education, Inc.; 2003.
[8] Owens PL. In: Clarke JL, editor. Lightweight aggregates for structural concrete. Structural lightweight aggregate concrete. London: Blackie Academic & Professional; 1993.
[9] Teo DCL, Mannan MA, Kurian VJ. Structural concrete using oil palm shell (OPS) as lightweight aggregate. Turk J Eng Environ Sci 2006;30:1–7.
[10] Hossain KMA. Properties of volcanic pumice based cement and lightweight concrete. Cem Concr Res 2004;34(2):283–91.
[11] Al-Khaiat H, Haque MN. Effect of initial curing in early strength and physical properties of a lightweight concrete. Cem Concr Res 1998;28(6):859–66.
[12] BS 3148:1980. Test for water for making concrete. British Standards Institution, Her Majesty Stationery Office, London.
[13] ACI Committee 211.2-98. Standard practice for selecting proportion for structural lightweight concrete. Detroit: American Concrete Institute.
[14] Abdullahi M, Al-Mattarneh HMA, Mohammed BS. Equations for mix design of structural lightweight concrete. Eur J Sci Res 2009;31(1):132–41.
[15] Abdullahi M, Al-Mattarneh HMA, Mohammed BS. Statistical modeling of lightweight concrete mixtures. Eur J Sci Res 2009;31(1):124–31.
[16] Abdullahi M, Al-Mattarneh HMA, Mohammed BS, Sadiku S, Mustapha KN, Norhisham S. A script file for mix design of structural lightweight concrete. J Appl Sci Res 2010;6(8):1132–41.
[17] Abdullahi M, Al-Mattarneh HMA, Mohammed BS. A matlab program for diagnosis and adjustment of mix proportions of structural lightweight concrete. Eur J Sci Res 2009;31(1):106–23.
[18] Mannan MA, Neglo K. Mix design for oil-palm-boiler clinker (OPBC) concrete. J Sci Technol 2010;30(1):111–8.
[19] BS 8110. Structural use of Concrete Part 1, code of practice for design and construction, British Standards Institution, London; 1985.
[20] Lee TK, Pan ADE. Estimating the relationship between tension reinforcement and ductility of reinforced concrete beam sections. Eng Struct 2003;25(8):1057–67.
[21] ACI 318. Building code requirement for reinforced concrete. Detroit: American Concrete Institute; 1995.