

SHEAR RESISTANCE OF OIL PALM SHELL REINFORCED CONCRETE BEAMS CAST WITH SHEAR REINFORCEMENT

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Abstract

Research on the use of Oil Palm Shell (OPS) to replace coarse aggregates in concrete to date has been focusing on the material properties of oil palm shell concrete (OPSC). So far, there has been a very limited amount of work conducted to investigate the shear performance of the OPSC beams indicating similar shear strength to that of normal-weight concrete (NWC) beams. This study investigates the shear failure mechanism of OPSC beams cast with shear reinforcement. The test results recorded from 11 OPSC beams and 5 NWC beams, simply supported with two-point loads, are presented. The variables considered include the concrete compressive strength, flexural reinforcement, shear-span, and shear-link spacing. The observed shear failure mechanism of OPSC beams was found to be similar to that of an NWC beam while the shear crack formation and shear resistance of OPSC beams are slightly higher (3% to 14%) and (4.2% to 8.5%), respectively. Comparative studies using the current provisions of EC2 and BS8110 have been performed and found that both codes' predictions, show good agreement with only a slight overestimation (VTest/VBS of 0.988) on the shear resistance, were consistent with the tests.

Keywords: Oil Palm Shell Concrete; Reinforced Concrete; Shear Reinforcement; Shear Resistance

Introduction

Oil Palm kernel Shell (OPS) is one of the by-products derived from Palm Oil production and it is mostly left to decay. As part of the efforts to minimize the environmental impacts, the idea of using OPS in concrete has been initiated in the last decade. Since then, several studies (Okpala, 1990; Basri *et al.*, 1999; Mannan *et al.*, 2001; 2002 and 2004; Teo *et al.*, 2006;

Alengaram *et al.*, 2010; Khankhaje *et al.*, 2016; Hamada *et al.*, 2020) have been carried out to study the potential use of OPS as coarse aggregate in normal strength concrete. The use of OPS as coarse aggregate in concrete has resulted in the reduction of concrete density, 1800 kg/m³, and hence, it can be considered as lightweight aggregate concrete.

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Recent research (Basri *et al.*, 1999; Mannan *et al.*, 2001; Alengaram *et al.*, 2010; Serri *et al.*, 2015; Khankhaje, *et al.*, 2016; Swamynadh *et al.*, 2018) have shown promising potential from the material properties of the oil palm shell concrete (OPSC), such as compressive strength, the effect of curing

condition, bond, and durability. It has been observed from tests that by varying the content of OPS the oil palm shell concrete (OPSC) can achieve up to 40 N/mm² in compressive strength.

So far, only a limited amount of research (Teo *et al.*, 2006; Jumaat *et al.*, 2009; Alengaram *et al.*, 2011; Chin *et al.*, 2012; Mo *et al.*, 2017; Hamada *et al.*, 2020) has been carried out to study the shear performance of the OPSC beams. It has been found that the OPSC beams exhibited similar shear resistance to NWC beams, and the inclusion of steel fiber further enhanced the shear resistance of OPSC beams. So far, no study has been reported on the shear behavior of OPSC beams cast with shear reinforcement.

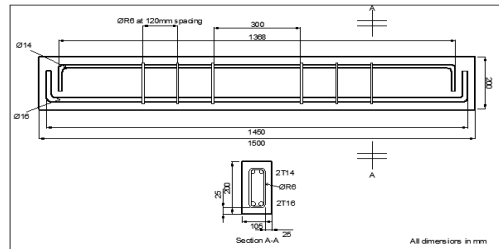
In view of the importance of the shear failure mechanism to the overall structural performance of a reinforced concrete structure, it is believed that more research will be needed to aid the current understanding of the shear performance of OPSC beams cast with shear reinforcement and to review the applicability of the code's design clauses for Normal Weight Concrete (NWC) beams for OPSC beams.

In this paper, the test results of eleven OPSC beams cast with shear reinforcement are presented and compared to the normal-weight concrete (NWC) beams. The variables considered were the concrete compressive strength, spacing of the shear reinforcement, and shear span.

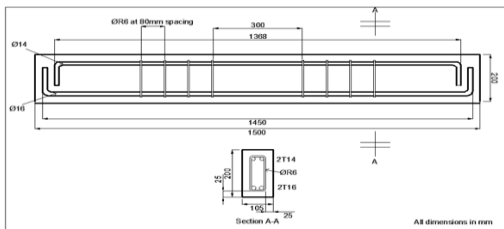
Materials and Methods

Beam Specimen

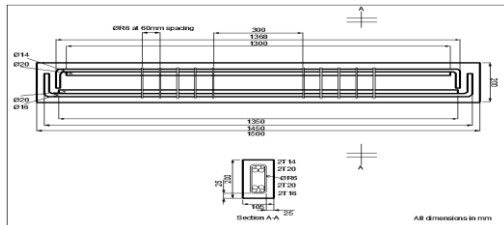
Tests have been performed on sixteen beam specimens cast with shear reinforcement, of which, five were NWC beams and eleven were OPSC beams. All beams have an overall depth of 200 mm, breadth of 105 mm, and length of 1500 mm, as shown in Figure 1. Variables considered in the



(a) Reinforcement details for beam: 3A, 3B, 3C and NWC-A



(b) Reinforcement details for beam: 4A, 4B, 4C, 4D, 4E, NWC-B, NWC-D, and NWC-E



(c) Reinforcement details of beam: 5A, 5B, 5C and NWC-C

Figure 1. Beam specimens and reinforcement details

Table 1. Details of beam specimens

Beam No.	Type of concrete	Beam dimensions, $b \times h \times l$ (mm)	Effective depth, d , (mm)	Shear span, a , (mm)	Shear-link spacing, s , (mm)	Mix design	Cube strength, f_{cu} (N/mm ²)
3A	OPSC	105 × 200 × 1500	161	240	120	A	25.79
3B	OPSC	105 × 200 × 1500	161	240	120	B	31.93
3C	OPSC	105 × 200 × 1500	161	240	120	C	34.60
4A	OPSC	105 × 200 × 1500	161	240	80	A	25.79
4B	OPSC	105 × 200 × 1500	161	240	80	B	31.93
4C	OPSC	105 × 200 × 1500	161	240	80	C	34.60
4D	OPSC	105 × 200 × 1500	161	200	80	D	30.15
4E	OPSC	105 × 200 × 1500	161	160	80	E	30.15
5A	OPSC	105 × 200 × 1500	151	240	60	A	25.79
5B	OPSC	105 × 200 × 1500	151	240	60	B	31.93
5C	OPSC	105 × 200 × 1500	151	240	60	C	34.60
NWC-A	NWC	105 × 200 × 1500	161	240	120	N	30.61
NWC-B	NWC	105 × 200 × 1500	161	240	80	N	30.61
NWC-C	NWC	105 × 200 × 1500	151	240	60	N	30.61
NWC-D	NWC	105 × 200 × 1500	161	160	80	N	30.61
NWC-E	NWC	105 × 200 × 1500	161	240	80	N	35.00

OPSC beams include concrete strength, flexural reinforcement, spacing of shear reinforcement, and shear span (as indicated in Table 1). Shear spans considered were 160mm, 200mm, and 240mm. The spacing of shear reinforcement considered was 60mm, 80mm, and 120mm. Concrete strength ranged from 25.79N/mm² to 34.60N/mm².

Concrete

All NWC specimens have been cast from concrete constituted of Portland Composite Cement, sand, water, and crushed granite as coarse aggregates with grading curves, as shown in Figure 2. All OPSC specimens have been cast from concrete constituted of Portland Composite Cement, sand, water, and OPS as coarse aggregates with grading curves, as shown in Figure 3. All OPS have been de-oiled with detergent to remove any residual oil deposits to allow for quality concrete binding. Prior to sieving the OPS were oven dried at 100°C for 24 hours. Three concrete mixes have been adopted in this research for casting the OPSC beam specimens to achieve variations in concrete compressive strength. One mix has been adopted for NWC beam specimens with the variations in concrete compressive strength obtained from differences in curing durations as indicated in Table 2. All specimens have been water-cured until the day of the test, and the concrete compressive strength described in this paper has been obtained

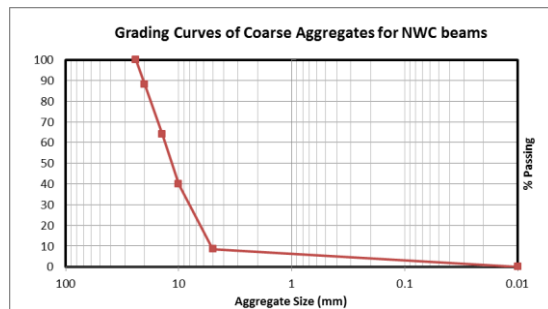
from 100 mm cubes cast simultaneously with the beam specimens.

Reinforcement

All flexural reinforcement has been fabricated from high-yield tension “T” bars with an average yield strength of 460 N/mm² with both ends bent up to prevent premature failure as shown in Figure 1. Shear reinforcement has been fabricated from 6 mm diameter plain steel bars denoted as “R” with a yield strength of 410 N/mm² from the test. In general, 25 mm clear covers to the shear reinforcement have been maintained in all specimens.

Test Procedures

All beam specimens have been tested in an upright manner to stimulate the prototype structures. All specimens have been simply supported at both ends as shown in Figure 4. The applying load was positioned at the centre via spreader beams with a mean of hand-operated hydraulic jack. The load was applied using a hydraulic jack with increments of 4.21 kN until failure. The hydraulic extruder applied force onto the midspan of spreader beams to distribute the loads equally via a pair of steel rollers. These steel rollers were held in place with steel wedges positioned on the concrete beam. The beams were supported by a pair of steel rollers held in positions by steel wedges, which in turn, were supported on a pair of steel stumps.



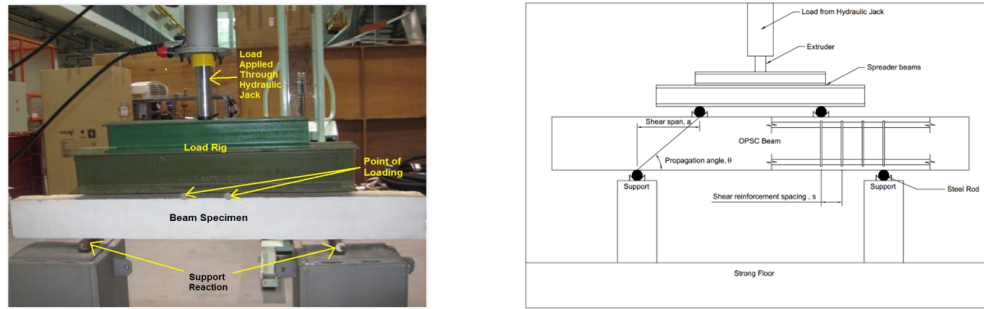


Figure 4. Test setup

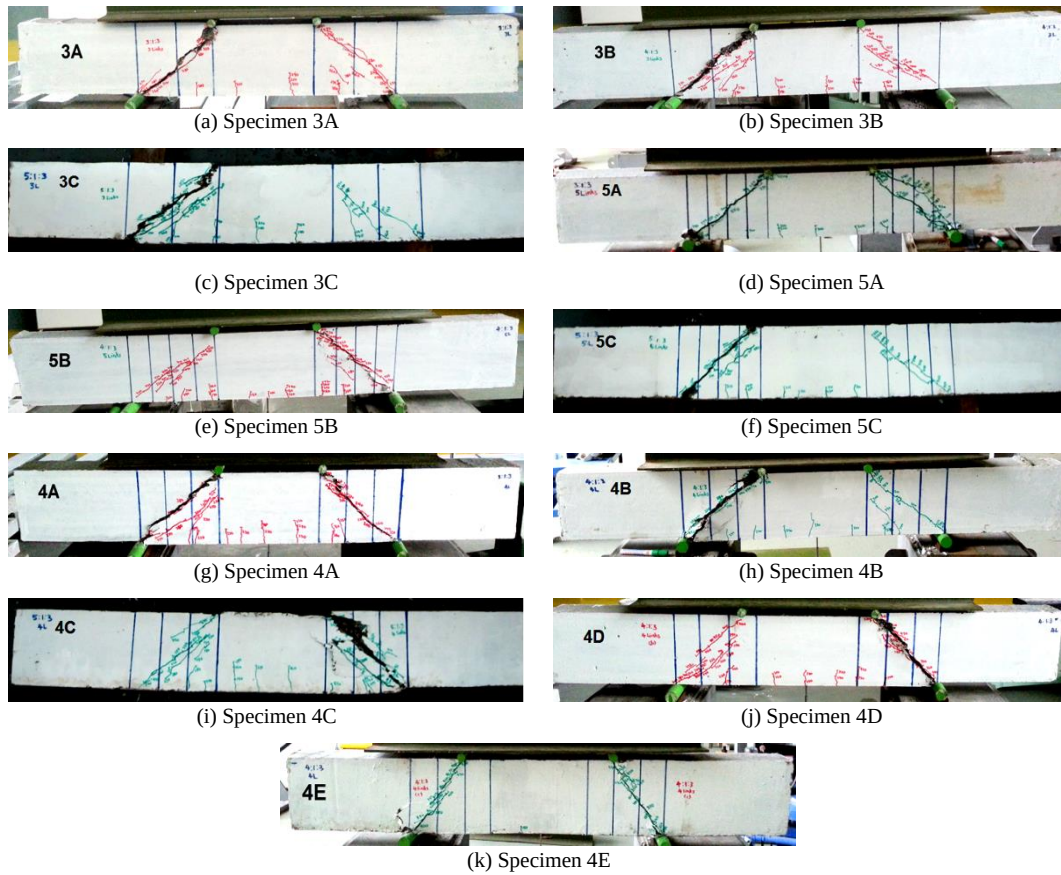


Figure 5. Shear failure of OPSC beams

Results and Discussion

Failure Modes

All sixteen beam specimens presented in this paper failed in the mode of shear failure with no distinct differences between the OPSC beams, as shown in Figure 5, and the NWC beams, as shown in Figure 6. In general, flexural cracks were observed, before the formation of shear cracks during tests, propagating from the tension face towards the neutral axis of the beam. The formation of flexural cracks for OPSC beams was observed to

range from 27% to 37% of the ultimate failure loads, and for NWC beams, from 36% to 46% of the failure loads, as indicated in Table 3. The inferred flexural cracks, however, were observed to occur at 17% to 30% for OPS beams, and 28% to 38% for NWC beams. These discrepancies have been expected as the early formation of micro-cracks has not been able to be detected by the naked eye.

The formation of shear cracks for the OPSC beams was observed to range from 52% to 68% of the ultimate failure loads, and for NWC beams, from 49% to 54% of the ultimate failure loads, as

indicated in Table 3. Further analysis of the test results revealed that specimens exhibited higher shear cracking load with respect to the ultimate failure loads have been loaded at positions nearer to the supports (specimens 4D, 4E, and NWCD, i.e., with lower a/d ratio), while specimens exhibited lower shear cracking load with respect to ultimate failure loads have been loaded at positions further away from the supports (specimens 4A, 4B, 4C, 3A, 3B, 3C, 5A, 5B, 5C, NWCA, NWCB, NWCC, and NWCE - i.e. higher a/d ratio). Such observations

have been expected, as increases in the shear span reduce the angle of inclination, thus resulting in a decrease in the shear resistance that can be mobilized from aggregates interlocking.

It has also been observed that the diagonal shear cracks initiated at near mid-height of the beam specimens and later propagated diagonally towards the load positions and the supports. These phenomena would have been expected due to the presence of the diagonal compressive action between the loads and the supports which resulted

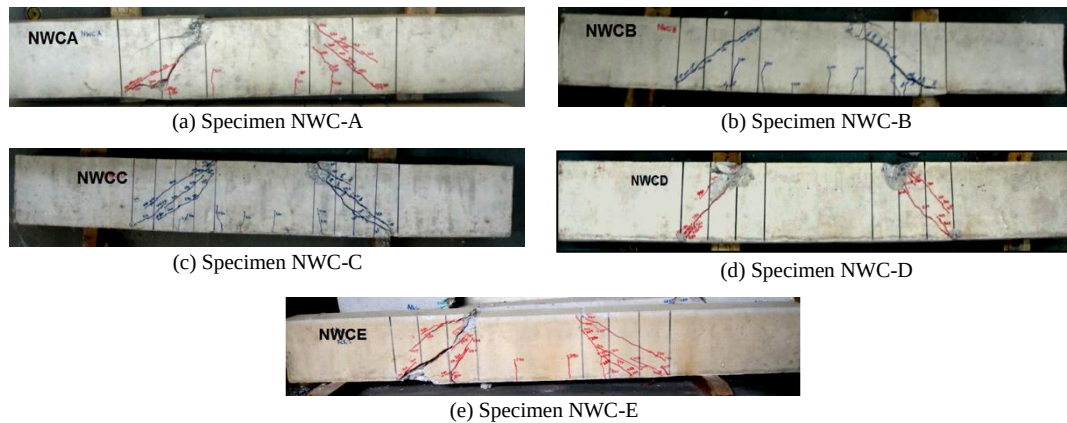


Figure 6. Shear failure of NWC beams

Table 3. Ultimate failure and cracking loads of beam specimens

Specimen No.	V_{test} (kN)	V_{crack} (kN) Inferred	$V_{flexural\ crack}$ (kN) Observed	$V_{shear\ crack}$ (kN) Observed
3A	75.78	12.63	21.05	42.10
		17%	28%	56%
3B	88.41	18.95	27.37	54.73
		21%	31%	62%
3C	92.62	16.84	25.26	48.42
		18%	27%	52%
4A	79.99	24.21	25.26	46.31
		30%	32%	58%
4B	94.73	25.26	29.47	54.73
		27%	31%	58%
4C	107.36	29.47	31.58	56.84
		27%	29%	53%
4D	101.04	27.37	31.58	67.36
		27%	31%	67%
4E	105.25	31.58	37.89	71.57
		30%	37%	68%
5A	88.41	25.26	29.47	56.63
		28%	33%	64%
5B	107.36	27.37	33.68	56.84
		25%	31%	53%
5C	119.99	23.16	37.89	63.15
		20%	32%	53%
NWCA	82.10	23.16	37.89	40.00
		28%	46%	49%
NWCB	88.41	29.47	31.58	46.31
		33%	36%	52%
NWCC	99.99	37.89	44.21	50.52
		38%	44%	51%
NWCD	101.04	35.79	-	54.72
		35%	-	54%

in the indirect tensile stress in the perpendicular direction thus the formation of the diagonal shear cracks. Such failure mechanism has been further confirmed by the concrete crushing failure observed at the load positions as shown in Figures 5 and 6.

Failure Loads

The ultimate shear failure loads from tests, V_{test} , are presented in Table 3. It is important to note that the self-weight of the beam specimens has not been included. In general, it can be observed from the failure loads that the shear resistance of OPSC beams is similar to NWC beams, which increase with the concrete strength and as the shear span and shear-link spacings decrease.

In comparison to specimen 3A, specimens 3B and 3C exhibited increments in shear resistance of 17% and 22% as the OPSC concrete compressive strength increased from 25.79 N/mm² to 31.93 N/mm² and 34.60 N/mm², respectively. Similar observations can be made from comparisons between specimens 4A and 5A with 4B, 4C, and 5B with 5C, respectively. These phenomena are to be expected because the increase in concrete strength increases the indirect tensile strength and hence the shear strength.

Similar to that in NWC beams, the shear resistance of an OPSC beam has been observed to increase as the shear-link spacing decreases. In comparison with specimens 3A, 3B, and 3C, specimens 4A, 4B, 4C, 5A, 5B and 5C, exhibited increments of 6%, 7%, 16%, 17%, 21%, and 30% as the shear-link spacing decreases from 120 mm to 80 mm and 60 mm, respectively. These phenomena are to be expected because a decrease in shear-link spacing increases the density of shear-link on the shear failure surface, thus increasing the number of shear-links to be mobilized in tension to carry the applied shear force. In comparison with specimen NWC-A, specimen NWC-B exhibited an increment in shear resistance of 7.7% as the shear-link spacing decreased from 120 mm to 80 mm, which is in agreement to that of the OPSC specimens having similar concrete strength (comparing with specimens 3B, 4B exhibited 7% increase in shear resistance).

Furthermore, the shear resistance of an OPSC beam has been observed to increase as the shear span decreases. In comparison with specimens 4B, specimens 4D and 4E exhibited increments in shear resistance of 6.7% and 11.1% as the shear span decreased from 240 mm to 200 mm and 160 mm, respectively. In comparison with specimen NWC-B, specimen NWC-D exhibited an increment in shear resistance of 14.7% as the shear span decreased from 240 mm to 160 mm. Such increases observed are believed to be attributed to the increase in

aggregate interlocking being mobilized as the shear span reduces. In comparison, a higher increment was observed in NWC as the shear span reduced, which is believed to be attributed to the higher strength found in granite aggregates constituted in the NWC.

However, in comparison with the NWC specimens having shear-span at 240 mm, NWC-A, NWC-B, NWC-E, the OPSC specimens 3B, 4B, and 4C, exhibited 7.7%, 7.2%, and 8.5% higher shear resistance, respectively. Furthermore, in comparison with specimen NWC-D with a shear span of 160 mm, specimen 4E exhibited 4.2% higher shear resistance. Such discrepancy is believed to be attributed to the rougher shear surface observed from the OPSC beam specimen compared to the NWC beam specimen (see Figure 7), where a higher degree of aggregate interlocking has been mobilized.

Midspan Deflection

The midspan deflection of each specimen has been measured using a mechanical dial gauge placed under the specimens at the midspan position. The deflection curves recorded from the tests are presented in Figure 8. In general, it can be observed from the midspan deflections that OPSC beams undergo longer plastic deformation than the NWC beams.

In comparison to specimen 3B, specimens 4B and 5B as shown in Figures 8(b) and 8(c) exhibited increments of 9% and 16% in the ultimate midspan



(a) Cross-section view of failure surface



(b) Isometric view of failure surface

Figure 7. Shear failure surface of specimens

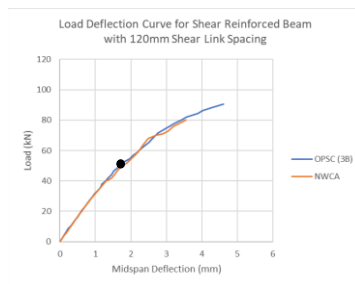
deflection from 4.6 mm to 5.0 mm and 5.8 mm as the spacing of shear reinforcement decreases from 120 mm to 80 mm and 60 mm, respectively. This phenomenon is to be expected because lowering the shear reinforcement spacing increases the shear resistance, which, in turn, increases the midspan bending moment, thus increasing the midspan deflection. Specimen 5B exhibits a longer plastic deformation in comparison to specimen 4B and, subsequently, specimen 3B as the shear span decreases. This is because as the shear resistance increases, the resulting midspan bending moment increases, which, in turn, increases the midspan deflection.

In comparison to specimen 4E, specimen 4B, as shown in Figure 8(e), exhibited an increment of 28% in ultimate midspan deflection from 3.9 mm to 5.0 mm as the shear span increased from 160 mm to 240 mm. This demonstrates that the increasing effect of shear angle by 29% from 39.8 degrees to

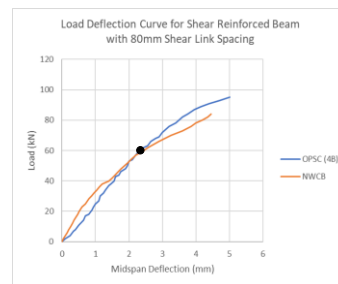
51.3 degrees outweighs the decreasing effect of a reduced number of shear reinforcements crossing the shear span.

In comparison to specimen 4B, specimen 4C, as shown in Figure 8(e), exhibited an increment of 8% in ultimate midspan deflection from 5.0 mm to 5.4 mm as the concrete strength increased by 17% from 30 N/mm² to 35 N/mm². This phenomenon is to be expected because concrete strength increases the load-carrying capacity of a beam and hence results in higher midspan deflection.

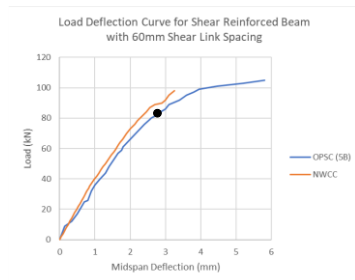
In comparison to specimen 4E, specimen 4C as shown in Figure 8(e) exhibited an increment of 38% in ultimate midspan deflection from 3.9 mm to 5.4 mm as the shear span increases from 160 mm to 240 mm and concrete strength from 30 N/mm² to 35 N/mm². With similar failure loads, this phenomenon shows that the increasing effect of a 17% increase in concrete strength outperforms the decreasing effect of a 50% increase in shear span.



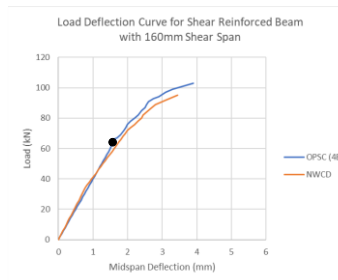
(a) 120 mm Shear-link Spacing, 240 mm Shear Span



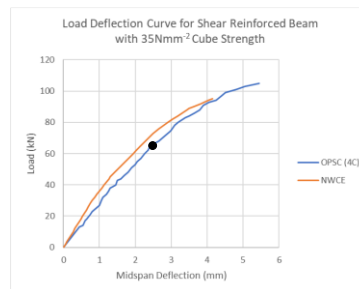
(b) 80 mm Shear-link Spacing, 240 mm Shear Span



(c) 60 mm Shear Spacing, 240 mm Shear Span



(d) 80 mm Shear-link Spacing, 160 mm Shear Span



(e) 80 mm Shear-link Spacing, 240 mm Shear Span

Figure 8. Load-deflection curves for OPSC and NWC beams

Stiffness

The stiffness of a beam can be obtained as the gradient of the load vs deflection curve before the point of plastic deformation (marked with black dot(s) in Figure 8). For similarly reinforced sets of OPSC and NWC beams, the percentage difference in stiffness ranges from 0% to 18%. Such observation is to be expected as it was reported by (Yew *et al.*, 2014) that heat treatment increases the Modulus of Elasticity of OPS up to 44% and into the range of normal-weight concrete.

Compared to specimen 4B, specimen 4E, as shown in Figure 8(d), exhibited an increment of 54% in stiffness from 26 kN/mm to 40 kN/mm as the shear span decreased by 50% (from 240mm to 160 mm). This demonstrates that beams loaded with shorter shear spans exhibited stiffer behavior and increases in shear resistance.

Comparisons with Codes' Prediction

Comparisons with test results have been carried out to evaluate the applicability of the current clauses for shear resistance from BS 8110-1:1997 (BSI, 1997) and BS EN 1992-1-1:2004: Eurocode 2 (BSI, 2004) on OPSC and NWC beams in Table 4 and Table 5, respectively. These predictions have been calculated with the partial safety factors set to unity.

BS EN 1992-1-1:2004: Eurocode 2 (BSI, 2004) (equation 6.8) provides conservative

predictions towards the observed failure loads with a mean of 2.106 and standard deviation of 0.446 for OPSC specimens and a mean of 2.031 and 0.384 for NWC.

Comparisons based on BS EN 1992-1-1:2004: Eurocode 2 (BSI, 2004) (equation 6.9) instead of the smaller values from equations (6.8 or 6.9) as required by clause 6.2.3(3), EC2 show good agreements with a slight overestimation towards the observed failure loads with a mean of 0.988 and standard deviation of 0.110 for OPSC and a mean of 0.937 and 0.100 for NWC. Further analysis into the comparisons revealed Eurocode 2 underestimates the shear resistance for specimens either cast with additional reinforcement (5A, 5B, 5C, and NWCC by 10.9%, 11.2%, 15.8%, and 7.5%) or loaded with reduced shear span (4D, 4E, and NWCD by 1.6%, 5.8%, and 0.2%).

BS8110-1:1997 (BSI, 1997) shows good agreements with a slight overestimation towards the observed failure loads with a mean of 0.988 and a standard deviation of 0.122 for OPSC and a mean of 0.951 and a standard deviation of 0.098 for NWC. Further analysis into the comparisons revealed BS8110-1:1997 (BSI, 1997) underestimates the shear resistance for specimens (3B, 3C, 4B, 4C, 4D, and NWCE by 13%, 16.5%, 1%, and 12.9%) having concrete strength, f_{cu} , more than 30 N/mm² and specimens with shear-link spacing of 120 mm (3A, 3B, 3C, and NWCA by 1%, 13%, 16.5%, and 5.8%).

Table 4. Comparisons of OPSC with code prediction

Specimen No.	V_{test} (kN)	$V_{EC2-(Eqn\ 6.8)}$ (kN)	$V_{EC2-(Eqn\ 6.9)}$ (kN)	V_{BS8110} (kN)	V_{test}	V_{test}	V_{test}
					$V_{EC2-(Eqn6.8)}$	$V_{EC2-(Eqn6.9)}$	V_{BS8110}
3A	75.78	33.595	84.983	74.999	2.256	0.892	1.010
3B	88.41	33.595	103.077	78.238	2.632	0.858	1.130
3C	92.62	33.595	110.511	79.516	2.757	0.838	1.165
4A	79.99	50.392	84.983	90.552	1.587	0.941	0.883
4B	94.73	50.392	102.963	93.791	1.880	0.920	1.010
4C	107.36	50.392	110.511	95.070	2.130	0.971	1.129
4D	101.04	41.994	99.470	102.146	2.406	1.016	0.989
4E	105.25	41.994	99.470	116.018	2.506	1.058	0.907
5A	88.41	63.017	79.705	115.898	1.403	1.109	0.763
5B	107.36	63.017	96.568	120.145	1.704	1.112	0.894
5C	119.99	63.017	103.647	121.821	1.904	1.158	0.985
Mean					2.106	0.988	0.988
Standard Deviation					0.446	0.110	0.122

Table 5. Comparisons of NWC with codes prediction

Specimen No.	V_{test} (kN)	$V_{EC2-(Eqn\ 6.8)}$ (kN)	$V_{EC2-(Eqn\ 6.9)}$ (kN)	V_{BS8110} (kN)	V_{test}	V_{test}	V_{test}
					$V_{EC2-(Eqn6.8)}$	$V_{EC2-(Eqn6.9)}$	V_{BS8110}
NWC-A	82.10	33.595	99.171	77.579	2.444	0.828	1.058
NWC-B	88.41	50.392	99.171	93.132	1.754	0.891	0.949
NWC-C	99.99	63.017	93.011	119.281	1.587	1.075	0.838
NWC-D	101.04	41.994	100.824	116.369	2.406	1.002	0.868
NWC-E	98.94	50.392	111.628	95.256	1.963	0.886	1.039
Mean					2.031	0.937	0.951
Standard Deviation					0.384	0.100	0.098

Conclusions

Experimental tests have been carried out on eleven OPSC beams to investigate the shear failure mechanism and to review the applicability of the current design methods. It is observed that:

1. The shear failure mechanism of an OPSC beam has been found to be similar in nature to that of an NWC beam. The flexural cracks have been observed, prior to the formation of shear cracks during tests, as that of an NWC beam. The formation of shear cracks has been observed at a slightly higher percentage (3% to 14%) to the ultimate failure loads in comparison to the NWC beams.

2. The OPSC beams exhibited similar behavior to NWC beams with respect to the variables considered: concrete strength, reinforcement ratio, shear span, and shear-link spacings.

3. The OPSC beams exhibited promising shear resistance, which was slightly higher (4.2% to 8.5%) than that of NWC beams.

4. The provisions of the current BS8110-1:1997 (BSI, 1997) for normal-weight concrete show good agreements with slight overestimation of the shear resistance for both the NWC beams (VTest/VBS of 0.951) and the OPSC (VTest/VBS of 0.988) beams reported in this paper.

5. The provisions of the current BS EN 1992-1-1:2004: Eurocode 2 (BSI, 2004) clause 6.2.3(3) for normal-weight concrete provide conservative predictions (smaller of equations 6.8 and 6.9) with respect to the observed failure loads. Equation 6.9 shows good agreement with a slight overestimation of the shear resistance for both the NWC beams (VTest/VBS of 0.937) and the OPSC beams (VTest/VBS of 0.988) reported in this paper.

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