Materials and Design 30 (2009) 1737–1742

Contents lists available at ScienceDirect

Materials and Design

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

Technical Report

Flexural behaviour of one-way concrete slabs reinforced with steel bars milled from scrap metals

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ARTICLE INFO

Article history:
Received 12 June 2008
Accepted 29 July 2008
Available online 5 August 2008

ABSTRACT

Laboratory tests were performed on 12 simply-supported one-way concrete slabs reinforced with steel bars that were milled from scrap metals. The slabs were subjected to concentrated line loads at the third points. Two different failure modes of flexural yielding of the tension bar or flexural crushing of the concrete were predicted. The observed failure modes, however, were either one or a combination of modes of tension failure, concrete crushing, diagonal shear, or shear bond failures. On the average, for one-way slabs with span-to-effective depth ratios varying between 14 and 24.37, and shear span-to-effective depth ratios varying between 4.6 and 8.12, a short-term factor of safety of approximately 1.3 against cracking and 0.94 against collapse were obtained from the experimental results. Based on the analysis of the experimental results, it is proposed that an average steel strength of about 370 N/mm^2 for steel bars milled in Ghana must be used in reinforced concrete design rather than the characteristic strength of 250 N/mm^2 conventionally prescribed by BS8110 for mild steel.

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1. Introduction

Reinforced concrete is a construction material that is most commonly used in developing countries. The ease with which reinforced concrete is used of construction and its generally well-established properties are some of the reasons for its popular choice. There is however the need to ensure that detailed information on the strength and deformational properties of reinforced concrete conform to technical specifications of the required code of practice, particularly when a new constituent material is substituted in it. Some researchers [1–4] have characterized conventional concrete and steel reinforcement characteristics, whilst others [5–8] have also characterized non-ferrous reinforcement bars for use in concrete.

In Ghana, reinforced concrete buildings constitute about 95% of the building stock in the urban centres. Reinforcing steel bars used in Ghana are milled from re-cycled scrap metals. Earlier experimental work was done in characterizing the strength and ductility of the steel bars subjected to direct tension and their use in under-reinforced concrete beams subjected to third-point loading [9].

Twelve reinforced concrete beams were reinforced with different percentages of steel in tension as well as compression. Chemical analysis performed on the steel bars obtained from three different steel millers showed that the percentage limits of elements such as carbon, silicon and phosphorus necessary to ensure ductility were all exceeded. Therefore the bars exhibited high tensile strength but low ductility, and therefore did not possess the characteristics of mild steel with which the bars were supposed to comply. They failed suddenly in brittle failure modes when tested under monotonic and cyclic loads. Others in Turkey [10], have compared steel bars produced from iron ore and scrapped steel in terms of tension and yield strength, elongation of rupture and yield strength. The tensile strength of rebars produced from iron ore is 15% higher than that of rebars produced from scraped steel. It was concluded that rebars produced from scraped steel are unsuitable for use in structures built in disaster prone areas.

Another study in Ghana, indicated that the average anchorage bond strength of the such bars developed at ultimate bond strength were higher than that of standard mild steel bars and of the same order as that of high tensile steel [11].

The shear strength of 18 beams reinforced to different shear capacities with similar steel bars milled from scrap metals were also studied [12]. The beams failed mostly in brittle failure mode either as concrete crushing or diagonal shear. Further research was conducted to study the behaviour of two-way slabs reinforced with bars milled from scrap metals. The slabs were subjected to central point load. Collapse of the slabs was predominantly due to a combination of concrete crushing following extensive flexural cracking and punching shear [13].

This paper presents the results of a study of the flexural strength and deformation characteristics of one-way slabs reinforced with similar steel bars milled from scrap metals and subjected to two line loads applied at the third points.

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0261-3069/$ - see front matter © 2008 Elsevier Ltd. All rights reserved.
doi:10.1016/j.matdes.2008.07.048

2. Details of experimental works

Twelve one-way concrete slabs reinforced with steel milled from scrap metal were prepared in the laboratory. The slabs measured (length × breadth × depth) 1000 mm × 300 mm × 80 mm, 1000 mm × 300 mm × 70 mm and 1350 mm × 375 mm × 65 mm. The slabs were reinforced in both directions with a clear concrete cover of 12 mm. The percentage of main longitudinal bars ranged from 0.54 to 1.41 whilst that of transverse direction ranged from 0.22 to 0.57. The concrete was prepared using ordinary Portland cement, natural river sand, crushed granitic rock of 10 mm maximum size. The mix proportions by weight of cement: sand: coarse aggregate were 1:2:4, and water–cement ratio of 0.5.

The modulus rupture of the concrete was determined by subjecting unreinforced concrete beams measuring 100 mm × 100 mm × 500 mm to flexural loading. Control cubes measuring 100 mm × 100 mm × 100 mm were also tested to obtain the compressive strength of concrete. The properties of the reinforcing steel bars, concrete at 28 days and reinforced concrete slabs are presented in Tables 1 and 2, respectively.

During loading the slabs were simply supported at their ends on steel beams which formed part of a rigid steel frame. A dial gauge was arranged to measure the central deflection of the slabs. Two line loads were applied at the third points of the slab by means of a hydraulic jack (Fig. 1a and b). The test procedure included crack monitoring and central deflection measurements for load increments of 2 kN. Six slabs were tested by monolithic loading whilst the rest were subjected to limited cyclic loading.

Table 1
Physical properties of steel reinforcement

Rod mark	Yield stress (N/mm^2)	Ultimate strength (N/mm^2)	Percentage elongation (%)	Type of failure
1	400	523	12.6	Brittle
2	376	517	12.2	Brittle
3	350	481	10.7	Brittle
4	362	543	9.8	Brittle
5	365	571	10.6	Brittle
6	356	578	11.2	Brittle
Average	368	535.5	11.2	

3. Flexural theory and shear strength

In an unreinforced concrete section, the cracking moment M_{cr}, derived from the modulus of rupture of concrete is given by

$$M_{cr} = f_{tc}bh^2/6, \quad (1)$$

where f_{tc} denotes the modulus of rupture of the concrete; b is the width of the slab and h is the overall thickness of the slab.

The ultimate flexural load of a one-way slab that is loaded equally at the third points is given by

$$P_{ult} = 6M_{ult}b/L, \quad (2)$$

where M_{ult} denotes the ultimate moment of resistance per unit width of the slab and L denotes the span of the slab.

The theoretical shear strength of the slab in accordance with the British Code of Practice BS8110 [14] was estimated by considering:

(1) The concrete section alone.
(2) Both the concrete section and the tension reinforcement with the assumption that the latter behaved in a manner similar to that of steel in resisting shear.

4. Theoretical and experimental results

4.1. Material properties of the slabs

The physical properties of the steel bars, concrete and reinforced concrete slabs are given in Tables 1 and 2. The tensile yield strength of the reinforcing steel bars ranged from 350 to 400 N/mm^2 whilst the ultimate strengths ranged from 481 to 576 N/mm^2. Since the requirement for ductility was inadequate as percent elongation values measured averaged 11.2% (Table 1) as compared to the code minimum of 22% [15], the failure of the bars subjected to direct tension was brittle. The compressive strength of the 100 × 100 × 100 mm concrete cubes ranged from 31.1 to

Table 2
Details of one-way slabs

Slab no.	Length (mm)	Width (mm)	Thickness (mm)	Span/eff. depth ratio	Steel reinforcement: Longitudinal Reinf., Steel area, As (mm^2)	Longitudinal Reinf., Steel area, 100As/bh (%)	Transverse Reinf., Steel area, As (mm^2)	Transverse Reinf., Steel area, 100As/bh (%)	Tensile: Total steel area (%)	Tensile: Strength (N/mm^2)	Concrete: Compressive strength (N/mm^2)	Concrete: Modulus of rupture (N/mm^2)
S1	1000	300	80	14.0	132.59	0.55	176.40	0.22	0.77	368	33.2	5.4
S2	1000	300	80	14.0	132.59	0.55	176.40	0.22	0.77	368	33.2	4.8
S3	1000	300	80	14.5	339.43	1.41	452.45	0.57	1.98	368	34.1	4.3
S4[a]	1000	300	80	14.0	132.59	0.55	176.40	0.22	0.77	368	31.1	5.4
S5[b]	1000	300	80	14.0	132.59	0.55	176.40	0.22	0.77	368	31.1	4.8
S6[c]	1000	300	80	14.5	339.43	1.41	452.45	0.57	1.98	368	34.1	4.3
S7	1000	300	70	16.6	132.59	0.63	176.40	0.25	0.88	368	34.1	5.2
S8	1350	375	65	24.7	132.59	0.54	220.92	0.25	0.79	368	38.6	5.2
S9	1350	375	65	24.7	132.59	0.54	220.92	0.25	0.79	368	38.6	5.0
S10[d]	1000	300	70	16.6	132.59	0.63	176.40	0.25	0.88	368	34.1	5.2
S11[e]	1350	375	65	24.7	132.59	0.54	220.92	0.25	0.79	368	31.1	5.2
S12[f]	1350	375	65	24.7	132.59	0.54	220.92	0.25	0.79	368	31.1	5.0

[a] 8 cycles.
[b] 15 cycles.
[c] 20 cycles.
[d] 20 cycles.
[e] 50 cycles.
[f] 50 cycles.

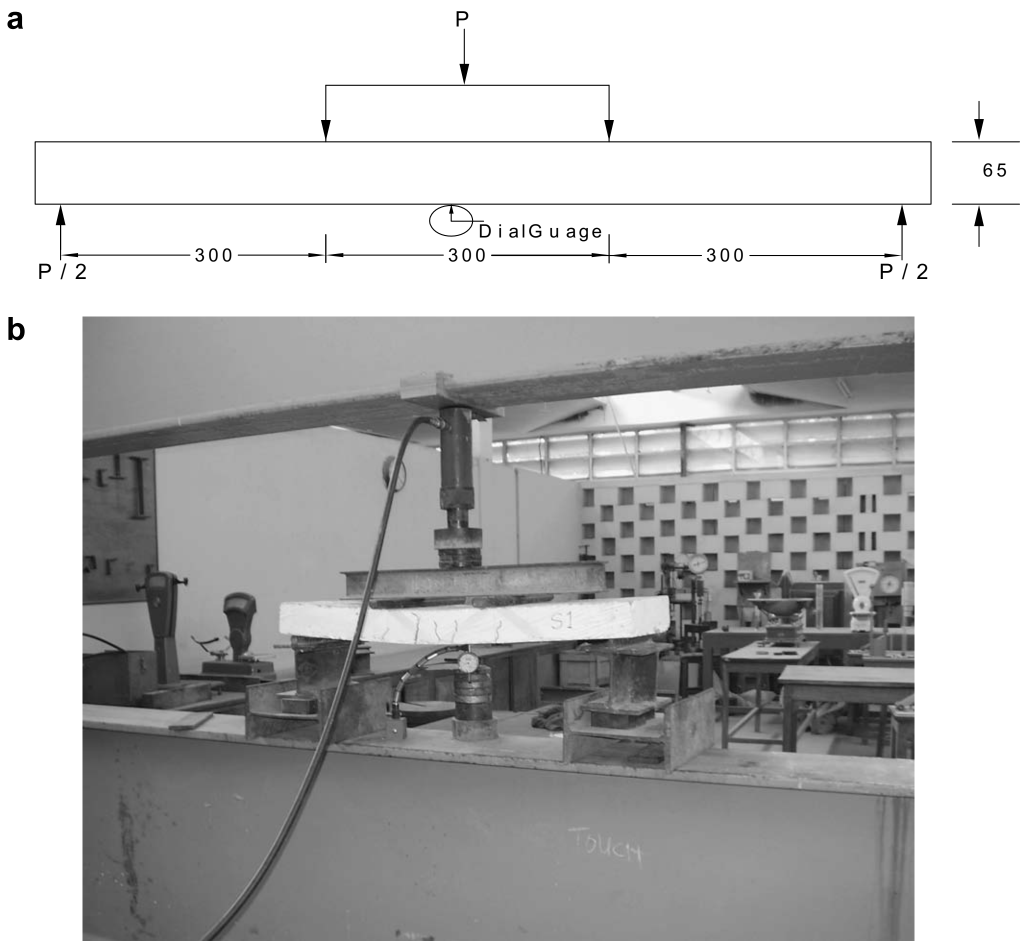

Fig. 1. (a) Schematic sketch of typical experimental set-up. (b) Specimen and Loading Instrumentation.

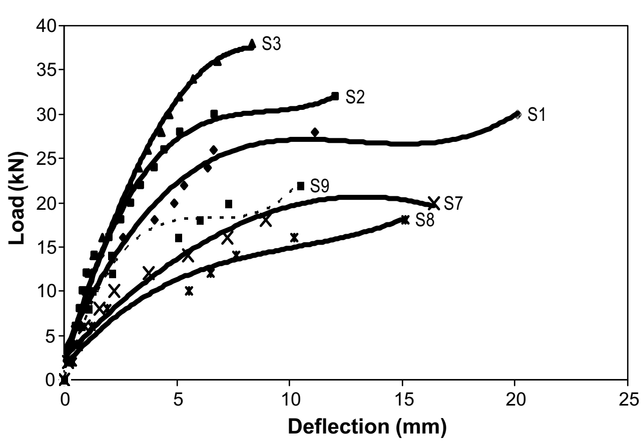

Fig. 2. Load–deflection curves of slabs subjected to monotonic loading.

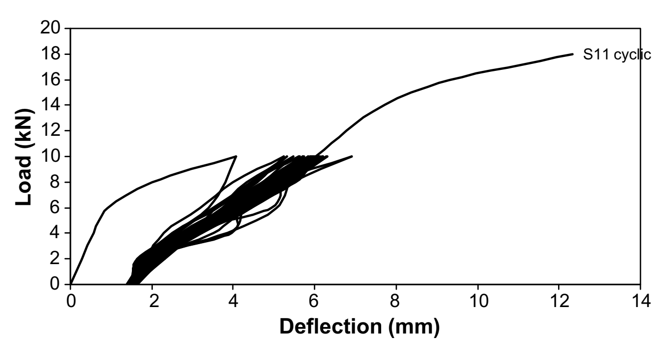

Fig. 3. Load–deflection curves of typical slab subjected to cyclic loading.

34.1 N/mm^2 whilst the tensile strength (modulus of rupture) of concrete ranged from 5.5 to 6.9 N/mm^2 (Table 2).

4.2. Load–deflection curves

The load–deflection curves of the slabs plotted from the test results are shown in Figs. 2 and 3. Fig. 2 shows the curves for slabs S1, S2 S3, S7, S8 and S9 which were all subjected to monolithic loading to failure. Initial loading of the slabs showed approximately linear elastic characteristics until the cracking load P_{cr} was exceeded and the first crack developed at the bottom of the slab within the middle third where maximum bending occurred. After cracking, the gradient of the initial load–deflection curve reduced and continued to reduce gradually until the steel yielded. The post-yield behaviour of the steel reinforced slab then resulted in a third region of greatly reduced gradient within which strain hardening occurred such that a slight increase in load resulted in a large increase in deflection until failure occurred. The load–deflection curves for the slabs subjected to monotonic loading showed different strength and ductility characteristics. Slab S3 which failed by the highest experimental failure load was the least ductile of all the slabs subjected to monotonic loading. This is because slab S3 which contained the highest main reinforcement (1.41%) was over-rein-

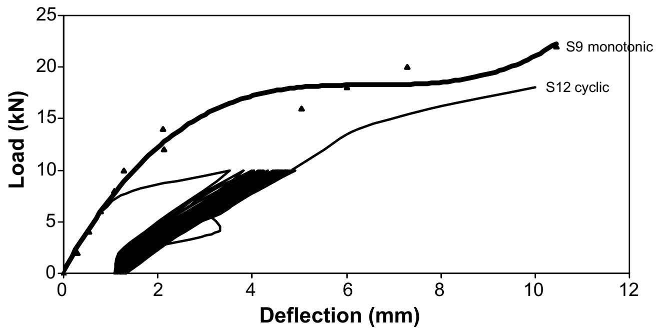

Fig. 4. Comparison of similar slabs (S9 and S12) subjected to monotonic and cyclic loading.

forced and therefore failed in the most brittle mode of concrete crushing and shear bond splitting. Slab S1 which contained one of the minimum main reinforcement (0.55%) exhibited the most ductile behaviour at failure.

Fig. 3 shows the curves for slab S11 subjected to 50 cycles of loading and unloading, and is representative of typical behaviour of the slabs subjected to cyclic loading. It is worthy of note that the envelope of the curve that serves as a boundary to the cyclic load–deflection curves have similar characteristics to those of Fig. 2 already discussed. A comparison of the curves of two similar slabs S9 and S12, subjected to monotonic and cyclic loading, respectively (see Fig. 4) indicates that limited cyclic loading caused a reduction in the post cracking stiffness, ultimate failure load and ultimate deflection as a result of limited hysteretic energy dissipated at service loads.

4.3. Cracking loads, failure loads and shear strength

The cracking loads, the experimental failure loads and the theoretical failure loads of the slabs are given in Table 3. The theoret-

Table 3
Cracking and failure loads

Theoretical failure load (kN)								
Slab no.	First-crack load	Experimental failure load	Concrete section only	Including steel bars in tension				
	P_{cr} (kN)	P_{ult} (kN)	P'_{cr} (kN)	P'_{ult} (kN)	P_{cr}/P_{ult}	P_{cr}/P'_{cr}	P_{ult}/P'_{ult}	P_{ult}/P'_{cr}
S1	14	30	11.52	28.27	0.82	1.21	1.04	2.60
S2	14	32	10.24	27.51	0.73	1.37	1.16	3.12
S3	14	38	9.17	51.84	0.66	1.51	0.73	4.14
S4[a]	15	30	11.52	28.79	0.77	1.3	1.04	2.60
S5[b]	16	24	10.24	27.51	0.64	1.56	0.87	2.34
S6[c]	16	42	9.17	51.84	0.57	1.75	0.81	4.58
S7	10	20	8.99	23.07	0.85	1.17	0.86	2.35
S8	8	18	6.86	16.79	0.86	1.16	1.07	2.62
S9	12	22	6.59	16.52	0.55	1.82	1.33	3.33
S10[d]	9	16	8.49	23.07	0.94	1.06	0.69	1.88
S11[e]	7	14	6.86	16.79	0.94	1.06	0.83	2.04
S12[f]	7	14	6.59	16.52	0.94	1.06	0.84	2.12
				Average	0.77	1.30	0.94	2.81

P'_{ult} – calculated using average steel strength of 368 N/mm^2.
[a] 8 cycles.
[b] 15 cycles.
[c] 20 cycles.
[d] 20 cycles.
[e] 50 cycles.
[f] 50 cycles.

Table 4
Shear strength of slabs

Theoretical shear strength (kN)						
Slab no.	Concrete section alone	Including reinforcing steel				
	P_{S1} (kN)	P_{S2} (kN)	P'_{ult}/P_{S2}	P_{cr}/P_{S1}	P_{ult}/P_{S1}	P_{ult}/P_{S2}
S1	19.26	29.83	0.95	0.73	1.56	1.01
S2	18.33	28.33	0.97	0.76	1.75	1.13
S3	18.81	39.54	1.31	0.74	2.02	0.96
S4[a]	18.63	28.85	1.00	0.80	1.61	1.04
S5[b]	18.63	28.85	0.95	0.86	1.29	0.83
S6[c]	18.52	38.98	1.33	0.86	2.26	1.08
S7	15.94	25.72	0.90	0.63	1.25	0.78
S8	19.34	29.64	0.57	0.41	0.93	0.61
S9	17.64	27.07	0.61	0.68	1.24	0.81
S10[d]	15.73	25.38	0.91	0.57	1.02	0.63
S11[e]	17.86	27.37	0.61	0.39	0.78	0.51
S12[f]	17.86	27.37	0.60	0.39	0.78	0.51
		Average	0.83	0.65	1.37	0.82

[a] 8 cycles.
[b] 15 cycles.
[c] 20 cycles.
[d] 20 cycles.
[e] 50 cycles.
[f] 50 cycles.

ical shear strengths of the slabs are compared with the cracking and failure loads in Table 4.

5. Discussion of test results

5.1. Modes of failure

For a simply supported one-way slab subjected to equal line loads at the third points, the middle third of the span is subjected to pure bending (such that it is under zero shear and maximum bending moment); whilst the remaining sections experience maximum shear force and varying bending moment. The middle third experiences the largest strains and therefore the concrete beneath undergoes cracking first.

The main reinforcement varied from 0.54% to 1.41% of the gross concrete section whist the shear-span-depth ratio (a_v/d) ranged between 4.67 and 8.12. According to theoretical evaluation almost all the slabs were under-reinforced with the exception of S3 and S6. Furthermore, as shown in Table 4, the predicted shear strengths of the slabs with the exception of S3, S4 and S6 exceeded their theoretical failure loads and were therefore not expected to fail in shear. Therefore the failure of the 10 under-reinforced beams were expected to be governed by yielding of the tension reinforcement whilst the other two (S3 and S6) were to fail by concrete crushing.

However, many of the slabs failed by combined modes of flexural tension and flexural shear. Each of the slabs developed at least one shear crack. The shear cracks developed after several flexural cracks had developed. The types and number of cracks and their maximum widths per slab at collapse are listed in Table 4. Typical crack configurations have been illustrated in Figs. 5–7. The presence of shear cracks conforms to previous brittle failure modes associated with other structural components reinforced with steel milled from scrap metal [9,11–13]. This is indicative of the fact that in the design of reinforced concrete components, using this type of steel, a conservative view of shear contribution must be taken by the designer. One such approach is to increase the partial safety factor on shear failure from say 1.25 to 1.5 so that predicted shear strength could be accurate. For both the slabs subjected to

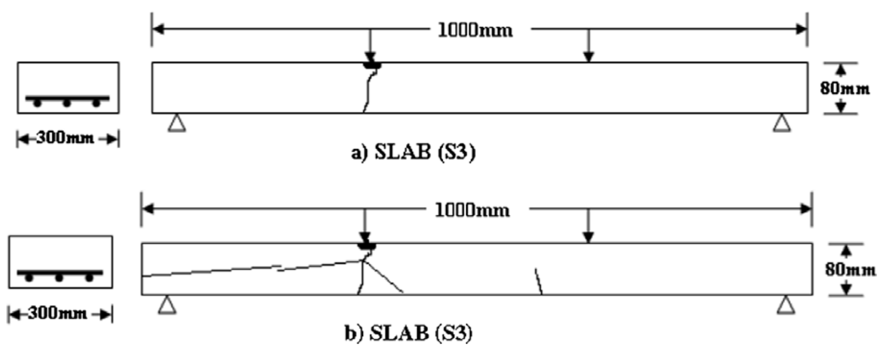

Fig. 5. Crack configuration of S3: (a) front view of the slab and (b) back view of slab.

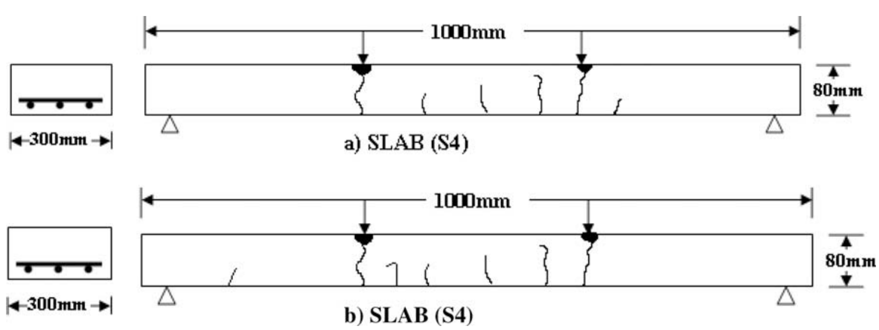

Fig. 6. Crack configuration of S4: (a) front view of the slab and (b) back view of slab.

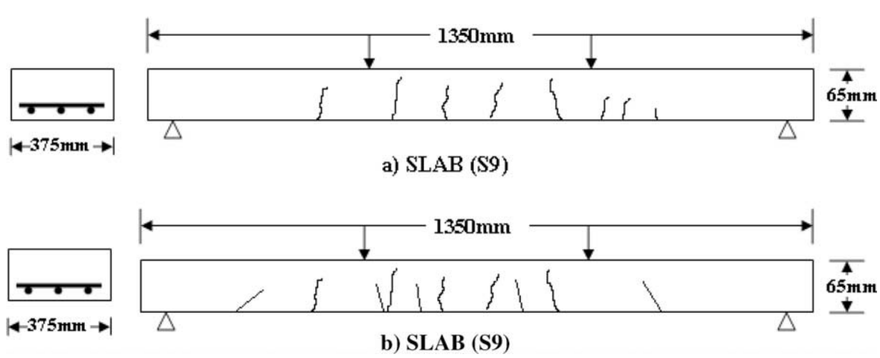

Fig. 7. Crack configuration of S9: (a) front view of the slab and (b) back view of slab.

Table 5
Slabs after failure

Slab no.	Mode of failure		Number and types of cracks	Maximum crack Width (mm)
	Predicted	Actual		
S1	Steel yielding	Flexural in tension bar	5 Pure flexural + 1 flexural shear	1.2
S2	Steel yielding	Concrete crushing and splitting shear	5 Pure flexural + 1 flexural shear	1.2
S3	Concrete crushing	Concrete crushing and shear bond	1 Diagonal shear + 2 bond shear + 2 flexural	1.5
S4	Steel yielding	Flexural in concrete crushing	6 Pure flexural + 1 flexural shear	1.0
S5	Steel yielding	Tensile failure and concrete crushing	5 Pure flexural + 2 flexural shear	2.5
S6	Concrete crushing	Flexural shear crushing	3 Pure flexural + 2 flexural shear	1.0
S7	Steel yielding	Flexural shear crushing	1 Diagonal + 6 Pure flexural + 7 flexural shear	1.0
S8	Steel yielding	Flexural in tension bar	1 Diagonal + 5 Pure flexural shear	1.1
S9	Steel yielding	Flexural in tension bar	1 Diagonal shear + 7 Pure flexural + 2 flexural shear	1.2
S10	Steel yielding	Flexural in tension bar	5 Pure flexural + 2 flexural shear	1.5
S11	Steel yielding	Flexural in tension bar	6 Pure flexural + 2 flexural shear	1.0
S12	Steel yielding	Flexural in tension bar	7 Pure flexural + 2 flexural shear	1.5

monolithic loads (S1, S2, S3, S7, S8, and S9) and those subjected to cyclic loads (S4, S5, S6, S10, S11, and S12) the difference between the global experimental yield loads and the failure loads extracted from the graphs were almost negligible, consequently strain hardening was very low as a result of the shear cracking.

5.2. Cracking and failure loads

From experimental results presented in Tables 3 and 4, it is indicated that the cracking load P_{cr}, averaged 65% of the design shear strength of the concrete without longitudinal tension steel reinforcement contribution, P_{si}, and 130% of the theoretical flexural strength, P'_{cr} of the concrete section alone. This suggested that crack initiation was primarily due to pure bending action rather than shear.

The experimental failure loads of the slabs, P_{ult}, averaged 137% of the design shear strength, P_{s1} based on the concrete section alone and only 82% of the design shear strength of concrete together with tension steel reinforcement contribution to shear, P_{s2}. This implies that the contribution of tension steel reinforcement to the shear capacity in the one-way slab must not be neglected (see Table 5).

The experimental failure loads (P_{ult}) averaged 94% of the theoretical flexural strength (P'_{ult}) and 82% of the total design shear strength (P_{s2}). The corresponding P_{ult}/P'_{ult} of the slab were 103% and 84% for slabs subjected to monolithic and cyclic loads, respectively. Slabs subjected to monolithic loading generally failed at higher loads than those of corresponding slabs subjected to limited number of cyclic loading (typically 30–35% of ultimate loads and 8–50 cycles of the loading–unloading). Therefore, the reinforcing bars in the slabs subjected to cyclic loading were not able to dissipate energy without permanent deformation at service loads. The one-way slabs also exhibited significant recovery of deflection on load removal. In addition the slabs showed further resistance on reloading whilst crack widths were generally between 1 and 2 mm. This may be explained by the fact that the tension bars were un-fractured when flexural shear cracking and concrete crushing occurred, reducing the flexural and shear capacities significantly.

6. Conclusions

A series of laboratory tests were performed on 12 one-way simply-supported slabs reinforced with steel milled from scrap metal. The predicted failure modes of tension steel reinforcement yielding and flexural concrete crushing for the slabs were accompanied by flexural shear cracking or bond shear splitting during testing. On the average, for one-way slabs with span-to-effective depth ratios varying between 14 and 24.37, and shear span-to-effective depth ratio of 4.6 and 8.12, a short-term factor of safety of approximately 1.3 against cracking and one of approximately 0.94 against collapse were obtained from the experimental results. These short-term factor of safety values obtained imply that the prediction of the cracking and failure loads by theoretical methods were also accurate.

It must, however, be emphasized that these values were obtained using the actual average steel strength values of about 370 N/mm^2 but not the 250 N/mm^2 characteristic yield strength for mild steel bars as prescribed by BS8110:85 [14] and designated for such steel milled in Ghana. For practical purposes, therefore, the structural design of one-way concrete slabs reinforced with bars milled from scrap metals in Ghana must be undertaken with the average steel strength values instead of the characteristic mild steel yield values. This is because the actual behaviour of the steel reinforcement milled in Ghana, in terms of strength and ductility requirement lies between that of mild steel and high tensile steel.

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