

Strength and ductility characteristics of reinforcing steel bars milled from scrap metals

Charles K. Kankam*, Mark Adom-Asamoah

Department of Civil Engineering, University of Science and Technology, School of Science and Engineering, Kumasi, Ghana

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Abstract

The physical and chemical properties of reinforcing steel bars milled from scrap metal such as vehicle parts and obsolete machinery in some developing countries were investigated. At present there is inadequate information on the actual behaviour of these reinforcing steel bars which are already in use in structural concrete for the construction of all types of buildings, bridges, hydraulic structures, etc., yet they are classified as mild steel in design specifications. The current state of the art can mean that design of reinforced concrete in such countries may not be fully reliable. The primary observations made with the physical properties are that for the type of mild steel produced in Ghana from re-cycled steel, the characteristic tensile strength is too high with very little elongation leading to limited ductility compared with standard mild steel and high-yield steel. The bars did not exhibit any necking down and cup and cone failure, similar to those observed in the case of standard mild steel plain bars. Concrete beams, reinforced with such bars were fabricated on the basis of under-reinforced concrete design, and tested under monotonic or cyclic loading to study the flexural behaviour of the beams. Tested beams exhibited little deflection and very low ductility and deformation prior to collapse. This paper presents a description of the experimental investigation carried out on the steel bars and reinforced concrete beams. Observations made on the physical and chemical properties of the steel bars, and the ultimate strength, mid-span deflections and crack width under monotonic and cyclic loadings of the beams are included. © 2002 Elsevier Science Ltd. All rights reserved.

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1. Introduction

A complete understanding and knowledge of the real behaviour of construction materials is of prime importance for the proper behaviour of engineered structures. The physical properties of structural materials are expected to meet the demand of the fundamental assumptions underlying structural codes of practice on which designs are based. Locally manufactured reinforcing steel bars from scrap metal are typical examples. In poor developing countries such as Ghana where imported steel is very expensive, milling companies have taken up the challenge to re-cycle obsolete vehicle and machine metal parts for the production of structural and

reinforcing steel. The Ghanaian steel industry, for example, recycles approximately 80% steel scrap estimated annually to be 80 000 tonnes.

Although the behaviour of steel is greatly affected by its chemical composition, heat treatment and the method of manufacturing, there are some physical properties that determine the behaviour of reinforcement for concrete such as yield strength, ultimate strength, Young's modulus of elasticity, Poisson's ratio and percentage elongation. The structural engineer may seem to be more interested in the physical properties of steel, but these properties, however, cannot be realistically attained without the proper chemical composition of the steel.

Typical stress–strain curves for standard steel bars used in reinforced concrete construction when loaded monotonically in tension are shown in Fig. 1. The curves exhibit an initial elastic portion, a yield plateau,

*Corresponding author. Tel.: +233-51-60226; fax: +233-51-60226.

E-mail address: ckkankam@hotmail.com (C.K. Kankam).

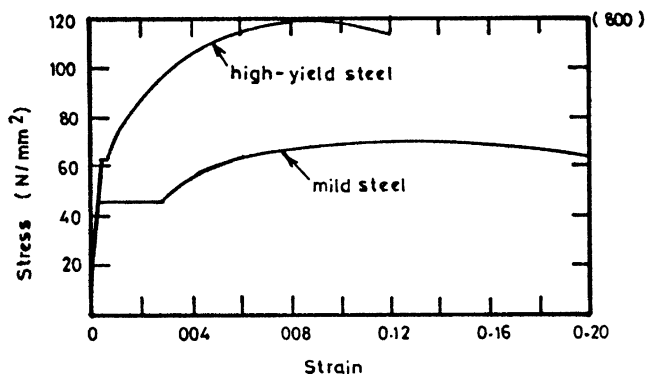


Fig. 1. Typical stress–strain curves for steel reinforcement.

a strain hardening range in which stress again increases with strain and finally, a range in which the stress drops off before fracture occurs. The length of the yield plateau is generally a function of the strength of the steel. High-strength, high-carbon types of steel generally have a much shorter yield plateau than lower strength low-carbon steel. Similarly, the cold-working of steel can cause the shortening of the yield plateau to the extent that strain hardening commences immediately after the onset of yielding. High-strength steels also have a smaller elongation before fracture than low-strength steels.

The minimum strain in the steel at fracture is also defined in steel specifications, since it is essential for the safety of the structure that the steel be ductile enough to undergo large deformations before fracture. ASTM specifications [1] for deformed high-yield bars require an elongation, defined by the permanent extension of an 8 inch (203 mm) gauge length at the fracture of the specimen, expressed as a percentage of the gauge length, which varies with the source, grade, and bar diameter of the steel and ranges from at least 4.5 to 12%. The influence of high-strength steel on cracking and deflection of structural concrete members led to a series of studies in the past into the service behaviour of such steel [2–6].

The British Standard BS 4449: 1988 [7] defines the characteristic strength (f_y) of steel reinforcement as that value of the yield stress below which not more than 5% of the test material should fall. The Ghana Standard DGS 527 [8] places a further limit on the yield stress of mild steel not to exceed 400 N/mm², a value that is 60% more than the characteristic strength. The DGS 527 and BS 4449 both give the minimum elongation at fracture for mild steel to be 22%. The presence of variation in the strength of bars is as a result of such factors as variation in the chemical composition, heat treatment, sampling and testing. With regard to quality control of chemical properties steel manufacturers must give the results of analysis for carbon (C), manganese (Mn), silicon (Si), sulfur (S) and phosphorus (P) for

all steels. Other elements include copper (Cu), chromium (Cr), nickel (Ni) and tin (Sn), which are required to fall within a specific range of values. The DGS 527 and BS 4449 give maximum percent composition of mild steel as S=0.06, P=0.06, C=0.25, Mn=0.65, and Si=0.25.

The various elements have varying effects on the behaviour of mild steel. The carbon level affects the strength and hardening properties of steel. Higher carbon contents increase strength but reduce ductility while manganese controls machinability. Excessive levels of phosphorus and sulfur, which are non-metallic impurities lead to brittle fracture and cracking problems in welded joints. For modern steel-making practice, sulfur and phosphorus are preferably maintained at less than 0.01%. Steel grades with a high level of dissolved gases, particularly oxygen and nitrogen, if not controlled by addition of small elements with a particular affinity for them to float out in the liquid steel at high temperature, can behave in a brittle manner. Manganese, chromium, molybdenum, nickel and copper also affect the strength to a lesser extent than carbon, although their sole effect is on the microstructure of the steel [9].

The present study was conducted to provide detailed information on the strength and ductility properties of reinforcing bars that are milled locally in Ghana from re-cycled metals. The investigation involved the chemical and physical testing of samples of steel bars produced by three different milling companies in Ghana. Twelve rectangular concrete beams reinforced with such steel bars were also fabricated and tested in the laboratory to investigate their strength and deformational characteristics.

2. Experimental method

Samples of steel bars were obtained from three local steel manufacturing companies in Ghana namely, Wahoma, Tema Steel Works and Ferro Fabric randomly referred to as M1, M2 and M3. One hundred specimens were randomly selected from each company from stock-piled batches at a frequency of not less than one test sample per cast as required by BS 4449 [7], DGS 527 [8] and BS18: Part 2 [10]. An Avery Denison universal testing machine was used to perform the mechanical tests. Nine samples were also tested for the chemical properties of the steel using a Spectrolab L/M analytical instrument. Twelve concrete beams were reinforced with different percentages of steel in tension as well as in compression from the three different millers in order to investigate the behaviour of these steel bars in concrete. Details of the reinforced concrete beams are presented in Table 1. The concrete consisted of ordinary Portland cement, river sand and crushed granitic rock of 12 mm maximum size. These materials were combined in the proportion 1:2:4 by weight, respectively. A water–

Table 1
Details of beams

Beam no.	$B \times D$	Span/ effective depth ratio (mm)	Percentage of steel		Concrete strength f_{cu} (N/mm ²)	Steel strength f_y (N/mm ²)
			Tension	Compression		
B1	110×160	12.0	1.55	0.39	26.5	337
B2	110×160	12.0	1.55	0.39	26.5	337
B3	110×135	14.8	1.90	0.48	28.0	337
B4	110×135	14.8	1.90	0.48	28.0	337
B5	110×135	13.9	2.69	0.67	18.5	370
B6	110×165	10.9	2.09	0.52	27.6	370
B7	110×135	13.9	2.69	0.67	30.5	370
B8	110×165	10.9	2.09	0.52	27.2	370
B9	110×160	10.7	1.28	0.32	24.7	490
B10	110×160	10.7	1.28	0.32	24.7	490
B11	110×135	14.0	1.52	0.38	24.9	490
B12	110×135	14.0	1.52	0.38	24.9	490

cement ratio of 0.50 was adopted for mixing the concrete. Beams and control cubes were cast and compacted by shutter vibrator. They were cured under damp hessian sacks for 28 days. The beams were simply supported at their ends and tested to failure using a two-point symmetrical loading system. Beams B1–B5 were subjected to monotonic loading while B6–B12 were subjected to 20 cycles of loading prior to collapse. Deflections at mid-span were measured using a dial gauge reading to 0.01 mm mounted beneath the beam. Crack width was measured on the concrete surface using a crack microscope reading to 0.02 mm. Load was applied in increments and at each load increment, all the required measurements were taken.

3. Test results

3.1. Physical properties

From a statistical analysis, the steel bars exhibited significant variability in yield strength with minimum values averaging 190, 260 and 230 N/mm² for millers M1, M2 and M3, respectively. The mean yield strengths for bars from M1, M2 and M3 were 490, 370 and 340 N/mm², respectively, as indicated in Table 2. The ultimate strengths averaged 560, 550 and 500 N/mm², respectively. The standard deviations were 112, 60 and 61 N/mm², respectively. Fig. 2 shows typical stress-strain curves for the steel bars, and the limited range of inelasticity and plasticity is clear in all cases.

The mode of failure was similar in most of the samples, showing very little or no necking and hardly any cup and cone surfaces. The type of failure indicates brittle behaviour of the steel. The percentage elongation of the samples averaged 9.6, 10.6 and 11.8, respectively, for types A, B and C.

3.2. Chemical properties

The chemical analysis of the tests carried on the steel samples shows that the steel bars from the three millers possessed average carbon contents of 0.20, 0.26 and 0.30, respectively. The silicon content averaged 0.36, 0.19 and 0.09, respectively. The results of the chemical tests are presented in Table 3.

3.3. Behaviour of beams

The cracking, theoretical and experimental failure loads of the 12 beams are presented in Table 4. Typical load–deflection curves for beams B1 and B5 under monotonic and cyclic loadings are shown in Figs. 4 and 5, respectively.

4. Discussion of test results

4.1. Physical and chemical characteristics

Bars manufactured by two companies out of the three licensed in Ghana did not meet the BS4449 maximum limit of 0.25% for carbon requirements for mild steel. The phosphorus and sulfur impurities in the steel bars from all the three companies exceeded the preferred limit of 0.01%. These excess carbon, sulfur and phos-

Table 2
Physical properties

Steel source	Yield stress (N/mm ²)	Ultimate strength (N/mm ²)	Percentage elongation	Type of failure
M1	490	560	9.6	Brittle
M2	370	550	10.6	Brittle
M3	340	500	11.8	Brittle

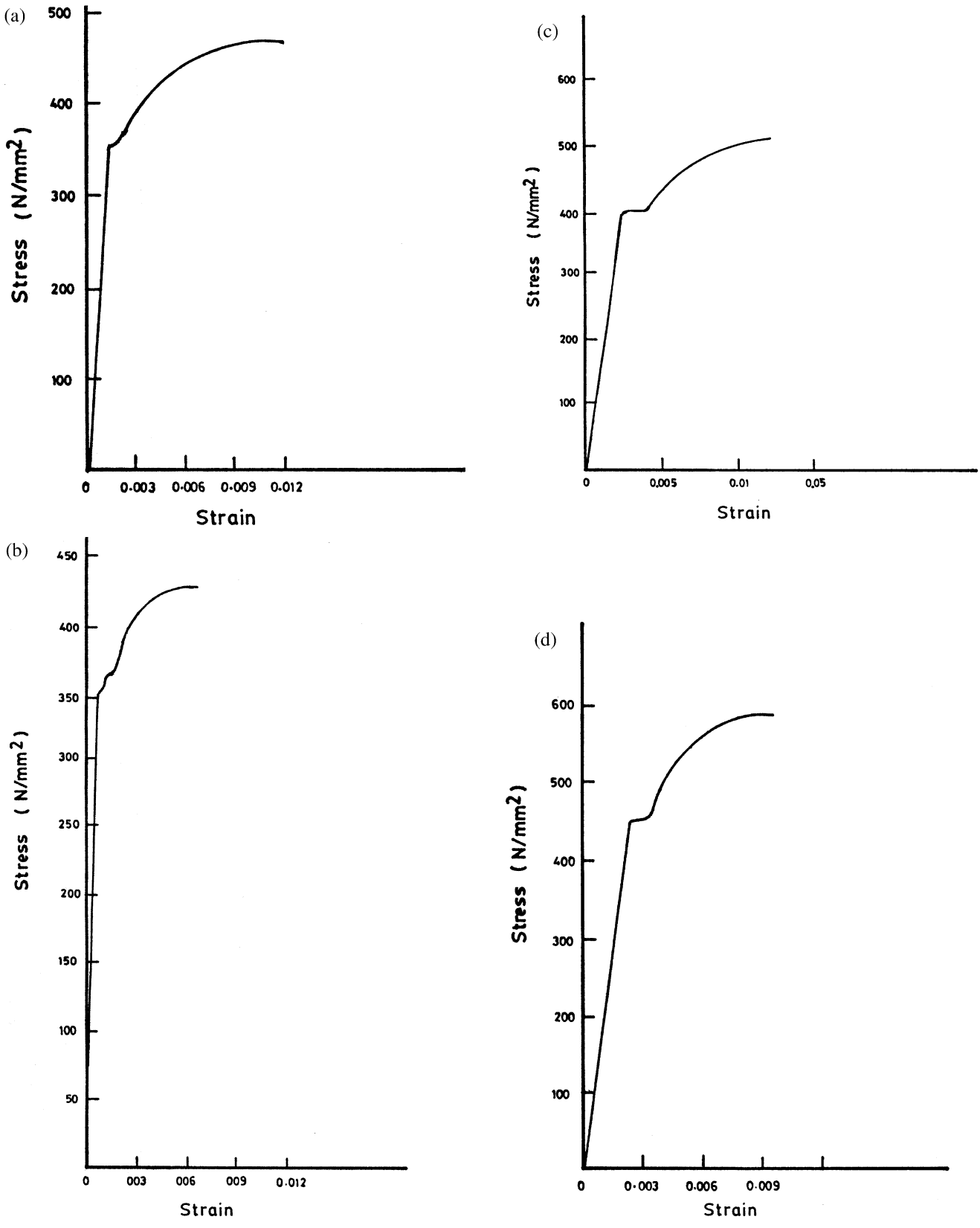


Fig. 2. Stress–strain curves for steel (Millers M1, M2, M3).

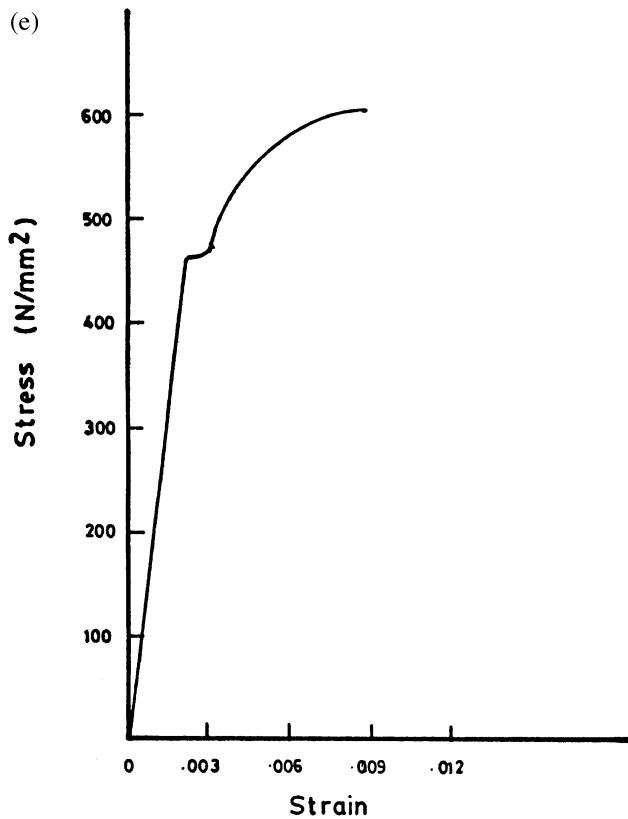


Fig. 2 (Continued).

phorus contents increase the strength and hardness of the steels, and at the same time decrease their ductility, making them brittle. Steel bars from the two companies that had excessive carbon content also exceeded the manganese limit of 0.65% and will therefore be adversely affected as far as machinability is concerned.

The stress–strain behaviour of these steel grades has a very short range of plasticity and strain hardening at constant stress. This is normally typical of high yield carbon steel for which the spread between the yield stress and ultimate stress is relatively small. Lastly, the percentage elongation of all the samples tested was less than the minimum requirement of 22% for mild steel, and the steel bars should therefore be classified as brittle.

The definition of the characteristic strength should not be seen to emphasise solely on a minimum yield stress value, which is more concerned with a lower bound value without any reference to an upper limit. Otherwise, without other important requirements relating to physical properties such as ductility or percentage elongation, all samples which pass for high yield reinforcement also automatically meet mild steel requirements, since in both cases not more than one in 20 samples has a strength less than 250 N/mm². The present building code definition of the characteristic yield strength, however, provides the basis for the production standard control to be governed by the tensile

Table 3
Chemical composition

Steel source	C	Si	Mn	P	S	Cu	Cr	Ni	Sn	V
M1	0.20	0.36	0.45	0.059	0.043	0.29	0.09	0.13	0.03	0.005
M2	0.26	0.19	0.70	0.041	0.029	0.20	0.10	0.13	0.03	0.005
M3	0.30	0.09	0.70	0.050	0.039	0.28	0.19	0.11	0.003	0.003

Table 4
Failure loads and deflections

Beam no.	Theoretical failure load P'_{ult} (kN)	Experimental failure load P_{ult} (kN)	Theoretical deflection at collapse δ'_{ult} (mm)	Experimental deflection at collapse δ_{ult} (mm)	P_{ult}/P'_{ult}	$\delta_{ult}/\delta'_{ult}$
B1	26	34	6.01	8.50	1.31	1.41
B2	26	36	6.36	10.07	1.38	1.58
B3	20.2	30	7.10	10.40	1.48	1.46
B4	20.2	32	7.58	11.94	1.58	1.58
B5	14.2	16	6.44	9.80	1.13	1.52
B6	30.7	38	8.89	7.44	1.24	0.84
B7	23.5	38	15.44	9.30	1.62	0.60
B8	21.6	35	8.39	7.45	1.62	0.89
B9	21.4	38	9.0	6.22	1.78	0.69
B10	21.4	38	9.0	5.83	1.78	0.65
B11	16.2	32	10.0	8.55	1.98	0.86
B12	16.2	32	10.0	8.07	1.98	0.81

Table 5
Crack details and mode of failure

Beam no.	Cracking load (kN)	Max. crack width (mm)	Mode of failure
B1	8	2.0	<i>Beams B1–B5</i> Brittle failure due to diagonal shear in monotonic loading
B2	5	2.3	
B3	3	3.0	
B4	4	3.2	
B5	4	3.0	
B6	5	3.0	<i>Beams B6, B7, B8, B11 and B12</i> Crushing of concrete in flexure after 20 cycles of loading prior to collapse
B7	4	3.0	
B8	6	3.0	
B9	8	3.5	
B10	6	3.6	<i>Beams B9, B10</i> Failure in diagonal shear after 20 cycles of loading prior to collapse
B11	4	3.0	
B12	6	3.0	

strength alone. With the tensile strength being the sole controlling factor for production by steel millers in Ghana, the yield strength generally tends to be too high for mild steel while unfortunately, very little or no recognition is given to ductility as necessarily required for the same type of steel. This situation renders the philosophy of under-reinforced concrete design vulnerable, in a sense that the yielding of the reinforcing steel bars, which is required to give ample warning of an impending failure of a structural concrete member, will hardly occur.

4.2. Structural behaviour of beams

Even though the concrete beams were designed as under-reinforced, expecting the steel which is classified as mild to yield first before the concrete started to crush

with an anticipated large and plastic regime in the load–deflection curve, failure of the beams was largely brittle with very little increased deflection prior to collapse (Figs. 4 and 5). Table 5 presents the modes of failure of the beams. The experimental failure loads were high and averaged approximately 160% over the predicted values which were based on yield strengths of steel of either 340, 370 or 490 N/mm² with a partial factor of safety of 1.15. Under monotonic loading, the maximum deflections exceeded the predicted values on the average by approximately 50%. However, in the case of the beams that were subjected to cyclic loading the actual deflections at collapse were less and averaged approximately 76% of the predicted values as indicated in Table 4. This is likely due to the brittle nature of the steel bars, which is adversely affected by fatigue even under this limited cyclic loading. The modes of failure of the

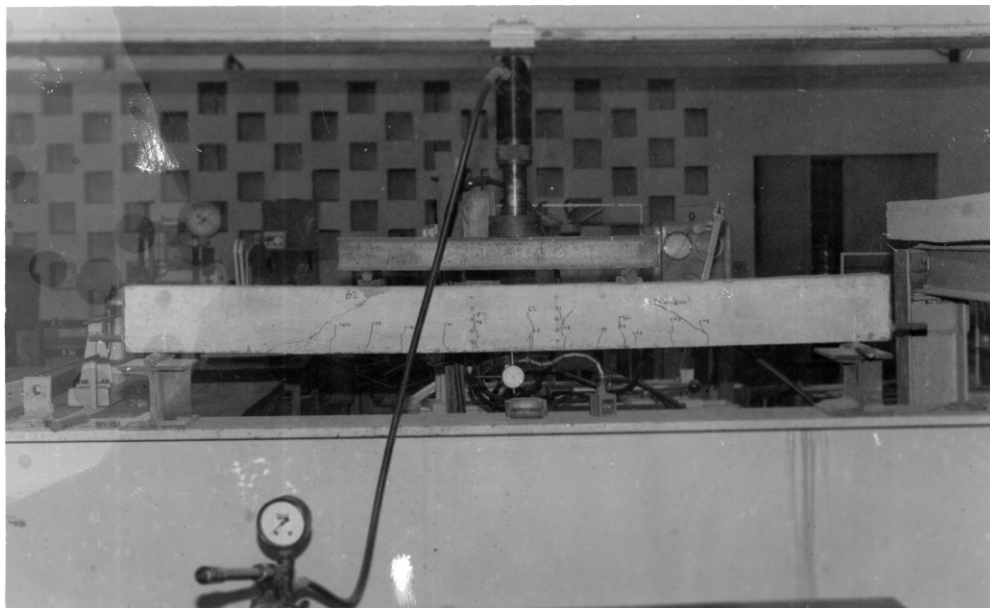


Fig. 3. Beam at failure.

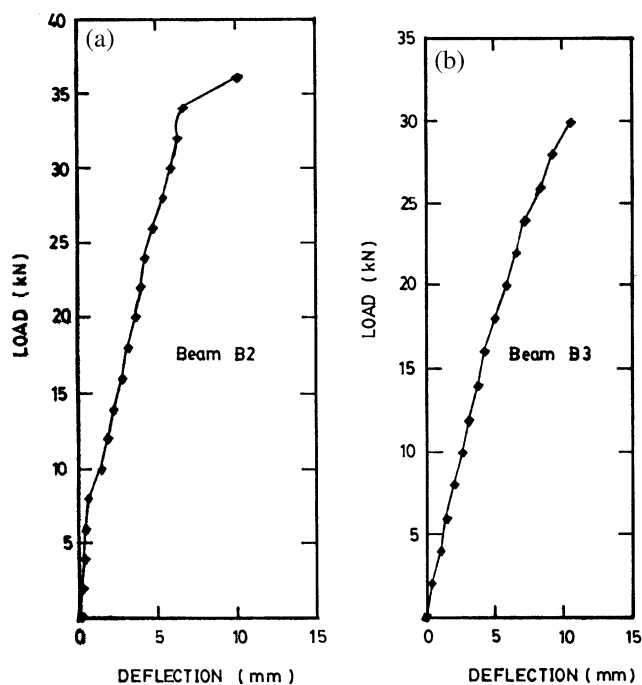


Fig. 4. Load-deflection curves for beams under monotonic loading.

different beams are presented in Table 5. In Fig. 3 is a typical beam near collapse showing several cracks that extended little into the concrete compression zone after being subjected to cyclic loading and failed by crushing of the concrete in flexure.

The ratio of maximum deflection near collapse to deflection at first crack ranges from 6.2 to 42.5 (Table 6). These low ratios indicate limited ductility and plasticity of the concrete beams. Similarly, as shown in Table 6, the post-cracking strain energy of the beams obtained from the areas under their curves of load–deflection relationships ranges approximately from a minimum of 90 to a maximum of 205 Nm.

The problem of control on reinforcing steel becomes an important issue when one considers seismic loading. In seismic design both structural and non-structural damage could occur, but collapse of the whole structure should exhibit ductility during an earthquake. It is important to ensure that in the extreme event of a structure being loaded to failure, it will behave in a ductile manner with large deformations at near maximum load-carrying capacity. The large deflections at near maximum load give ample warning of failure, and by maintaining load carrying capacity, total collapse may be prevented and lives saved. Also, ductile behaviour of members enables the use in design of distributions of bending moments that take into account the redistribution possible from the elastic bending moment pattern. In areas requiring design for seismic loading, ductility becomes an extremely important consideration. This is because the present seismic design philosophy

relies on energy absorption and dissipation of postelastic deformation for survival in major earthquakes. Thus structures incapable of behaving in a ductile fashion must be designed for much higher seismic forces if collapse is to be avoided. Most building codes for seismic loading, however, recommend design of structures to resist only relatively moderate earthquakes elastically; in the case of a severe earthquake, reliance is placed on the availability of sufficient ductility after yielding to enable a structure to survive without collapse. Hence, the recommendations for seismic loading can be justified only if the structure has sufficient ductility to absorb and dissipate energy by post-elastic deformations when subjected to several cycles of loading far within the yield range.

The criterion of special interest in the light of this discussion is the fact that failure of members at ultimate load should occur by tensile yielding of reinforcing steel followed by compression failure in the concrete. The type of reinforcement to use and hence, percentage of steel, therefore becomes a priority. In order to ensure that tensile yielding will occur in steel prior to crushing of concrete in under-reinforced concrete design, it is necessary that the properties of the reinforcement be fully known to the engineer. Thus, the appropriate steel to use in earthquake design should be a steel whose properties are controlled within given limits, has a definite yield plateau and a definite strain hardening region.

5. Conclusions

Various physical and chemical tests conducted on locally manufactured bars in Ghana speculate a stress–strain behaviour falling between that of mild steel and high-yield steel. The results of the tests have shown that the characteristic strength of locally produced steel bars is not consistent. Building codes of practice should place an upper limit value on the yield stress of mild steel, just as the DGS 527 states that the yield stress of mild steel bars shall not exceed 400 N/mm².

Even though most of the individual samples had strengths greater than the characteristic strength, a basis which erroneously gives manufacturers confidence in their quality control level, the fact that the percentage elongation of all the test samples did not meet the code minimum requirement of 22% for mild steel, places the behaviour of the locally milled steel bars in a domain between that of mild and high-yield steel. The average percentage elongation of the bars varied from 9.6 to 11.8. This requires a revision of the specifications generally assigned to locally milled steel from re-cycled metal. The current specification that the steel is mild rather than medium-yield inevitably leads to an unnecessarily high steel ratio in design. This may result in making structural members brittle and unsafe during

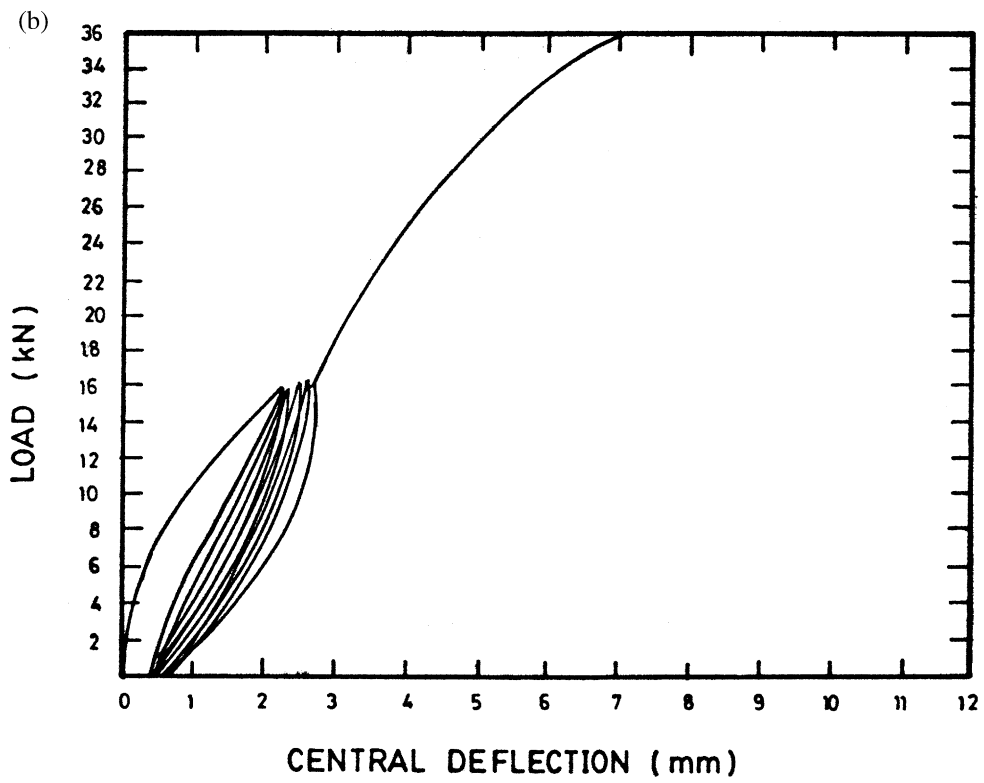
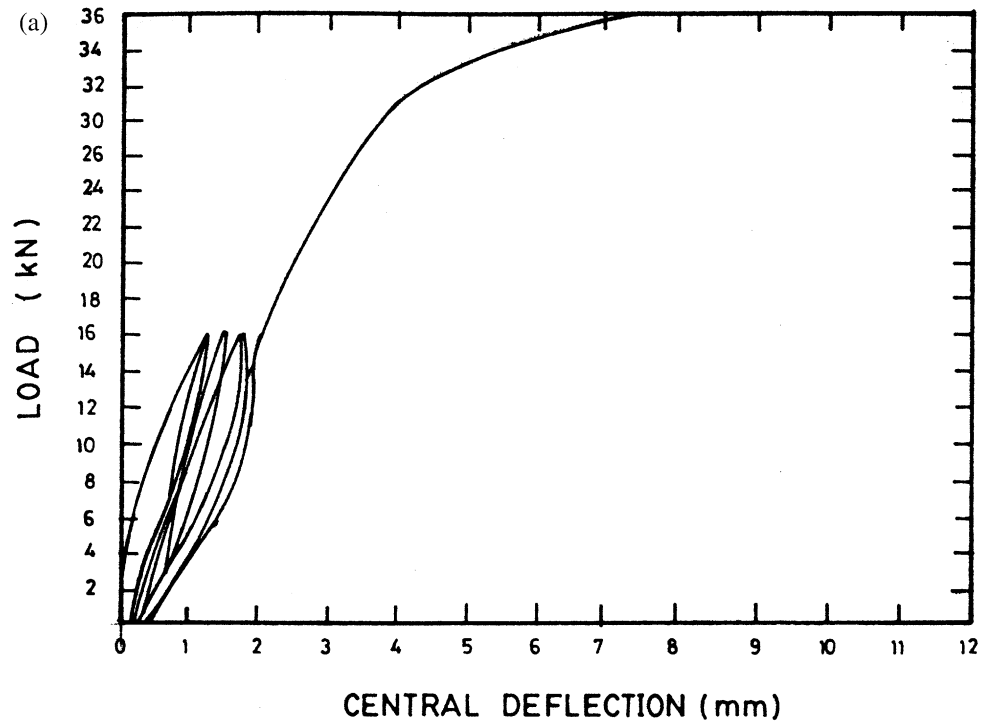


Fig. 5. Load-deflection curves for beams B8 and B7 under cyclic loading.

Table 6
Post-cracking deflection and energy absorption of beams

Beam no.	Deflection at first crack δ_{cr} (mm)	Deflection at collapse δ_{max} (mm)	δ_{max}/δ_{cr}	Post-cracking strain energy (Nm)
B1	1.0	8.5	8.5	157.5
B2	0.7	10.0	14.3	190.7
B3	0.3	10.5	35.0	168.3
B4	0.61	12.0	19.7	205.0
B5	0.8	9.8	12.2	90.0
B6	0.2	8.5	42.5	178.5
B7	0.3	9.3	31.0	189.0
B8	0.31	7.45	24.0	153.5
B9	1.0	6.2	6.2	119.6
B10	0.6	5.8	9.7	114.4
B11	0.45	8.6	19.1	146.7
B12	0.8	8.0	10.0	136.8

earthquake occurrence and other dynamic loadings. Concrete beams reinforced with such locally manufactured steel bars failed suddenly with very little ductile behaviour even though they had been designed as under-reinforced. Cyclic loading, though limited in extent, further reduced the low ductility in the bars.

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