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Evaluation of Ductility Index of Concrete Beams Reinforced with Rebars Milled from Scrap Metals

J.A Apeh[1,c] and O.G Okoli[2]

[1]*Department of Building, Federal University Technology, Minna*, Nigeria
[2]*Department of Building, Faculty of Environmental Design, Ahmadu Bello University, Zaria*, Nigeria

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ABSTRACT

Ductility index is essential both in structures and structural elements in service. Its inadequacy may lead to brittle failure and jeopardize the lives of occupants. In reinforced concrete beams that experience large inelastic deformation in service, its ductility index cannot be over- emphasized. In Nigeria, the steel sector is now sustained through the recycling of scrap metal obtained mainly from municipal solid wastes which find application in the construction Industry. The study evaluated the ductility index of a rectangular concrete beam reinforced with rebars milled from scrap metal. This was achieved by designing the beam, produced samples and assessed its behavior under load experimentally and analytically with emphasis on the deflection ductility index. Eighteen (100 mm x 200 mm x 1000 mm) concrete beams reinforced with rebars milled from scrap metal were produced; six each with concrete strength of 20.33, 26, 30 N/mm^2 and steel ratio (ρ) of 0.0058 to 0.012 respectively. The samples were tested under a four- point loading and analyzed using the Hognestad models for concrete and steel, theoretical equations of strain compatibility and equilibrium of forces at the beam section. Based on the test data obtained in the laboratory and analytical approach, the failure mode of most beams was classified as ductile flexural failure accompanied by yielding of the tension steel preceding the crushing of concrete. The flexural capacity of the test samples ranged from 43.25 to 88.25 Kn with a deflection ductility index of 1.72 to 2.80. The analytical load-deflection relationship compared with experimental values show good agreement. This confirms the applicability of the theoretical approach which provides a useful tool for evaluating the deflection ductility index and load-deflection response of concrete rectangular beams reinforced with rebars milled from scrap metal. Key words: Ductility index, concrete beams, Rebars, scrap metal, deflection response.

1.0 Introduction

The traditional method of producing construction materials is the continuous use of natural or artificial resources which is depleting. Besides, the industrial and urban management systems are also generating solid wastes most often dumping them in open fields. These activities pose serious detrimental effects on the environment. To safe guard the environment; efforts are being made for the recycling of different types of wastes with a view to utilizing them in the production of various construction materials.

[C] Corresponding Author: CK Mukhopadhyay; E-mail: apehjoe@futminna.edu.ng

In developing countries like Nigeria where imported steel bars is very expensive, Milling companies has taken up the challenge to recycle obsolete vehicle and machine scrap metal for the production of structural and reinforcing steel bars which is currently sustaining the steel sector (Ohimain, 2015). The use of such reinforcing steel bars regarded as mild steel in construction works means that the construction of structural elements may not be fully reliable. Kankam and Asamoah (2002) opined that actual behavior of such reinforcing steel bars has not been ascertained; yet they are being used in construction of buildings and other infrastructures as mild steel bars to the detriment of occupants and the society at large. A complete understanding and knowledge of the real behavior of construction materials like rebars milled from scrap metals is of prime importance for the proper behavior of engineering structures.

Ductility of a structure, structural element is its ability to undergo inelastic deformation and with no substantial reduction in strength. Ductility index is used to measure ductility. It is defined as the ratio of curvature, rotation and deflection at ultimate state to the deflection at yield point of steel as in equation (1):

$$\mu_\phi = \frac{\Phi u}{\Phi y}, \quad \mu_\theta = \frac{\theta u}{\theta y}, \quad \mu_\Delta = \frac{\Delta u}{\Delta y} \tag{1}$$

where ϕ_u is curvature at ultimate state, ϕ_y is curvature at yield point of steel; θ_u is rotation at ultimate state and θ_y is rotation at yield point of steel; Δ_u is deflection at ultimate state, Δ_y is deflection at yield point of steel and μ_ϕ is the curvature ductility index, μ_θ is the rotation ductility index and μ_Δ is the deflection ductility index.

The displacement ductility index required of typical reinforced concrete beams may vary between 1 for elastically responding beam to 6 for ductile beams depending on the level of deformation used to determine the required strength of the beam (Kumar et al., 2008). Ashour (2000) mentioned that the displacement ductility index, μ_Δ, in the range of 3 to 5 is considered imperative for adequate ductility especially for seismic design and redistribution of moments. Generally, high ductility ratios indicate that a structural member is capable of undergoing large deformations prior to failure.

In the search for adequate ductility, Kumar et al., (2006) worked on six high performance concrete beams with varying concrete strength and tensile steel ratio (ρ) and obtained displacement ductility index of 1.17 to 2.26. Zaki et al., (2011) used steel slag coarse aggregate and obtained values ranging from 2.22 to 9.66. However, the environmental impact of steel production in terms of co_2 emission and particulates into the air leading to global warming necessitates the use of steel bars milled from scrap metal because the environmental impact is very low (Johnson, 2006).

The work of Asamoah and Kankam (2002) on flexural behavior of Twelve simply supported slabs reinforced with steel bars milled from scrap metals and subjected to a four point loading recommends that an average steel strength of 370 N/mm^2 for steel bars milled in Ghana should be used in reinforced concrete design rather than the characteristic strength of 250 N/mm^2 conventionally prescribed by BS 8110 for mild steel. In a related study by Kankam and Asamoah (2002), they established that the ductility index of concrete beams reinforced with steel bars milled from scrap metal in Ghana is in the range of 0.81 to 1.58 with brittle mode of failure. However, the study did not address flexural behavior in terms of deflection and stiffness of the beams. Using recycled technology, Asamoah et al., (2009) established that 50% of recycled aggregates and rebars milled from scrap metal can be used for lintels in single storey buildings provided the design is in accordance with BS 8110.

The studies reviewed so far addressed evaluation of ductility index of concrete beams reinforced with rebars produced conventionally and rebars milled from scrap metal. It showed the influence of steel ratio, concrete strength on ductility index of RC beams. There is paucity in literature on the evaluation of ductility index of concrete beams reinforced with rebars milled from scrap metal in Nigeria and this is the focus of the study.

2.0 Experimental study

2.1. Test specimens

All specimens were designed to simulate typical field behavior of concrete beams. All beams were rectangular section of 100 mm width and 200 mm deep with a nominal length of 1000mm, and effective span of 900 mm. The concrete dimensions of the test specimens were kept constant and as shown in Figures 1a and 1b. Details of the tested beams are shown in Table 1. The steel ratios for all beams satisfied the minimum and maximum values recommended in ACI 318 -05. Three compressive strengths were used in design. All specimens were loaded statically up to failure using a four-point loading scheme.

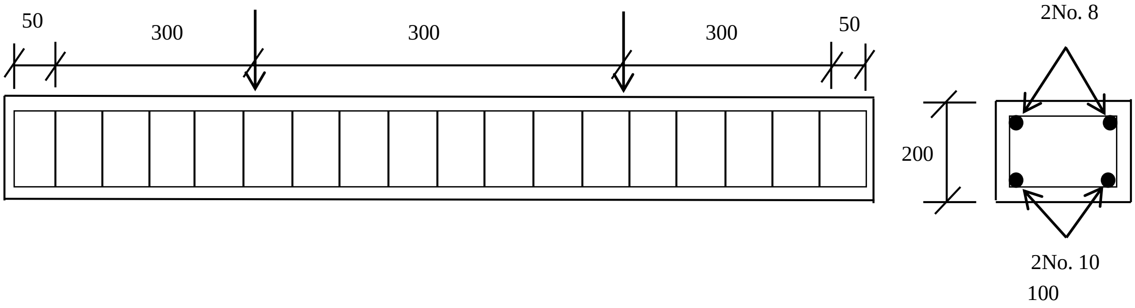

Figure 1a. Test specimen's dimensions.

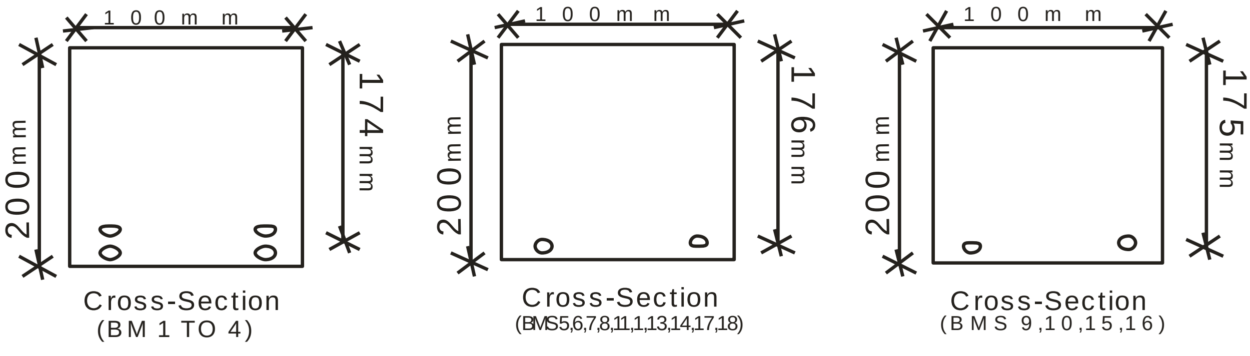

Figure 1b. Beam cross section dimensions.

2.2. Materials

The steel reinforcement used in the current study was obtained from the open market (Dei-Dei) Abuja, Nigeria. Three typical sample bars, Φ 8 mm and Φ10 mm were tested for each beam for their tensile strengths and young modulus of elasticity. Three concrete mixes were used for all beams designed in accordance with the DOE Method using locally manufactured ordinary Portland cement with a specific gravity of 3.15, locally available sand with a fineness modulus of 2.32 and crushed granite aggregate with a maximum nominal size of 19 mm. Three standard 150 mm x 150 mm x 150 mm cubes were cast with each beam, cured and kept in the same environmental conditions for 28 days, tested and obtained concrete compressive strengths of 20.33, 26.74 and 30N/mm^2 respectively. Two standard 100 mm x 200 mm x 500 mm Prisms were also cast for each beam used to determine the modulus of rupture of concrete.

2.3. Beam design

The beam is assumed to be a lintel bearing three courses of block work in a residential Building. The design of a flexural member takes into account the overall behavior of the member throughout the service range and up to the nominal capacity of the member. Flexural members are required to have reinforcement ratios, (ρ), not greater than 75 percent of the balanced reinforcement ratio, ρ_b (ACI 318-05) for satisfactory behavior. The steel ratio (ρ) used is in the range of 0.0058 to 0.012. This is about 22 to 40% of the balanced steel ratio (ρ_b).Thus the test specimens were designed as under-reinforced. Details of beam section, materials strength are shown in Table 1.

Table 1. Details of test specimens.

Beam ID	Width (mm)	Depth (mm)	f_{cu} (N/mm^2)	steel ratio (ρ)	f_y (N/mm^2)
Bm 1	100	200	21.33	0.012	400
Bm 2	100	200	21.33	0.012	360
Bm 3	100	200	21.33	0.0092	400
Bm 4	100	200	21.33	0.0092	360
Bm 5	100	200	21.33	0.0058	387
Bm 6	100	200	21.33	0.0058	353
Bm 7	100	200	26.71	.012	390
Bm 8	100	200	26.71	0.012	350
Bm 9	100	200	26.71	0.0092	395
Bm 10	100	200	26.71	0.0092	355
Bm 11	100	200	26.71	0.0058	390
Bm 12	100	200	26.71	0.0058	355
Bm 13	100	200	30.00	0.012	385
Bm 14	100	200	30.00	0.012	355
Bm 15	100	200	30.00	0.0092	395
Bm 16	100	200	30.00	0.0092	355
Bm 17	100	200	30.00	0.0058	390
Bm 18	100	200	30.00	0.0058	355

2.4. Beam Production

Beam moulds (100 mm x 200 mm x 1000 mm) were cleaned and oiled before casting. Eighteen beams were cast and de molded after 24hrs and cured for 28days prior to testing. Two Prisms (100 mm x 200 mm x 500 mm) for each beam were also cast and cured for 28 days alongside the beams for the determination of modulus of rupture, f_r, of concrete.

3.0 Instrumentation and Test Procedure

To monitor the behavior of the tested beams under the applied loading, a dial gauge to measure mid-span deflection was placed at the soffit of the mid-span of the test specimen (Plate I). All beams were tested under a four-point loading scheme. Each beam was positioned on top of a strong graduated structural arm of the testing machine fixed with movable supports. The supports (Roller and Pin) were adjusted for an effective span of 900mm. Each beam was tested up to failure with a Universal testing machine with a hydraulic jack (Plate I). The loading procedure comprised one loading cycle, with the load incrementally increased by 5kN up to the first crack Load. Mid-span deflection was measured with the dial gauge having a sensitivity of 0.002 mm. The loading continued at same incremental rate aforementioned to yielding of the tensile steel and ultimate failure of the beam. Crack patterns at first crack load and failure loads were duly observed.

All beams were white washed prior to loading to mark crack pattern. First crack load, yield and ultimate loads and corresponding deflections were recorded accordingly. A load-deflection graph was plotted for all the beams which defined its behavior when loaded from inception to failure.

Plate I – Test set-up.

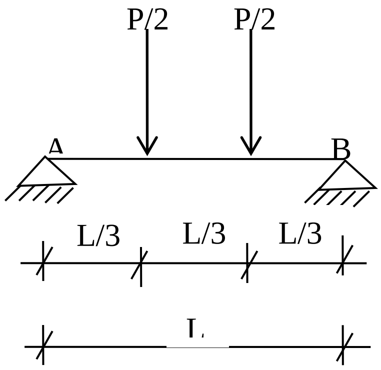

Figure 2: Beam set up

4.0 Analytical computation

An incremental deformation technique assuming strain compatibility was used to predict the flexural behavior of the beam as well as assess the deflection ductility index. The beam section (Figure 3) is divided into fibers with assumed value of strain acting on each fiber. The neutral axis depth, c, is obtained by iteration and summation of the force components in the beam section. The technique used Hognestad models for concrete and steel and theoretical equations based on equilibrium of forces.

Four assumptions were made in developing the technique:

(i) The bond between steel and concrete is perfect.

(ii) The beam fails either by concrete crushing (when $\varepsilon_c = 0.003$) or tensile failure of the steel by yielding (when $\varepsilon_{st} > 0.002$).

(iii) The tensile strength of concrete is neglected.
(iv) Plane section remains plane before and after bending.

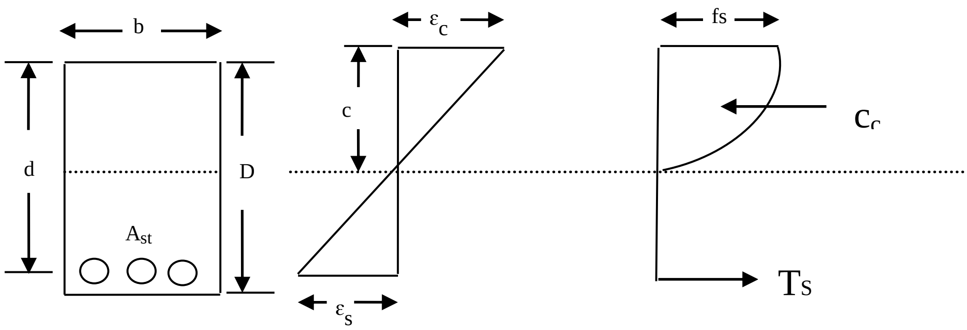

Figure 3. Stress – strain distribution at beam section and diagram of forces.

The technique first assumes the first concrete fiber from the neutral axis to be in compression under a strain, $\varepsilon_c = 0.0001$. For this strain the neutral axis depth of the fiber is calculated by applying equilibrium condition of forces and iteration. Once the neutral axis depth and compressive strain of concrete are known, the technique uses the force in tension, T_s and compression, C_c to calculate the moment at both sections. The overall moment of the fiber is the summation of moment at tensile and compressive sections.
The load acting on the fiber is obtained from the overall moment (Fig. 2):

$$P = \frac{6M}{L} \tag{2}$$

The corresponding deflection due to the load is calculated using the relation:

$$\delta = \frac{23pl^3}{1296EI} \tag{3}$$

Another fiber is chosen and the assumed strain increased ($\varepsilon_c = 0.0002$). The aforementioned steps are repeated to obtain another set of load-deflection relation. The iteration is terminated when a failure criteria is reached; either concrete section crushes ($\varepsilon_c = 0.003$) or tensile steel yields ($\varepsilon_{st} > 0.002$). A load-deflection curve is plotted which defines the flexural behavior of the beam. Ductility index of the beam is assessed in terms of deflection which is the ratio of deflection at ultimate state to the deflection at yield point of steel as expressed in equation (1).
The flexural capacity of the beam under tensile, compressive or balanced failure can be evaluated on the basis of conventional procedure recommended by the ACI 318-08 code as expressed in equation (4):

$$M_n = \rho fybd^2 \left(1 - 0.59\frac{fy}{fcu}\right) \tag{4}$$

Where:

Ρ is tensile steel ratio, f_y is yield strength of steel, f_{cu} is 28- days compressive strength of concrete; b is width of beam, d is effective depth of beam.
The code also recommends that the first cracking moment, M_{cr} is given by the relation as expressed in equation (5):

$$M_{cr} = fr\frac{I}{yt} \tag{5}$$

Where:

$f_r = 0.62\ (f_{cu})^{0.5}$ is the modulus of rupture of concrete, f_{cu} is the 28 days compressive strength of concrete, I is the gross moment of inertia of the beam section.

5.0 Test results and discussion

A summary of the test results is presented in table 2 and 3 including failure loads and ductility indices of the beams. The failure mode of most beams was classified as ductile flexural failure accompanied by yielding of the tensile reinforcement preceding the crushing of concrete.

5.1. Flexural Behavior

The load-mid span deflection of experimental and analytical behavior of beams 1 and 2 is shown in Figures 4 and 5. Linear behavior was observed up to the initiation of the first crack at an average load level of 15 KN followed by a non-linear behavior with significant stiffness reduction up to yielding of tensile steel and then ultimate failure. Similar trend were observed for the rest beams. This is inherited from the linearity of both concrete and steel at initial stage of loading. The non – linearity observed for the beam 1 and 2 and others (not shown) could also be attributed to the non- linearity of the stress-strain curves of both materials at later stage of loading.

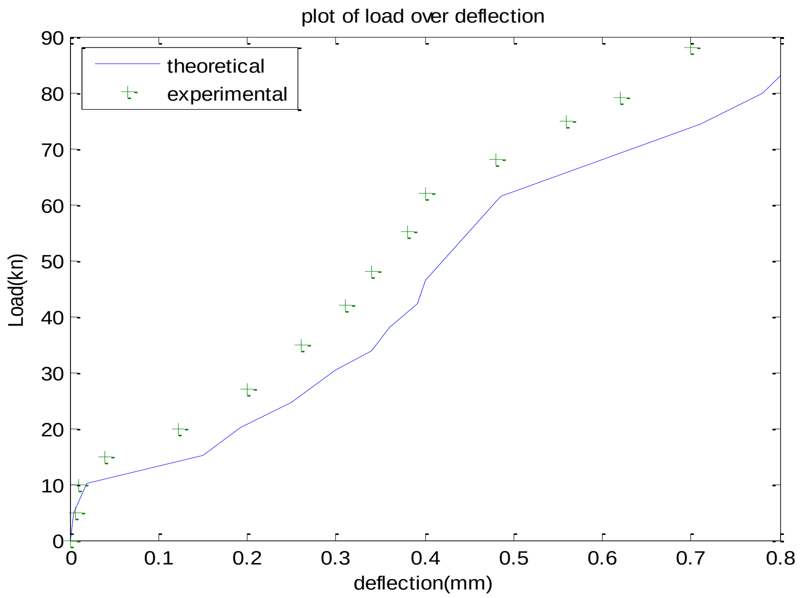

Figure 4. Experimental and Analytical Load-deflection response of beam1.

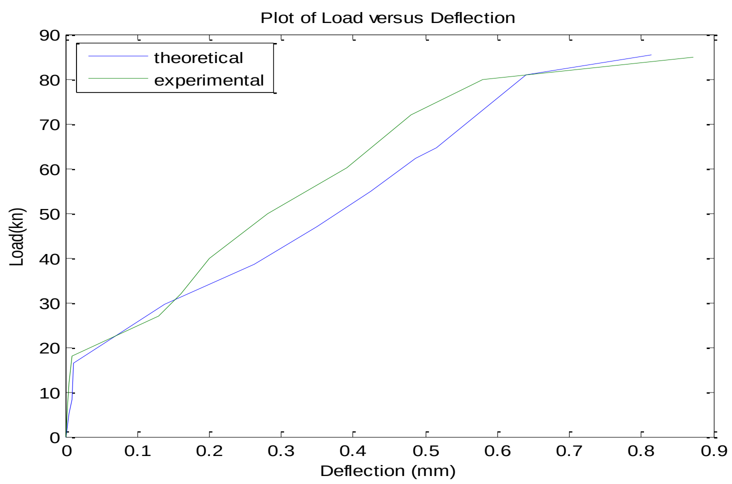

Figure 5. Experimental and Analytical Load – deflection behaviour of beam 2.

To evaluate the load-mid span deflection response of the beams, the behaviour of each beam is compared with others based on steel ratio, ρ, used. The load – mid span deflection behaviour for beams 18, 2, 4 and 6 is shown in Figure 6. Linear behaviour was observed for all beams up to the initiation of the first crack followed by a non-linear behaviour with varying reduction in stiffness up to failure. Yielding of steel bars was observed for beams 1 and 2 (Figures 4 and 5) at loading levels of 80 and 78KN and for beams 4 and 6 at 64 and 52 KN respectively. The corresponding mid-span deflections were 0.62, 0.70, 0.85 and 0.80 mm for the beams.

In Figure 6, beams 2 and 4 exhibited low reduction in stiffness compared with beams 6 and 18. High reduction in stiffness is primarily influenced by low reinforcement ratio (0.0058) used in beams 6 and 18 compared with 0.012 used in beams 2 and 4. The non-linear behaviour of beams from cracking up to failure is due to non – linearity of the stress – strain relationship of the rebars milled from scrap metal. For beams 1 and 2 (Figures 4 and 5), the load- mid span deflection response of the experimental and the theoretical values are similar. This means that the theoretical approach predicts the load-deflection behaviour of the beam within reasonable limits.

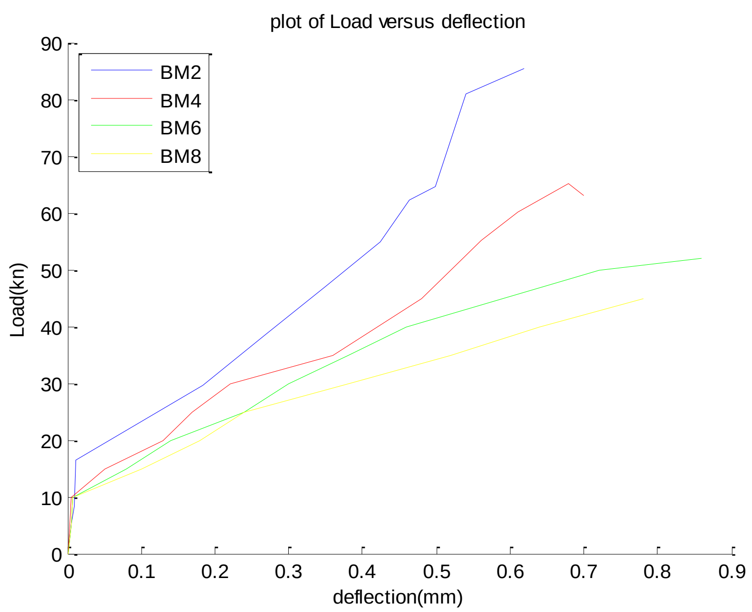

Figure 6. Load – deflection behaviour of Beams 2, 4, 6 and 18.

The load – mid span deflection response for beams 3, 5, 11 and 17 is shown in figure 7. Linear behaviour was observed for the beams up to initiation of the first crack at a load level of approximately 16Kn. This can be attributed to the stress – strain relationship of the concrete section of the beam which is linear prior to cracking. The post cracking behaviour is non-linear with varying stiffness reduction up to failure. The non – linear behaviour can be attributed to the non-linearity of the stress – strain relationship of the rebars. Yielding of steel bars was observed at load levels of 58 and 50Kn for beams 11 and 17, 80 and 87Kn for beams 3 and 7. Their corresponding deflections are 0.62, 0.63 for beams 3 and 7, 0.80 and 0.70 mm for beams 11 and 17 respectively. Beams 11 and 17 exhibited higher stiffness reduction after cracking due mainly to amount of steel ratio (0.0058) used and the yield strength (f_y = 355N/mm^2) as compared with steel ratio (0.012) and yield strength, f_y = 400N/mm^2 used for beams 3 and 7.

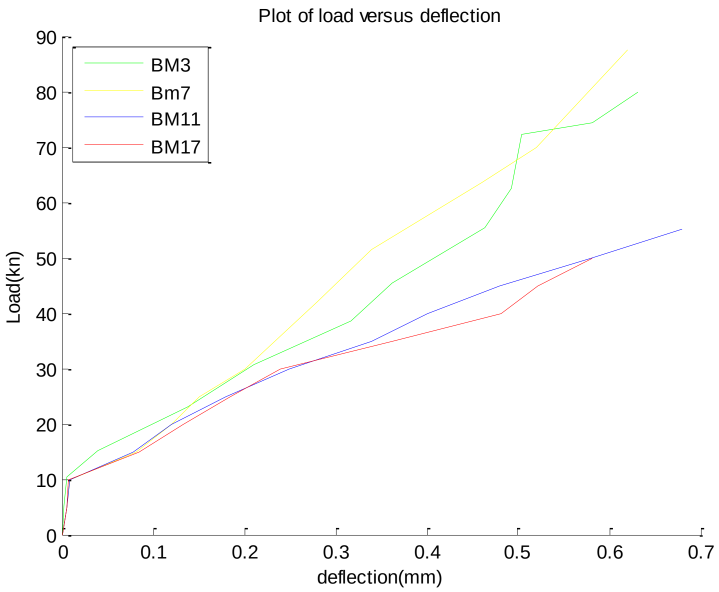

Figure 7. Load – deflection behaviour of beams 3, 7, 11 and 17.

The load – mid span deflection for beams 5, 9, 15 and 8 as shown in Figure 8 exhibit similar behaviour as aforementioned.

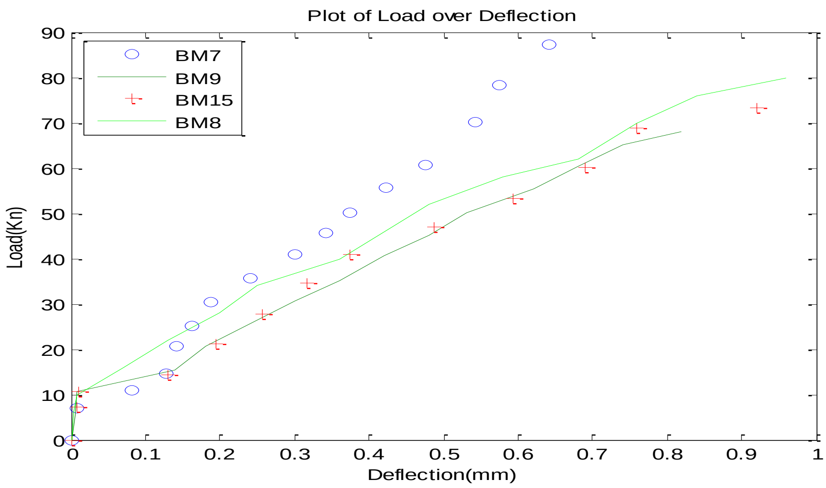

Figure 8. Load – deflection behaviour of beams 7, 9, 15 and 8.

5.2. Cracking and ultimate Loads

The theoretical first cracking load and flexural capacity of the beam are compared with experimental values in table 2. From table 2, there is no significant difference in first cracking load for the beams. This is because prior to cracking, load on a beam is resisted wholly by its concrete section which is dependent on concrete grade. However, for flexural capacity for beams, there is an average of 42% increase on flexural capacity for beams with steel ratio (ρ = 0.012) compared with that of beams with steel ratio = 0.0058, 33% increase for beams with steel ratio (ρ = 0.0092) compared with that of beams with steel ratio (ρ = 0.0058) respectively. This result shows that flexural capacity of beams increase with increase in steel ratio.

Table 2. First cracking Load and Flexural capacity of beam.

Beam ID	**First crack Load (KN}**	**Flexural Capacity (KN)**
BM 1	15.81	88.25
BM 2	14.32	82.00
BM 3	15.00	79.00
BM 4	15.90	64.40
BM 5	18.00	55.70
BM 6	16.00	52.75
BM 7	14.00	87.25
BM 8	21.00	79.55
BM 9	15.20	69.45
BM 10	15.40	65.00
BM 11	15.20	58.75
BM 12	15.40	55.15
BM 13	14.30	87.15
BM 14	14.50	82.25
BM 15	14.30	72.10
BM 16	14.50	66.25
BM 17	18.00	50.00
BM 18	15.00	43.25

5.3. Crack pattern and failure mode

The crack patterns at first crack load and collapse loads for the tested beams are shown in Plates I, II and III. From the crack patterns, it shows that some of the beams failed by flexural failure (yielding of steel) while others is by flexure – shear failure. For this type of failure, a crack normally initiates in the vertical direction and as the load increases, it moves in an inclined direction due to the combined effect of shear and flexure. If the load is increased further, cracks propagate to top and the beam splits. But for flexural failure, the cracks are solely vertical.

Plate I: Early first crack at mid-span of Beam 4


Plate II: Crack pattern for shear and flexural failure of beam


Plate III: Crack pattern for flexural failure of beam

5.4. Deflection ductility index

For a general reinforced concrete section and a specified reinforcement ratio, the load – deflection relationship of the section can be established. From the load– deflection relationship, the deflection ductility index, (μ_Δ), can be determined. It is based on deflection computation at mid-span of beam. The deflection ductility index (μ_Δ) is the ratio of deflection at ultimate state of beam to the deflection at yielding point of steel.

The deflection ductility index, (μ_Δ), for beams tested in this study experimentally and analytically is presented in Table 3. From the Table, the analytical deflection ductility index ranges from 1.72 to 3.19 while for the experimental deflection ductility index, it ranges from 1.74 to 3.24 which shows considerable agreement. The ductility index for most of the beams is below 3.0 except for beams 10 and 12.

Generally, a high ductility index indicates that a structural member is capable of undergoing large deformations prior to failure. For beams with ductility index in the range of 3 to 5 is considered imperative for adequate ductility especially in the areas of seismic design and redistribution of moments (Ashour 2000; Kumar 2008; Rashid and Mansur 2005 and Iffat et al., 2011). Beams with ductility index only up 1.99 lacked adequate ductility and cannot redistribute moment.

Table 3 Indicates the Theoretical and experimental values of ductility index of beams

S/No	Beam ID	DI (exp)	DI (theo)	Ratio
1	BM 1	1.73	1.72	1.01
2	BM 2	2.26	2.31	0.98
3	BM 3	1.78	2.10	0.85
4	BM 4	2.28	2.32	0.98
5	BM 5	1.87	1.85	1.01
6	BM 6	2.72	2.73	0.99
7	BM 7	1.82	1.91	0.95
8	BM 8	2.72	2.85	0.95
9	BM 9	1.98	1.92	1.03
10	BM 10	3.16	3.00	1.05
11	BM 11	1.86	1.76	1.06
12	BM 12	3.24	3.19	1.02
13	BM 13	1.99	2.03	0.98
14	BM 14	2.89	2.59	1.12
15	BM 15	1.93	1.90	1.02
16	BM 16	2.78	2.74	1.01
17	BM 17	1.83	1.85	0.98
18	BM 18	2.38	2.35	1.01

6.0 Conclusions

In this study, an experimental and analytical evaluation of the deflection ductility index of concrete beams reinforced with rebars milled from scrap metal is conducted. Based on observed behavior, experimental results and analytical predictions, the following conclusions can be drawn:

(1) Rectangular concrete beams reinforced with rebars milled from scrap metal exhibit linear flexural behavior prior to first crack load and then non-linear up to yielding of steel and failure point.

(2) The mode of failure of rectangular beams reinforced with rebars milled from scrap metal is fairly ductile.

(3) The ductility index of concrete beams reinforced with rebars milled from scrap metal is inadequate cannot redistribute moment and a structural member not capable of resisting large displacement.

(4) The flexural capacity of rectangular beams reinforced with rebars milled from scrap metal increase with increase in steel ratio.

(5) Hognestad models for concrete and steel and theoretical equations for a beam section based on strain compatibility and equilibrium of forces could be used to evaluate the deflection ductility index of a rectangular beam reinforced with rebars milled from scrap metal as well as model its flexural behavior adequately.

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