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# Flexural Behavior of Self-Compacting Concrete Beams with Steel Fiber Reinforcement with B500 Ribbed Rebar

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## Abstract

This research aimed to study the bending behavior of self-compacting concrete (SCC) beams reinforced with steel fiber and B500 ribbed reinforcing bar. The studied variables were four: cracking patterns and failure mode, effect of main reinforcement area, effect of rebar grades, and effect of fiber content. Six rectangular beams of cross section of size 100×200 mm and a length of 2000 mm were casted and tested under four-point bending. One beam was used as a control beam without steel fibers, while the other five beams were casted using steel fibers. The steel fiber content was varied from zero to 25 and 50 kg/m$^3$. Three types of rebar main reinforcement were used B500DWR, B500CWR, and B500C-R. According to the experimental results, the tensile strain at failure load ranged from 64,4.7 and 99.2% of that of control beam B1 and the compressive strain at failure load was 76.2 % and 57.1 % of that of beam B1, the control beam. The failure load for beams were 101.5.3 and 106.7 % of that of the control beam B1. Changing the main reinforcement for beams lead to the maximum deflection for beams ranged from 101.5, 109.9, and 81.3% of that of the control beam.

**KEYWORDS:** *Steel Fiber, Fiber Self Compacting Concrete, Flexural Strength*

## Introduction

Steel fiber self-compacting concrete (SFSCC) is a unique material that can flow under its own weight in the fresh state, eliminating the need for mechanical motion and formwork complexity, and that leverages the benefits of steel fiber additions in the hardened stage [1]. Ozawa et al., describe SCC as concrete with a high flow-ability that provides flow and passage ability as well as resistance to segregation. Steel fiber reinforced concrete (SFRC) is concrete reinforced with steel fibers [2]. There are several varieties of steel fibers (SFs) with varying aspect ratios and characteristics. SFRC proved advantageous for several applications in construction and architecture, such as airport pavements, ground-supported slabs, and mine and tunnel linings [3]. The benefits of employing SFRC include multidirectional reinforcement supply, increased productivity, higher intensity resistance, and reduced corner and edge damage caused by spalling pressures [4]. By bridging gaps, distributing stress across cracks, and inhibiting crack growth, the inclusion of steel fiber in concrete can increase the strength and hardness of concrete [5]. In the past ten years, steel fiber reinforced self-compacting concrete (SFRSCC) has been utilized in a variety of structural applications, including precast pre-stressed concrete members, sheet piles, and slab-on-grade, etc. Since fibers in SCC may be compacted without the requirement for vibration, lowering the possibility of fibers segregation and downward settling during compaction results in the uniform distribution of fibers throughout the concrete component [6].

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In the last few decades, self-compacting concrete (SCC) has been a significant advancement in concrete technology [7:12]. SCC can be described as a special type of flowable concrete that is consolidated due to its own weight without any signs of undesirable segregation or bleeding. Such property can save both time and cost by eliminating the effort required for external vibration. Moreover, it reduces the site noise leading to a better work environment [13:16]. SCC was first developed in 1986 [17]. Numerous nations, including Canada, Japan, and the United States, have investigated the potential of SCC for use in building construction and structural activities [18]. SCC provides several benefits, including high productivity, easy manufacturing, and good structural quality [19].

According to EFNARC [20] and ACI 273R [21], some concrete should pass the required limitations of the tests of three properties to be considered as a successful SCC. Filling capacity, ability to flow through steel reinforcement, and resistance to segregation during casting are desirable qualities of SCC. Several tests were developed by research and adopted by different standards to examine the required properties. Like the ordinary concrete, SCC is relatively a brittle material that shows weak behavior under tensile loads. This brittleness also leads to low dynamic response and impact resistance. Steel and synthetic fibers can be added to SCC to make it more ductile, which enhances the structural behavior both under static and dynamic loads. However, such fibers significantly reduce the flowability of SCC mixtures, which imposes the use of more liquidity and viscosity enhancement agents.

Many previous studies explored the effect of different steel fiber diameters and kinds on the fresh characteristics and mechanical properties of Steel Fiber-Reinforced Self-Compacting Concrete (SFRSCC)[22:28]. In comparison to traditionally vibrated concrete, the addition of fibers to self-compacting concrete (SFSCC) increases its efficacy in the fresh and hardened stages [29]. The addition of SF to concrete mixtures is a non-traditional mass reinforcement that enhances the mechanical properties, ductility, and toughness of concrete and controls the spread of cracks [30]. These effects are caused by the ability of SFs to transmit tensile stress across fracture surfaces, commonly known as crack bridging, and the fact that such fibers create significant shear resistance across existing fissures. The debonding and separation of randomly scattered SFs in concrete is connected with SFRC fracture. As a result, SFSCC has a pseudo-ductile tensile response and greater energy dissipation capacity than standard concrete, which exhibits a brittle behavior[3].

In 2012, S.A. Bhalchandra and Pawase Amit [29], studied the SCC performing with steel fiber of beams as flexural strength. The results showed that there was improvements in mechanical properties of concrete and increased strength capacity due to presence of self-compact additive and steel fiber in the concrete. In 2013, M. Pająk and T. Ponikiewski [30], explored the flexural behavior of SCC with hooked end steel fiber concrete beam. The results indicated that the increased in beam capacity and reduced in deflection. In 2013,Fritih et al. [31] examined the influence of stainless SFs (0.25 percent by volume) on the bending and shear performance of SCC beams with various rebar ratios. They determined that the cracking of RC components in hostile settings might be prevented by incorporating stainless SF in accordance with Euro code 2 requirements.

In 2016, R. Vengadesan et al [32],studied the behavior of SCC-reinforced steel fiber beams. As the fiber content of the tested beams increased, the mode of breakdown switched from brittle to ductile when the beams were exposed to stress and bending. Also, the strength of concrete beams will increase with increase in amount of fiber content.

In 2018, M. Mahmod et al [33], examined the flexural behavior of steel fiber reinforced self-stressed concrete beams. Under monotonic stresses, they examined fourteen reinforced concrete beams: two pairs of six SCCs (with and without SFs) and two ordinary concretes (NCs). Capacity at failing, displacement, fracture pattern, and failure mode were documented. The examined SFSCC beams exhibited a more ductile behavior than the SCC and NC beams, resulting in more impact energy. The fracture width was significantly reduced due to SF confinement. The midspan deviation of all tested beams with SCC reduced as the concrete compressive strength rose. The addition of SF to SCC significantly improved function and flexural capacity.

Siefaldeen Odaa et-al [34], have examined and analyzed the ductility index in terms of absorbed energy in SFRSCC under flexural stress. Under flexural loads, twelve reinforced SCC beams, divided into two groups of six beams, were evaluated (with and without steel fibers). The research included minimum and maximum steel ratios as well as three concrete grades (G20, G50, and G60).The results show that the flexural stiffness of the fiber SCC beam specimens is increased to avoid beam movement and, as a result, cracking. Furthermore, the findings show that the change of fibrous material into SCC is extremely efficient. A material is stronger as its ductility index, energy dissipation, and flexural capacity increase (lE). In addition, the flexural strength increases with an increase in the proportion of steel fibers, concrete compressive strength, and steel reinforcement ratio. In comparison to non-fibrous SCC beams, the ductility index increased by about (2.23%–12.57%), (5.88–38.55%), and (6.62–17.49%) for grade 20, grade 50, and grade 60 SSC beams reinforced with SF.

In 2021, I.G. Shaaban et al., [35] studied the flexural characteristics of lightweight ferrocement beams with various types of core materials and mesh reinforcement. They cast sixteen reinforced concrete beams having the cross-sectional dimensions of 100*200*2000 mm and clear span of 1800 mm. They tested beams until failure under a single mid-span concentrated load.. Ferrocement beams contained either an Autoclaved Aerated lightweight brick Core They found that, ferrocement beams with EMM generally gave higher ductility index than those with WWM. Ferrocement beams were found to show better crack control and less spalling compared to the conventional beams(AAC), Extruded Foam Core (EFC),or a Lightweight Concrete Core (LWC); and were reinforced with either Expanded Metal Mesh (EMM), Welded Wire Mesh (WWM) or Fibre Glass Mesh (FGM).Ductility was found to be highly affected by type of mesh reinforcement

In 2021a study on the behaviour of reinforced concrete beams containing hybrid fibres on flexure. Fifteen beams were cast using three three types of fiber as ( polypropylene, (PP), polyvinyl alcohoor hybrid fibres) and tested under flexure. The fibres were used up to 2.5% in the beams which were reinforced with and without shear reinforcement . All the beams were tested under four point bending with span to depth (a/d) ratio of 2.25. It was found that the PP, PVA fibres, and their hybrid in RC beams showed higher ductility in terms of multiple cracking before failure as compared with control beam without fibres. It was noticed also that PVA fibre showed a relatively greater flexural strength and recovery effect compared to PP fibre. Adding more than 1.5% PVA or hybrid fibres (1.5% PVA and 0.375% PP) without shear reinforcement contributed towards increasing shear capacity and ductility compared to the control beam containing shear reinforcement without fibres. A combination of small amount of hybrid fibres (0.75% PVA and 0.75% PP) and stirrups reinforcement resulted in a higher shear strength and higher ductility compared to other studied beams without shear reinforcement, which contain PVA, PP fibres up to 2.5% or hybrid fibres (1.5% PVA and 0.375% PP).[36]

Beshara ,A.F.B., et al [37] presented a paper in the development of simple semi-empirical formulae for the analysis of nominal flexural strength of high strength steel fiber reinforced concrete (HSFRC) beams. Such developed formulae were based on strain compatibility and equilibrium conditions for fully and partially HSFRC sections in joint with suitable idealized compression and tension stress blocks. The stress blocks were given by suitable empirical functions for the compressive and post-cracking strengths of HSFRC. The enhancement in compressive strength due to fibers inclusion is proposed as a function of concrete matrix strength and fiber reinforcing index.He found that there is a good agreement between the flexural strengths for HSFRC beams predicted by the proposed formulae and the experimental results reported in the literature. while the predicted flexural strengths as computed by ACI Committee 544.4R (ACI Struct J 85:563–580, 1988) [38] and ACI Committee 544.1R (ACI Struct J 94:1–66, 1997)[39] were very conservative. The parametric studies indicate that the nominal moment section capacity increases with the increase of fiber content and fiber aspect ratio.

Joseph P. Rizzuto et al.[40] investigated the effect of self-curing admixture on concrete properties in hot climate Conditions. The following study was cried out by mixing concretes at a temperature of 25 °C and 50 °C by incorporating 1.5% of PEG 400 in the concrete. Trials were also carried out by varying the temperature of mixing water which was varied from 5 °Cto35°C. The results indicated that self-curing concretes out performed those concretes which were subjected to normal curing regime [40].

## 2. Research Significant

Based on the research gap from the above literature review, the current study aims to investigate the flexure behavior of reinforced concrete beams containing various steel fiber content, different type with different areas of main reinforcement. The used main reinforcement was the highest grade of steel rebar included in the Egyptian Standards Specifications ES 262-2/2015. The research focuses on the effect of cracking patterns and failure mode, effect of main reinforcement area, effect of rebar grades, and effect of fiber content. This will be achieved by testing 6 concrete beams containing different percentages of steel fiber and different types of main reinforcement

## 3.Experimental Program

### 3.1Tested Specimens

Six rectangular beams (B1 to B6) of cross section of size 100×200 mm and a length of 2000 mm were casted and tested under four-point bending until failure. One beam (B1) was used as a reference beam without steel fibers, while the other five beams (B2 to B6) were cast using steel fibers. The clear span of the beams was 1750 mm, and the two different loading points were 350 mm apart and proportionally positioned near the beam's mid span. The tested beams had a shear span of 700 mm. Figure 1 depicts the beam dimensions, placement of the loading points, and reinforcing details of the specimen beams, whereas Figure 2 depicts the specimen beams' cross section.

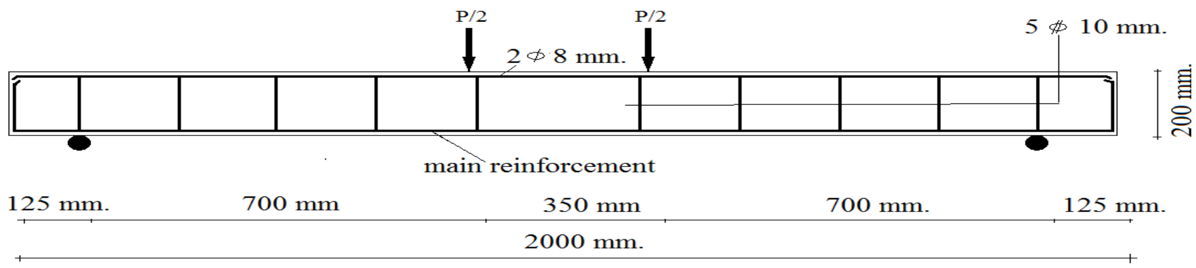
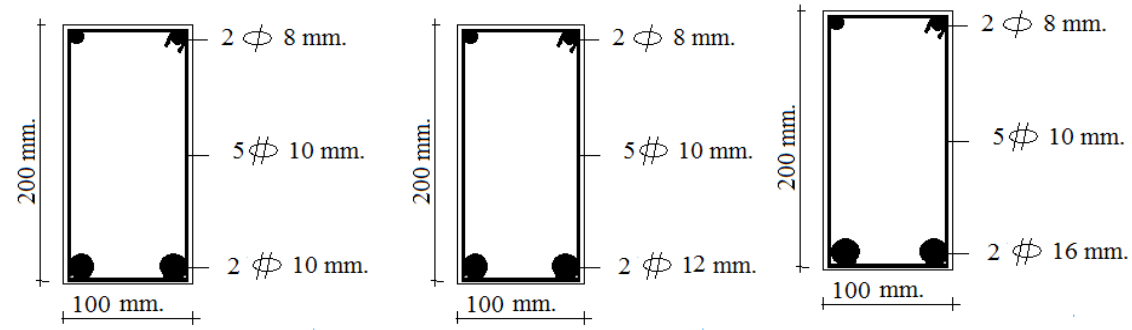



Figure 1: Beam dimensions, and reinforcement details.




Cross section of beams B1 : B4 Cross section of beam B5 Cross section of beam B6

Figure 2: Cross section of the beam specimens

Table 1 shows the details of tested specimens. The main reinforcement for beams B1 to B3 were two B500DWR 12 mm diameter. The main reinforcement for beam B4 were two B500CWR 12 mm. The main reinforcement for beam B5 were two B500DWR 10 mm and for beam B6 were two B500C-R 16 mm.

For all beams, the secondary reinforcement was two bars, eight mm diameter each, mild steel and ten mm diameter B500DWR closed stirrups at 180 mm intervals were used in the shear zone.

Table 1: Details of beam specimens

| Beam code | B1 | B2 | B3 | B4 | B5 | B6 |
|---|---|---|---|---|---|---|
| fiber content kg/m$^3$ | 0 | 25 | 50 | 25 | 25 | 25 |
| Main reinforcement | 2 Ø 12 | | | 2 Ø 12 | 2 Ø 10 | 2 Ø 16 |
| Rebar grades | B500DWR | | | B500CWR | B500DWR | B500C-R |

## 3.2 Materials

The beams were casted with Self-Consolidating Concrete (SCC) of grade 50. Table2 shows concrete mix proportion for specimen. The cement used in the mixes was CEMI-52.5 N has a specific gravity of 3.15 according to the European Standard EN 197-1[41] and produced by Misr Beni Suef Company, Egypt. The fine aggregate was naturally siliceous sand that complied with EN196-1 specifications [42]. The maximum aggregate size of the used coarse aggregate was 19 mm as it was the commercial type available. High range water-reducing, Type F admixtures with dark brown color and density 1.225 kg/L was used. The cementations materials were 475 kg CEM1 52.5 N and 25 kg silica fume. The water to cementations materials ratio was 0.34. Three standard cubes with length of 150 mm were prepared and cured under standard conditions 28 days until tested day.

The used main reinforcement rebar was the highest grade of steel rebar included in the Egyptian Standards Specifications ES 262-2/2015 which is compliant with the International Standards ISO 6935-2/2007, and is accredited in the Egyptian Code for the Design and Construction of Reinforced Concrete Structures ECP 203-2020. It has Highest Ductility and Tensile Strength to Yield Strength ratio ≥1.25 which is the only Ductility Class permitted in the design of structural elements resisting earthquake loads according to the Egyptian Code.

Table 2: Concrete mix proportion for beam specimens

| Cement | Silica fume | Dolomite | Sand | Water | Admixture |
|---|---|---|---|---|---|
| 475 kg | 25 kg | 827 kg | 827 kg | 170 lit. | 13 kg. |

Table 3 illustrates the results of the properties of the concrete mix which was calculated using the Empirical method for concrete mix design with and without steel fiber in fresh and hardened states. The (D) numbers reflect the greatest ultimate diameter of the spread slump flow, whilst the (T50) values indicate the duration needed for concrete flow to reach a 50cm diameter circle. These findings meet the BS-EN-12350-8 requirements for SCC acceptability[43]and show outstanding deformability without blockage. Figure 3 shows the slump flow diameter for SCC mixture while, figure 4 shows the specimens during concrete pouring.

Table 3: Characteristics of the concrete mix in fresh and hardened states.

| Beam Id. | B1 | B2 | B3 | B4 | B5 | B6 |
|---|---|---|---|---|---|---|
| Fiber content kg/m$^3$ | 0 | 25 | 50 | 25 | 25 | 25 |
| T50 sec. | 4 | 3.55 | 3.53 | 3.55 | 3.55 | 3.55 |
| Slump flow final diameter (D) mm. | 740 | 700 | 680 | 700 | 700 | 700 |
| Concrete compressive strength Mpa | 50 | 54 | 54.6 | 54 | 54 | 54 |

Figure 3: The slump flow test.

Figure 4: Beams during concrete pouring.

**3.3 Position of strain gages and LVDT**

To assess the strain variation as a function of loading, two strain gauges, ST1 and ST2 (10 mm Kyowa gage length), were mounted in two distinct locations on each RC beam. The first strain gauge (ST1) was installed at the top of the steel bar, while the second strain gauge (ST2) was positioned at the bottom of the steel bar. The deflections of the tested beams were determined using linear variable displacement transducers (LVDT). The data from the "LVDTs" and electrical straining gages were recorded using a data logger. Figure 5, and Figure 6 show the used strain gauges LVDT.

**3.4 Test procedure**

The four-point loading method was adopted for testing all beams with central section subjected to pure bending without shear effect. The beams were loaded monotonically utilizing a digitally controlled, 500 kN-capacity Shimadzu Universal Testing Machine (UTM).The crack propagation, mode of failure and failure load for each beam were recorded. Figure 6shows the locations of "LVDTs", and figure7shows the test set up.

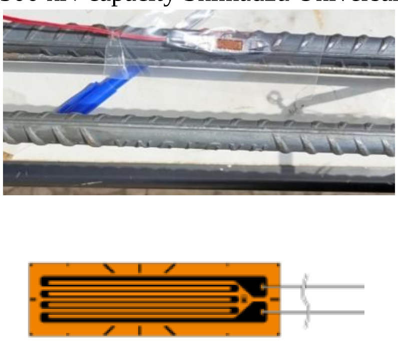


Figure 5:The used strain gauges.

Figure 6:LVDTs location.

Figure 7:Test set up.

# 4.Results and discussion

### 4.1Effect of fiber content

As shown in table3, the steel fiber content for beamsB1, B2, and B3 was 0, 25, 50 kg/m$^3$ respectively. Table 4 summarizes the test results for the beams B1 to B3.

Table 4: Test Results for beams B1:B3.

| Beam code | Fiber content kg/m$^3$ | Failure load | | Maximum deflection | | Tensile strain at failure load | | Compressive strain at failure load | |
|---|---|---|---|---|---|---|---|---|---|
| | | kN | Ratio to Control | mm. | Ratio to Control | mm./ mm. | Ratio to Control | mm./ mm. | Ratio to Control |
| B1 | 0 | 57.56 | 100.0% | 32.3 | 100.0% | 0.0261 | 100.0% | 0.0021 | 100.0% |
| B2 | 25 | 58.44 | 101.5% | 32.6 | 101.2% | 0.0168 | 64.4% | 0.0016 | 76.2% |
| B3 | 50 | 61.43 | 106.7% | 30.0 | 93.1% | 0.0259 | 99.2% | 0.0012 | 57.1% |

Figure 8 depicts the connection between load and maximum deflection for these beams. The results show that, for beams B1:B3 the relation between load and maximum deflection is linear up to initial cracking load after which the relationship is non-linear. The results show that, for all stages of loading and until failure, the recorded deflection for beams B2, and B3, in which the steel fiber was used, is less than that for beams B1 without steel fiber. The maximum deflection for beams B2 and B3 were 101.2% and 93.1 % of that of the control beam B1. On the other hand, Figure 9 depicts the load-tensile strain relationship in the primary steel of beams B1:B3. The relation between load and maximum compressive strains in secondary steel of beams B1:B3 at different stages of loading is drawn in Figure 10. The addition of steel fibres in the SCC has been found to increase the flexural strength for medium and high strength up to 46% and 37.5%, respectively The use of steel fibers affected positively the maximum load. The addition of 30 kg/m$^3$ and 60 kg/m$^3$ of steel fibers caused the maximum loads to increase by 26.6% and 36.6% respectively, with respect to the reference specimen [44].

The use of fibres of different sizes are needed to improve the control of multi-level cracking of reinforced concrete (RC) as it was indicated by Shabaan,I.G., et al [36]. He also noticed that PVA fibre showed a relatively greater flexural strength and recovery effect compared to PP fibre. Adding more than 1.5% PVA or hybrid fibres (1.5% PVA and 0.375% PP) without shear reinforcement contributed towards increasing shear capacity and ductility compared to the control beam containing shear reinforcement without fibres. A combination of small amount of hybrid fibres (0.75% PVA and 0.75% PP) and stirrups reinforcement resulted in a higher shear strength and higher ductility compared to other studied beams without shear reinforcement, which contain PVA, PP fibres up to 2.5% or hybrid fibres (1.5% PVA and 0.375% PP)[36]

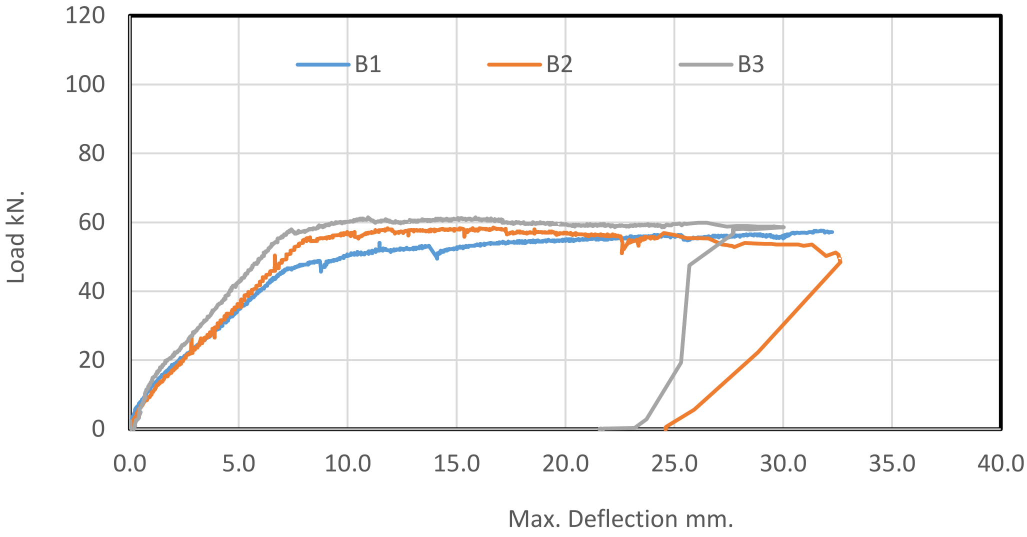

120
100
80
60
40
20
0
Load kN.
B1
B2
B3
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
Max. Deflection mm.

Figure 8: The relation between load and maximum deflection for beams B1: B3

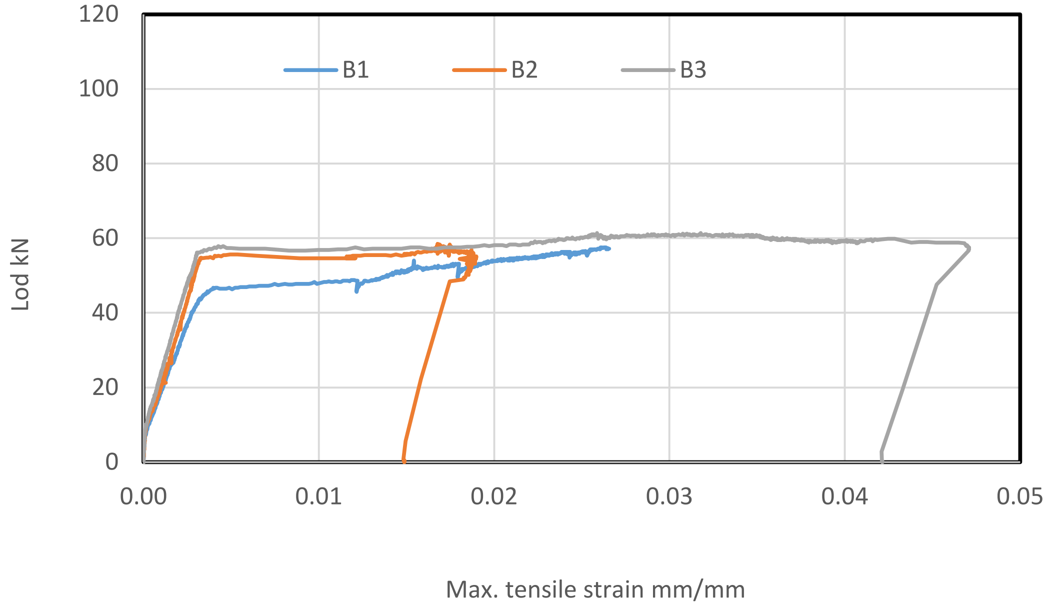



Figure 9: The relation between load and maximum tensile strain for beamsB1: B3

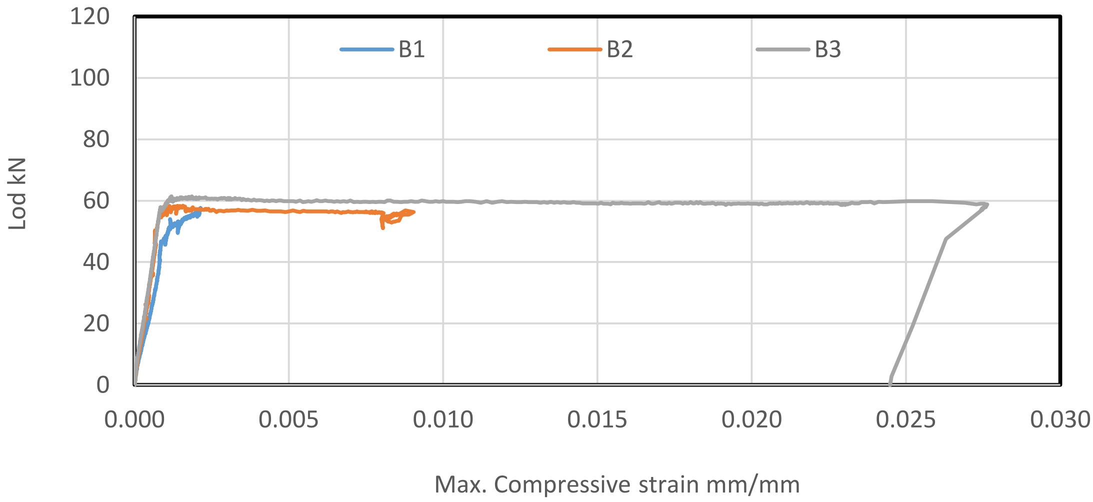



Figure 10: The relation between load and maximum compressive strain for beams B1: B3

It is clear that, the recorded tensile and compressive strains for beams B2 and B3, in which the steel fiber was used, is less than that for beams B1which does not have steel fiber, for all stages of loading and until failure. The tensile strain at failure load for beams B2 and B3 were 64.4.7% and 99.2 % of that of the control beam B1. For beams B2 and B3, at failure load, the compressive strain was 76.2 and 57.1 percent of that of the control beam B1.Beams B2 and B3 had failure loads that were 101.5.3% and 106.7% of the control beam B1.

**4.2 Effect of Rebar Grades**

In this research, two rebar grades B500D and B500C were used for beams B2 and B4 respectively. Table 5 summarizes test results for the beams B2 and B4.

Table 5: Test Results for beams B2 and B4

| Beam code | Rebar grades | Failure load | | Maximum deflection | | Tensile strain at failure load | | Compressive strain at failure load | |
|---|---|---|---|---|---|---|---|---|---|
| | | kN | Ratio to Control | mm. | Ratio to Control | mm./ mm. | Ratio to Control | mm./ mm. | Ratio to Control |
| B2 | B500D | 58.44 | 101.5% | 32.6 | 101.2% | 0.0168 | 64.4% | 0.0016 | 76.2% |
| B4 | B500C | 59.14 | 102.7% | 37.1 | 114.9% | 0.0127 | 48.7% | 0.0019 | 90.5% |

Figure 11depicts the relationship between beam B2 and B4 load and maximum deflection. The results show that, the recorded deflection for beams B2 and B4 is almost the same. depicts the relationship between beam B2 and B4's load and tensile strain Figure 12. Figure 13 depicts the relationship between compressive strain and load for beams B2 and B4. The results show that, the recorded tensile or compressive strains for beams B2, and B4 is very close to each other. The failure load of beams B2 and B4 was 101.5 and 102.7 percent of the control beam B1, respectively. The greatest deflection of beams B2 and B4 was 101.2 and 114.9% of that of beam B1, which served as the control beam. The tensile strain at failure load for beams B2 and B4were64.4 % and48.7 % of that of the control beam B1. The compressive strain at failure load for beams B2 and B4 were76.2 % to 90.5 % of that of the control beam B1.It can be concluded that, the Rebar grades has no effect on structure behavior of beams B2 and B4.

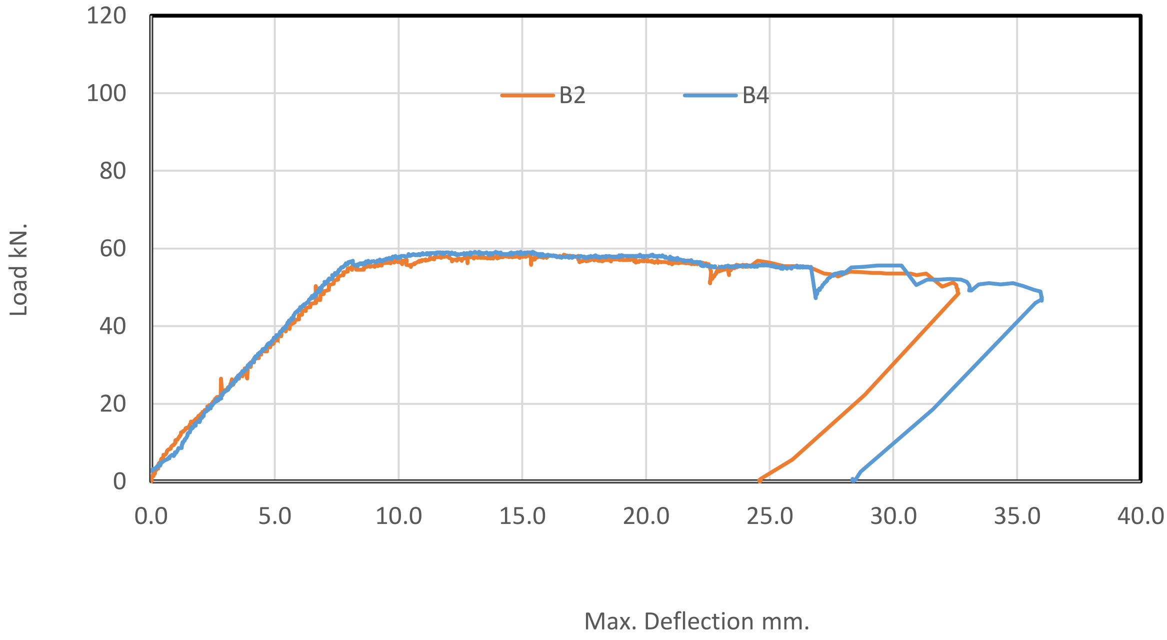



Figure 11: The relation between load and maximum deflection for beams B1: B3

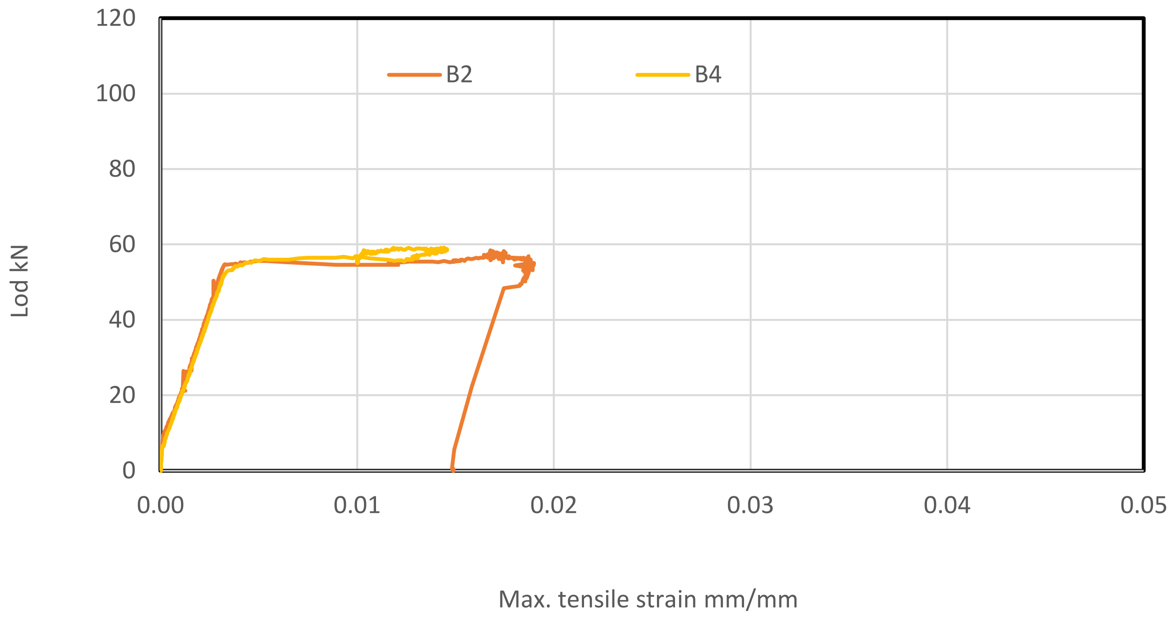



Figure 12: The relation between load and maximum tensile strain for beamsB1: B3

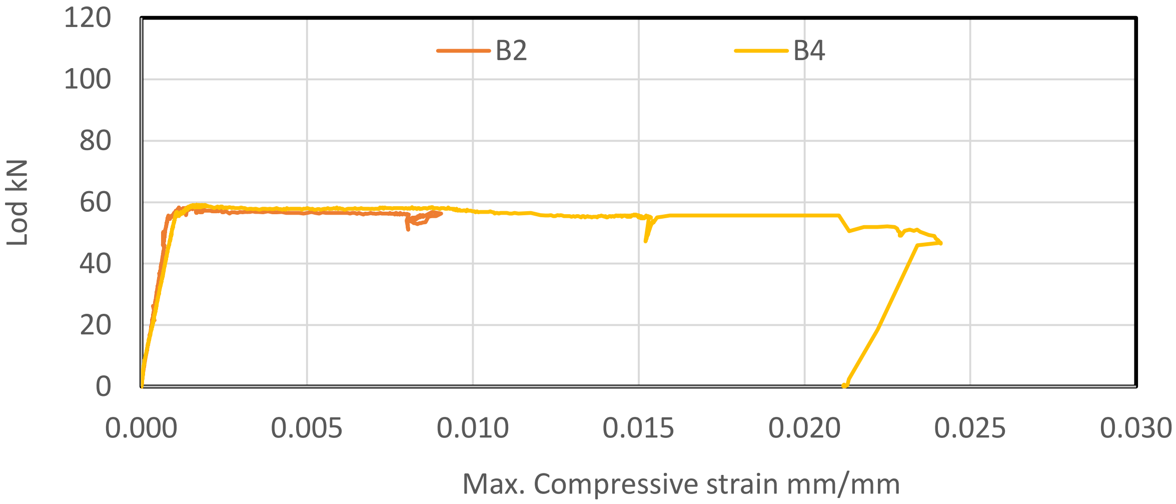



Figure 13: The relation between load and maximum compressive strain for beams B1:B3.

**4.3Effect of Main Reinforcement Area**

The main reinforcement for beams B2, B5 and B6 was 2 ∅ 12, 2 ∅ 10 and 2 ∅ 16, respectively. Table 6 summarizes the test results for the beams B2, B5 and B6. The failure load for beams B2, B5 and B6 ranged from 75.3% to 164.9% of that of control beam B1. Beam B6 exhibited the maximum load carrying capacity which is about 164.9 % of the control beam capacity.

Table 6: Test Results for beams B2, B5, and B6.

| Beam code | Main Reinforcement % | Failure load | | Maximum deflection | | Tensile strain at failure load | | Compressive strain at failure load | |
|---|---|---|---|---|---|---|---|---|---|
| | | kN | Ratio to Control | mm. | Ratio to Control | mm./ mm. | Ratio to Control | mm./ mm. | Ratio to Control |
| B2 | 1.131 | 58.44 | 101.5% | 32.6 | 101.2% | 0.0168 | 64.4% | 0.0016 | 76.2% |
| B5 | 0.786 | 43.35 | 75.3% | 35.5 | 109.9% | 0.0363 | 139.1% | 0.0009 | 42.9% |
| B6 | 2.01 | 94.95 | 164.9% | 26.2 | 81.3% | 0.0178 | 68.2% | 0.0012 | 57.1% |

The relationship between load and maximum deflection for beams B2, B5, and B6 is depicted in Figure 14.The results show that: the recorded deflection for beams B2, and B6, is less than that for beam B5, for all stages of loading and until failure. The recorded deflection for beams B6, is less than that for beam and B2, and B5, for all stages of loading and until failure. The maximum deflection for beams B2, B5 and B6 was 101.5%, 109.9%, and 81.3% of that of the control beam B1, respectively.

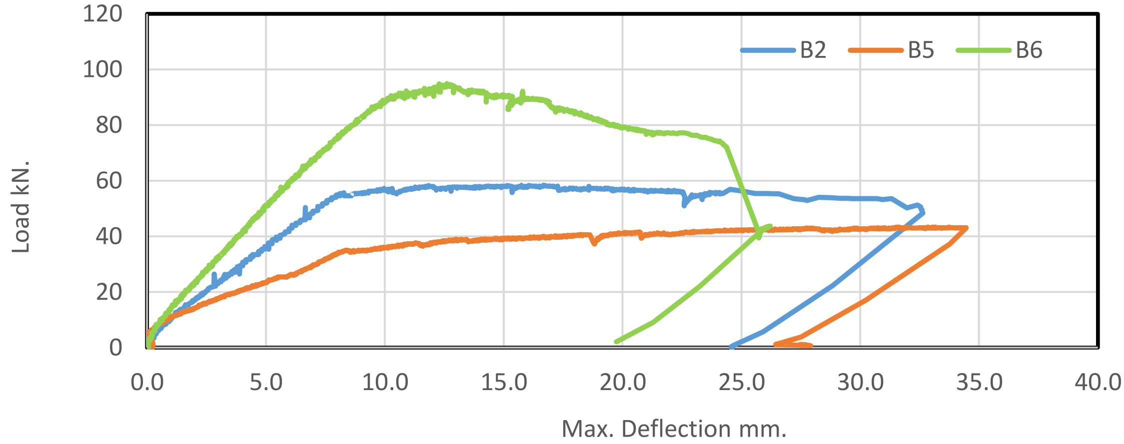



Figure 14: The relation between load and maximum deflection for beamsB2,B5and B6

The correlation between load and tensile strain for beams B2, B5, and B6 is illustrated in Figure 15. The results show that: the recorded tensile strain for beams B2, and B6, is less than that for beam B5, for all stages of loading and until failure. The recorded tensile strain for beams B6 was less than that for beam B2 and B5, for all stages of loading and until failure. The tensile strain at failure load for beams B2, B5 and B6 ranged from 64.4% to 139.1% of that of the control beam B1. Figure 16 shows the relation between load and compressive strain for beams B2, B5and B6. The compressive strain at failure load for beams B2, B5 and B6 ranged from 42.9% to 76.2% of that of the control beam B1.

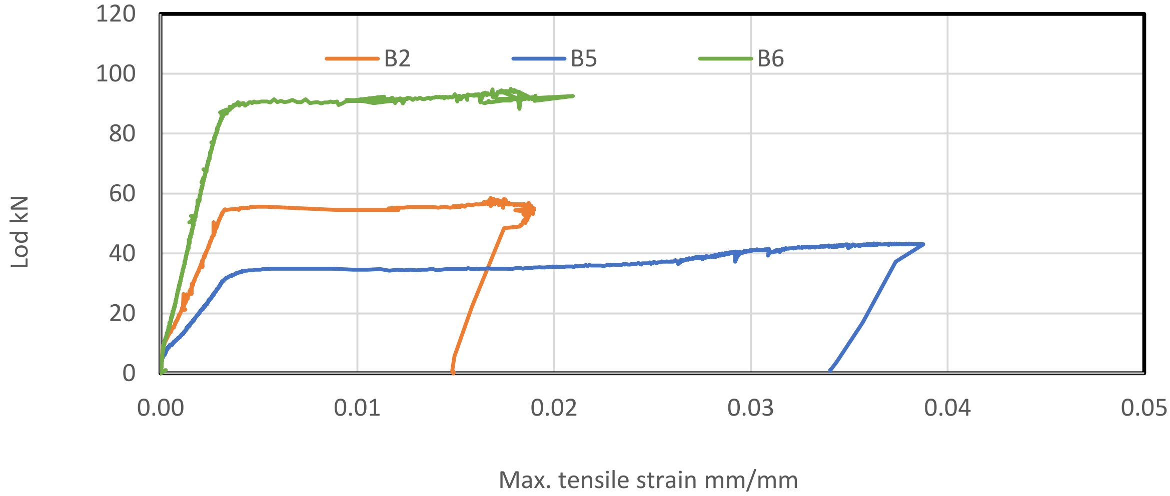



Figure 15: The relation between load and maximum tensile strain for beamsB2, B5,and B6.

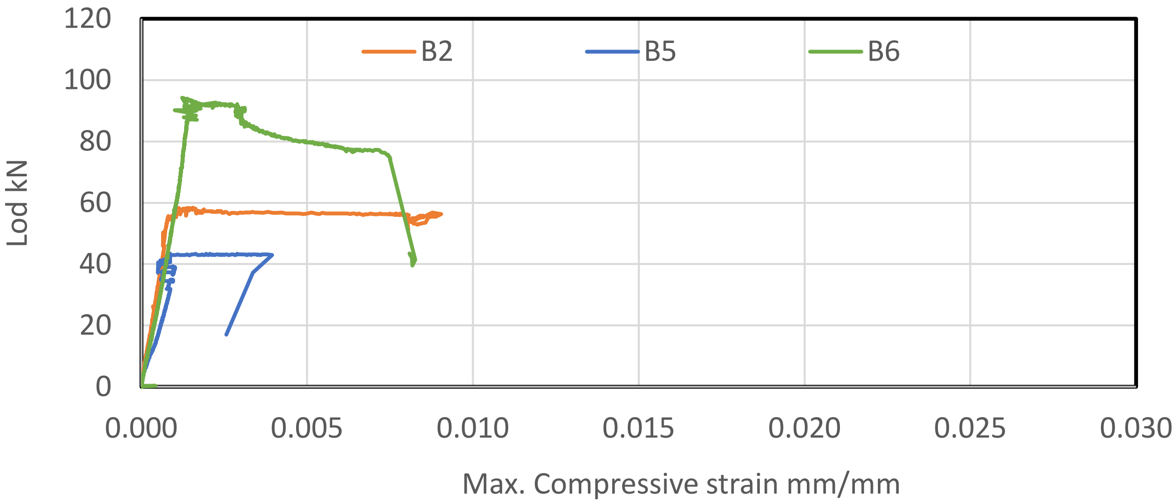



Figure 16: The relation between load and maximum compressive strain for beams B2, B5 and B6.

**4.4 Cracking Patterns and Failure Mode**

Figure 17 shows the cracking pattern recorded at the ultimate load for beams B1 (reference) and beams B2 to B6 ( with steel fibers). Some remarks were observed as follow: the cracking pattern of beams B2 to B5 is more or less the same as beams B1. The failure mode of these beams was flexure tension. The number of cracks for beam B5 is less than the number of cracks of beam B1 this may be due to the effect of main reinforcement area. For beam B6, in addition to the flexure cracks during test, the compression zone failed at ultimate load, the mode of failure of beam B6 was flexure compression. According to Maher [45] in all specimens, the first crack was flexural in the maximal moment zone . The crack spacing was lighter while the crack network was denser in the beams with SFs than in the SCC beams. Crack propagation was delayed in the fiber-reinforced beams. This phenomenon can be explained bratioy the capability of the fibers to transfer stresses to the concrete through a crack. Crack distribution was slightly more regular in the fiber-reinforced beams than in the SCC beams. Thus, the contribution of the concrete area between two existing flexural cracks to the tensile strength (i.e., concrete tension stiffening) was enhanced [45]. The first crack loads recorded ranged from approximately 2.65–4.58% for all beams. The ultimate and cracking load carrying capacities of the SCC (with SFs) beams were greater than those of the SCC beams. This result can be attributed to the capability of the SFs to confine crack growth [46].

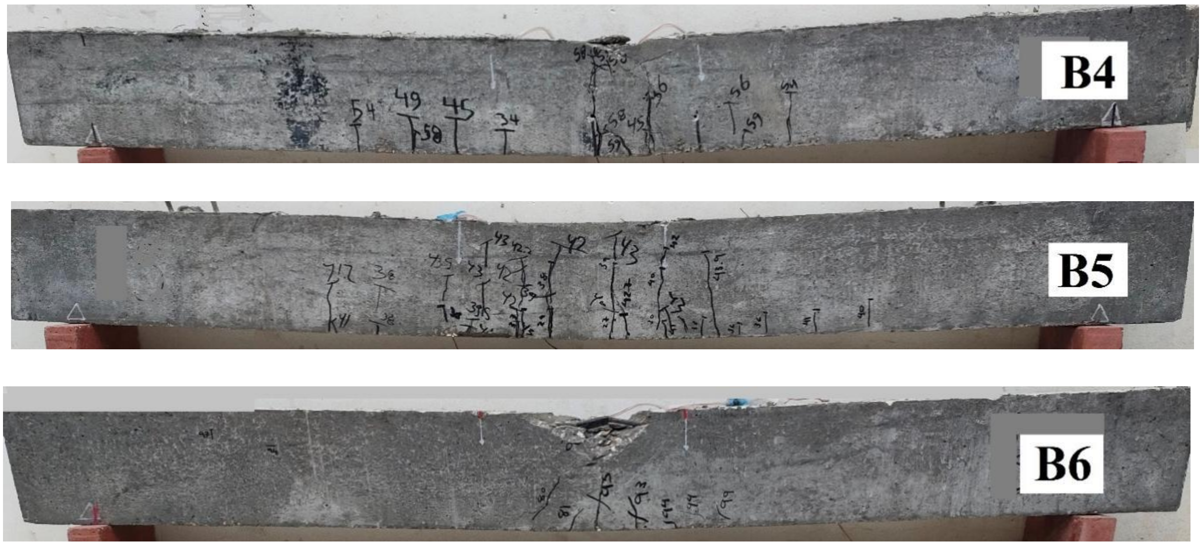



Figure 17: Cracking patterns and modes of failure at ultimate load for tested beams

**4.5 Structural Ductility**

Azizinamini et al. [47] adopted a displacement ductility ratio as a measure to assess the ductility of lap-spliced RC beam specimens. The ratio of the largest midspan to the initial yield displacement of beams (equation (1)) is used as this indication. The initial yield displacement, y, correlates to the junction of the load displacement curve's tangents at the source and the maximum displacement, max (figure 18-a). In addition to the strength requirement, the displacement–ductility ratio provides a new criterion for forecasting the behavior of lap-spliced reinforced concrete beams.

Displacement ductility ratio$\mu_{\Delta} = \Delta_u / \Delta_y$ (1)

Cohn and Bartlett [48] proposed a much more acceptable concept for a displacement ductility index.Thus, the displacement ductility index can be calculated as the ratio between the displacement corresponding to 85% of the maximum load on the post-peak region to the displacement of the first return of the beam(eq. (2) and figure. 18-b).

Displacement ductility index$\mu_{\Delta} = \Delta_{0.85Pmaximum} / \Delta_y$ (2)

Energy ductility, on the other hand, may be determined as the ratio of the area under the load-deflection diagram at ultimate load to the region under the load-deflection graph up to yielding of tension steel (elastic energy) eq. (3).

Energy ductility ,$\mu_E = E_u / E_y$ (3)

Where

$\Delta_{max}$ = midspan deflection at ultimate load;

$\Delta_{0.85Pmaximum}$ = displacement equivalent to 85% of the maximum load in the region of the curve after the peak;

Eu = the area beneath the load–deflection diagram at maximum load;

$\Delta y$ = midspan deflection at yielding of tension steel;

$E_y$ = area under the load–deflection diagram of tension steel till yielding (elastic energy)

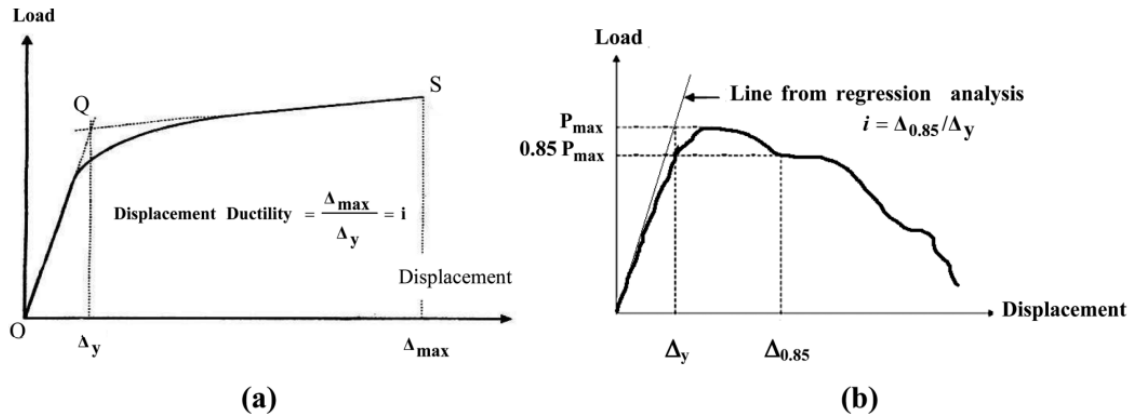



Figure 18 definition of displacement-ductility ratio.

Table (7) shows the Displacement ductility ratio, Displacement ductility index and Energy ductility While, table (8) shows Ductility ratios for tested beams.

Table 7: Ductility indices for tested beams.

| Beam | B1 | B2 | B3 | B4 | B5 | B6 |
|---|---|---|---|---|---|---|
| Failure load (kN) | 57.56 | 58.44 | 61.43 | 59.14 | 43.35 | 94.95 |
| Δmax | 32.26 | 32.64 | 30.03 | 37.06 | 35.45 | 26.22 |
| Δy | 8.3 | 8.0 | 7.9 | 8.1 | 9.7 | 9.2 |
| Displacement ductility ratio | 3.89 | 4.08 | 3.80 | 4.58 | 3.65 | 2.85 |
| Δ0.85Pmaximum | 32.26 | 32.56 | 26.5 | 35 | 33.8 | 19.3 |
| Δy | 8.3 | 8.0 | 7.9 | 8.1 | 9.7 | 9.2 |
| Displacement ductility index | 3.89 | 4.07 | 3.35 | 4.32 | 3.48 | 2.10 |
| Energy ductility | 6.87 | 7.30 | 6.04 | 7.5 | 5.5 | 4.4 |

Table 8: Ductility ratios.

| Beam | B1 | B2 | B3 | B4 | B5 | B6 |
|---|---|---|---|---|---|---|
| Failure load (ratio to control) | 1.000 | 1.015 | 1.067 | 1.027 | 0.753 | 1.649 |
| Displacement ductility ratio | 1.000 | 1.049 | 0.977 | 1.177 | 0.938 | 0.733 |
| Displacement ductility index | 1.000 | 1.046 | 0.861 | 1.111 | 0.895 | 0.540 |
| Energy ductility | 1.000 | 1.063 | 0.879 | 1.092 | 0.801 | 0.640 |

Referring to tables 7 and 8, it could be concluded that:-

1 –The displacement ductility ratio for beams B2: B6 ranged from 73.3% to 117.7% as that of control beam.

2 –The displacement ductility index for beams B2: B6 ranged from 54.0 % to 111.1 % as that of control beam.

3 –The energy ductility for beams B2: B6 ranged from 64.0 % to 109.2 % as that of control beam.

**5.CONCLUSIONS**

Based on the previous results, it can be concluded that:

1-The tensile strain at failure load for beams B2 and B3 was 64,4.% and 99.2% of that of beam B1, which served as the control. The compressive strain at failure load for beams B2 and B3 was 76.2 and 57.1 % of that of beam B1, the control beam. The failure load for beams B2 and B3 were 101.5.3 and 106.7 % of that of the control beam B1.

2- The rebar grades has no effect on structure behavior of beams B2 and B4.

3- Changing the main reinforcement for beams B2, B5 and B6 was 2 ∅ 12, 2 ∅ 10 and 2 ∅ 16, respectively. The recorded deflection for beams B2, and B6, is less than that for beam B5, for all stages of loading and until failure. The recorded deflection for beams B6, is less than that for beam and B2, and B5, for all stages of loading and until failure. The maximum deflection for beams B2, B5 and B6 was 101.5,109.9, and 81.3% of that of the control beam B1, respectively.

4- The number of cracks for beam B5 is less than the number of cracks of beam B1 this may be due to the effect of main reinforcement area.

5- For beam B6, in addition to the flexure cracks during test, the compression zone failed at ultimate load, the mode of failure of Beam B6 was flexure compression.

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