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BEHAVIOUR OF FIBER-REINFORCED BEAM-COLUMN CONNECTIONS SUBJECTED TO CYCLING LOADING

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ملخص البحث

تعرف وصلة عمود والكمرة على أنها أحد المكونات الضعيفة للأطواق المقاومة للعزم الخرساني المسلح الخاضع للتحميل الجانبي. إن المعرفة الراسخة بسلوك القص المشترك للخرسانة المسلحة (RC) أمر ضروري لأن الضرر الشديد داخل وصلة العمود والكمرة قد يؤدي إلى تدهور الأداء الكلي لوصلات الاطارات أو وصلات العمود والكمرة RC. إن سلوك وصلة العمود والكمرة في لحظة مقاومة هياكل الإطار يكون عرضة للتلف الناجم عن التأثيرات الزلزالية بسبب الأداء الضعيف للوصلات. تم إجراء عدد كبير من الأبحاث لفهم الآلية المعقدة لاتصال الوصلة الخرسانية التي يتم النظر فيها في رموز التصميم الزلزالي الحالية. أسفر التفصيل التقليدي للبناء عن التعزيز العرضي عن فشل كبير في الاتصال أثناء الزلازل. وقد أظهرت هذه الاختبارات التجريبية أن استخدام الألياف في الاتصالات هو طريقة فعالة لتحسين سلوك الاتصال وقدرة امتصاص الطاقة بالإضافة إلى تحسين تحمل الضرر للوصلات وتقليل عدد الركائب في التوصيلات الزلزالية. في هذه الدراسة ، شُيدت إحدى عشر عينة من عوارض العمود الداخلي بنصف المقياس مع العديد من تفاصيل التعزيزات الإضافية والتشكيلات. تم تقسيم الاختبارات التجريبية إلى ثلاث مجموعات. المجموعة الأولى التي تتكون من أربع عينات ذات قوة خرسانية مختلفة ، درست تأثير الألياف الفولاذية مع قوة خرسانة مختلفة على وصلة عمود الشعاع تحت الحمل الدوري. تكونت المجموعة الثانية من ثلاث عينات ودرست تأثير الألياف الفولاذية مع تقوية عرضية مختلفة على وصلة عمود الشعاع تحت الحمل الدوري. تكونت المجموعة الثالثة من خمس عينات ودرست تأثير الألياف الحديدية مع حجم متغير على وصلة عمود الشعاع تحت الحمل الترددي.

ABSTRACT *:* Beam-column joint has been identified as potentially one of the weaker components of reinforced concrete moment resisting frames subjected to seismic lateral loading. Well-established knowledge of reinforcement concrete (RC) joint shear behavior is necessary because severe damage within a joint panel may trigger deterioration of the overall performance of RC beam-column connections or frames.

The behavior of beam-column connection in moment resisting frame structures is susceptible to damage caused by seismic effects due to poor performance of the connections. A good number of researches were carried out to understand the complex mechanism of RC connection considered in current seismic design codes. The traditional construction detailing of transverse reinforcement has resulted in serious connection failures during earthquakes.

These experimental tests have shown that using fiber in connections is an effective method for improving connection behavior and energy absorption capacity as well as enhancing the damage tolerance of connections and reducing the number of stirrups in seismic connections. In this study, eight half- scale interior beam-column specimens were constructed with various additional reinforcement details and configurations. The experimental program showed promising results regarding beam column connections subjected to earth quake loads.

KEYWORDS - *R.C joints, energy disruption, ductility, stiffness, time history.*

I. INTRODUCTION

The use of Steel Fiber Reinforced Concrete (SFRC), is enlarging itself to numerous domains of construction. Steel Fiber Reinforced Concrete has very potential application in building frames due to its high seismic energy absorption capability and relatively simple construction technique. To explore such potential, the existing body of knowledge on SFRC must be expanded to cover for enhancing the flexural strength of concrete. A lot of research has been done on improving the concrete strength. There was need to see the improvement in strength with addition of Fibers in high-rise building in addition to rebar. This search aims to add to that body of knowledge through experimental investigation especially with respect to earthquake scenario. Also hundreds of thousands of successful reinforced cement concrete (RCC) framed structures are annually constructed worldwide, there are large numbers of them that deteriorate, or become unsafe due to changes in loading, changes in use, or changes in configuration. Occurrence of natural calamities may also lead to review of engineering notions that make reworking of existing structures inevitable. The parameters studied in these tests were the column transverse reinforcement ratio and steel fiber volume. Eight beam column joint specimens were cast, one of which using convehioal reinforcement concrete (R-Z), and seven specimens using Steel Fiber reinforcement concrete (SFRC).

II. EXPERIMENTAL PROGRAM

In all specimens, the beam had a rectangular cross section of 120 mm×300 mm dimension, whereas the column was of rectangular cross section with dimension 120 mm×300 mm. The longitudinal reinforcement of each beam included 12 T12 mm deformed bars of Grade 360 steel arranged as six bars as top reinforcement and the other six bars as bottom reinforcement. The longitudinal reinforcement of each column consisted of 8T10 mm deformed bars of Grade 360 steel (1.74 % reinforcement), with four corner bars. Hooked steel fiber reinforcement is used with deferent volume and with aspect ratio (L/d) 50. The first three seismic beam-column joints (Group 1) reinforced with steel fiber in the joint region with variable column transverse reinforcement ratio, were analysised to find the contribution of steel fibers to the joint shear resistance capacity. The second five beam column joints (Group 2) with seismic detail were designed with concrete strength 25 Mpa, with the same column transverse reinforcement ratio and with different steel fiber volume. Figure (1) shows details of Reinforcement.

Table 1: Details of group1 specimens.

spec	Fiber (v)	column (b*t)	Col RFT	Beam (b*t)	beam RFT T&B	Fcu N/mm2	Joint stirrups	Trans RFT ratio
1A	1 %	12*30	8T10	12*30	6T12	45	0	0.00 %
1B	1 %	12*30	8T10	12*30	6T12	45	8@125	0.56 %
1C	1 %	12*30	8T10	12*30	6T12	45	8@80	0.85 %

Table 2: Details of group2 specimens.

spec	Fiber (v)	column (b*t)	Col RFT	Beam (b*t)	beam RFT T&B	Fcu N/mm2	Joint stirrups	Trans RFT ratio
2A	0.0%	12*30	8T10	12*30	6T12	45	8@125	0.56 %
2B	0.5 %	12*30	8T10	12*30	6T12	45	8@125	0.56 %
2C	1.0 %	12*30	8T10	12*30	6T12	45	8@125	0.56 %
2D	1.5 %	12*30	8T10	12*30	6T12	45	8@125	0.56 %
2E	2.0 %	12*30	8T10	12*30	6T12	45	8@125	0.56 %

Three different parameters were measured during loading, and the values were recorded for the corresponding loading values. The three parameters were the deflections, the strains in the steel reinforcement and the transverse reinforcement and beam end load. Linear Variable Displacement Transducers (LVDT) with an accuracy of 1/100-mm were used for measuring deflection, electrical strain gauges (type FLA-6-11-1L) were used for measuring steel strains. Figures (2) shows the Experimental set up. Displacement and strain reading were recorded automatically during the test, using data acquisition system and a “lap view” software at every load increment.

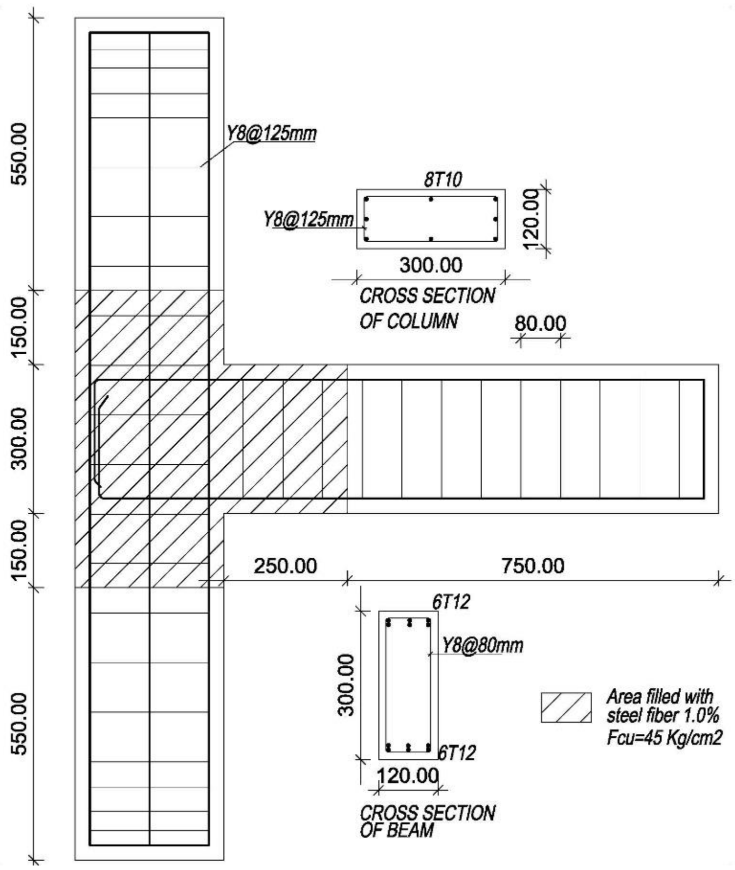

Figure 20: Details of Reinforcement

Figure 21: Full setup for specimen before testing

III EXPERIMENTAL RESULTS

The experimental tests were divided into two groups. The first group which consisted of three specimens with different transverse reinforcement ratio. The second group consisted of five specimens and studied the effect of the steel fiber with variable volume.

Group 1: For specimen (1A) the peak load in the positive direction was obtained as 104.34 KN and that in the negative direction was -104.08 KN. The positive peak load was obtained during 20.10 mm cycle and that in the negative direction was 20.40 mm. The first crack was observed in the joint at 6.0mm cycle. The peak load in the positive direction for specimen (1B) was obtained as 121.6 KN and that in the negative direction was -121.33 KN. The positive peak load was obtained during 24.7 mm cycle and that in the negative direction was 25.2 mm and the first crack was observed in the joint at 10.0mm cycle. For specimen (1C) the peak load in the positive direction was obtained as 130.13 KN and that in the negative direction was -130.82 KN. The positive peak load was obtained during 28.05 mm cycle and that in the negative direction was 28.40 mm. The first crack was observed in the joint at 12.0mm cycle. Figure (3) shows Backbone load deflection curve. Figure (4) and (5) shown core damage for specimens

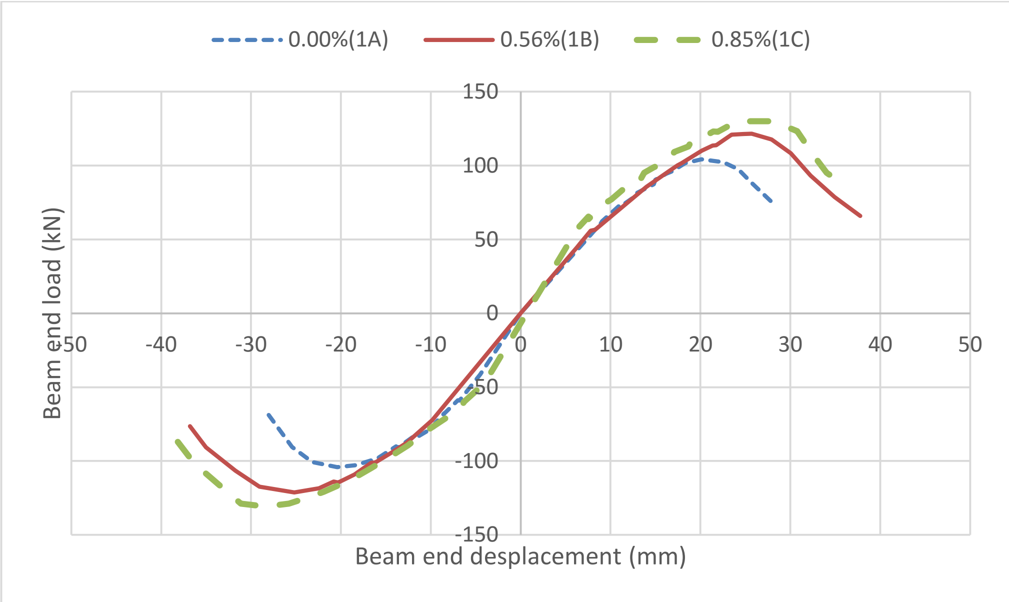

Figure 3: Backbone load deflection curve for Group (1)

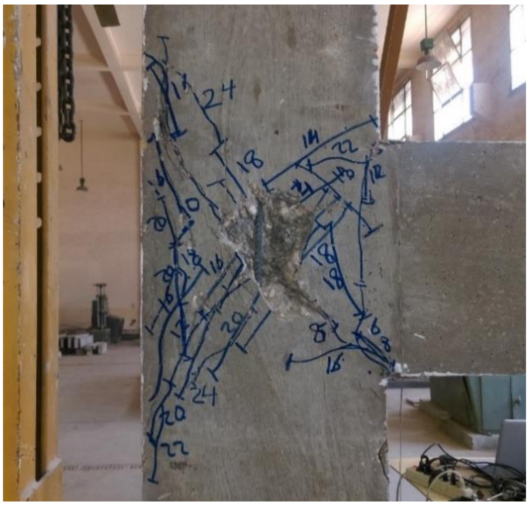

Figure 4: Core damage for specimen (1A) **Figure 5: Core damage for specimen (1B)**

Group 2: For specimen (2A). the peak load in the positive direction was obtained as 112.1 KN and that in the negative direction was -111.90 KN. The positive peak load was obtained during 21.10 mm cycle and that in the negative direction was 20.80 mm. The first crack was observed in the joint at 8.0mm cycle. For specimen (2B) the peak load in the positive direction was obtained as 115.10 KN and that in the negative direction was -114.98 KN. The positive peak load was obtained during 21.60 mm cycle and that in the negative direction was 21.70 mm. The first crack was observed in the joint at 8.0mm cycle. The peak load in the positive direction for specimen (2C) was obtained as 121.6 KN and that in the negative direction was -121.33 KN. The positive peak load was obtained during 24.7 mm cycle and that in the negative direction was 25.2 mm and the first crack was observed in the joint at 10.0mm cycle. The peak load in the positive direction for specimen (2D) was obtained as 113.70 KN and that in the negative direction was -107.14 KN. The positive peak load was obtained during 17.60

mm cycle and that in the negative direction was 24.25 mm. and the first crack was observed in the joint at 10.0mm cycle. For specimen (2E) the peak load in the positive direction was obtained as 117.60 KN and that in the negative direction was -118.07 KN. The positive peak load was obtained during 25.25 mm cycle and that in the negative direction was 27.85 mm. The first crack was observed in the joint at 12.0mm cycle. Figure (6) shows Backbone load deflection curve.

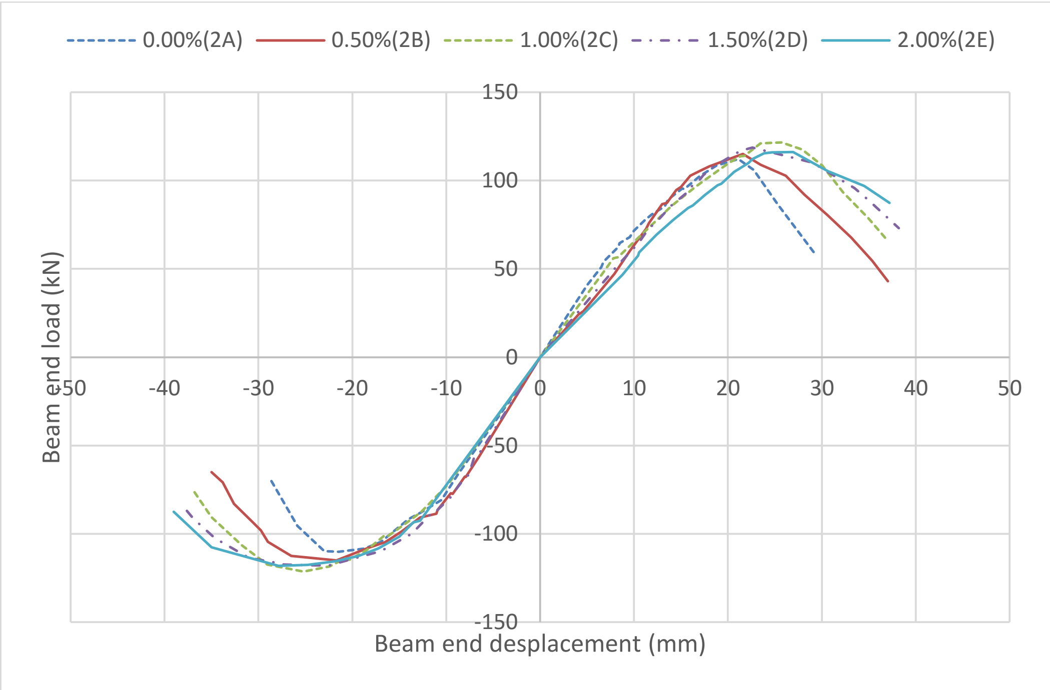

Figure 6: Backbone load deflection curve for Group (2)

ANALYSIS OF THE EXPERIMENTAL RESULTS

GROUP 1: Table (3) shows that the increase of transverse reinforcement on beam column joint induces an increase in load corresponding to the initial diagonal cracking and increase of ultimate shear strength. The average beam end load of joints (1B and 1C) was 16.56 % and 25.21% higher than 1A respectively.

Table 3: Yield and Ultimate Load of the Experimental Tests Group (1)

specimens	ρs	yield load			ultimate load		
		downward	upward	Average	downward	upward	Average
1A	0.00%	83.47	83.26	83.37	104.34	104.08	104.21
1B	0.56%	97.28	97.06	97.17	121.60	121.33	121.47
1C	0.85%	104.10	104.66	104.38	130.13	130.82	130.48

Figures (7) shows the effect of transverse reinforcement of beam column joint on ultimate joint shear strength. The joint shear strength was enhanced due to higher transverse reinforcement of beam column joint. The joint shear strength of specimen (2-a) is 0.99 N/mm2, the joint shear strength of specimen (1B) is 1.35 N/mm2 and the joint shear strength of specimen (2-b) is 1.52 N/mm2. Then the joint shear strength of joints (1B and 1C) was 28.57% and 44.76% higher than 1A respectively.

Figure 7: Effect of transverse reinforcement ratio on ultimate joint shear strength

Figure (8) present the displacement ductility factors for the test specimen of group (1). For specimen (1A) the displacement ductility in positive direction equals 1.58% and negative direction 1.61%. For specimen (1B) the displacement ductility in positive direction equals 1.61% and negative direction 1.64%. For specimen (1C) the displacement ductility in positive direction equals 1.64% and negative direction 1.69%. So the average displacement ductility of specimens (1B) and (1C) is 1.90 % and 5.00% were higher than (1A) respectively. These values indicate that the higher transverse reinforcement on beam column joint was determined to displacement ductility.

Figure 8: Average Displacement ductility for Group (1)

GROUP 2: Table (4) shows that the increase of steel fiber volume from 0.00% to 1.00% induces an increase in load corresponding to the initial diagonal cracking and increase of ultimate shear strength and the increase of steel fiber volume from 1.00% to 2.00% induces an increase in load corresponding to the initial diagonal cracking and decrease of ultimate shear strength. The average beam end load of joints with steel fiber volume (0.50%, 1.00%, 1.50% and 2.00%) was 2.71%, 8.46%, 5.62% and 5.21% higher than specimen (2A) (0.00% steel fiber volume) respectively.

Table 4: Yield and Ultimate Load of the Experimental Tests Group (2)

specimens	fiber volume	yield load			ultimate load		
		down	up	Average	down	up	Average
2A	0.00	89.68	89.52	89.60	112.10	111.90	112.00
2B	0.50	92.08	91.98	92.03	115.10	114.98	115.04
2C	1.00	97.28	97.06	97.17	121.60	121.33	121.47
2D	1.50	94.91	94.35	94.63	118.64	117.94	118.29
2C	2.00	94.08	94.46	94.27	117.60	118.07	117.84

Figure (9) shows the effect of different steel fiber volume on ultimate joint shear strength. The joint shear strength was enhanced due to higher steel fiber volume tile 1.00%. Then the joint shear strength reduces due to higher steel fiber volume. The joint shear strength of specimen (2A) is 1.19 N/mm2, the joint shear strength of specimen (2B) is 1.22 N/mm2, the joint shear strength of specimen (2C) is 1.35 N/mm2, the joint shear strength of specimen (2D) is 1.30 N/mm2, and the joint shear strength of specimen (2E) is 1.27 N/mm2. Then the joint shear strength of joints (2B, 2C, 2D and 2E) was 2.52%, 13.45%, 9.24% and 6.72% higher than specimen (2A) respectively.

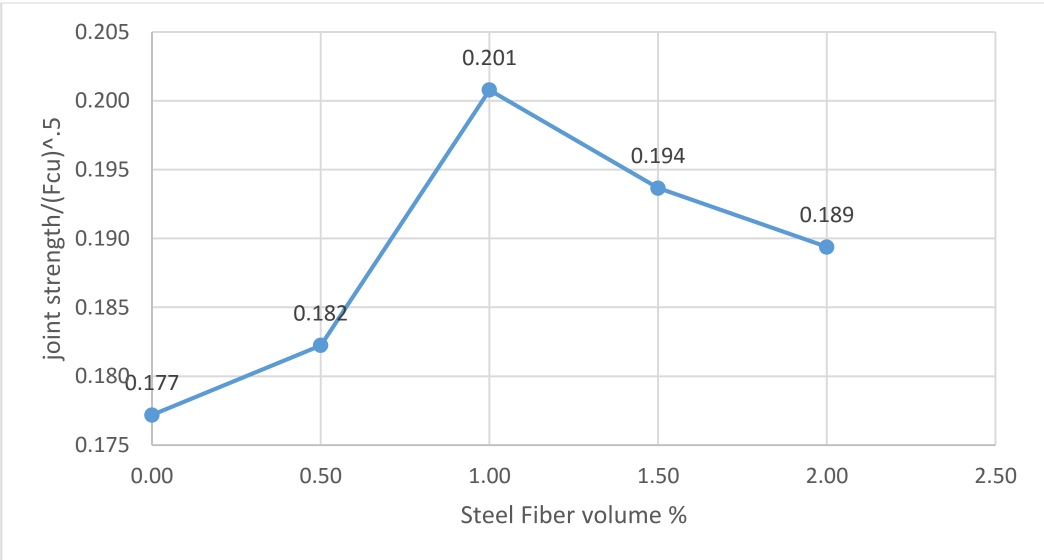

Figure 9: Effect of steel fiber volume on ultimate joint shear strength

Figure (10) present the displacement ductility factors for the test specimen of group (2). For specimen (2A) the displacement ductility in positive direction equals 1.40% and negative direction 1.55%. For specimen (2B) the displacement ductility in positive direction equals 1.49% and negative direction 1.64%. For specimen (2C) the displacement ductility in positive direction equals 1.61% and negative direction 1.64%. For specimen (2D) the displacement ductility in positive direction equals 1.67% and negative direction 1.81%. For specimen (2E) the displacement ductility in positive direction equals 1.89% and negative direction 1.76%. So the average displacement ductility of specimens (2B, 2C, 2D and 2E) is 6.80%, 10.20%, 18.37% and 19.73% were higher than specimen (2A) respectively. These values indicate that the higher Steel Fiber volume on beam column joint was determined to displacement ductility.

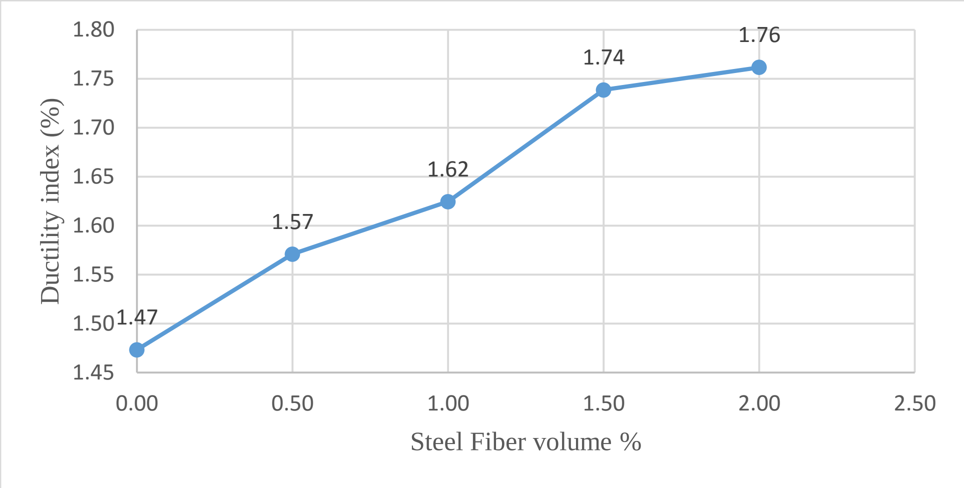

Figure 10: Average Displacement ductility for Group (2)

SUMMARY AND CONCLUSIONS

The findings of the results of the eight beam-column connections tested in the experimental phase, lead to the following conclusions:

i. The joint shear strength of beam column connections with transverse reinforcement (0.56% and 0.85%) were 28.57% and 44.76% higher than beam column connection with transverse reinforcement 0.00% respectively.

ii. The displacement ductility μ δ-b of beam column connections with transverse reinforcement (0.56% and 0.85%) were 1.90 % and 5.00% higher than beam column connection with 0.00% respectively.

iii. The joint shear strength was enhanced due to higher steel fiber volume tile 1.00%. Then the joint shear strength reduces due to higher steel fiber volume. The joint shear strength of joints with steel fiber volume (0.50%, 1.00%, 1.50% and 2.00%) were 2.52%, 13.45%, 9.24% and 6.72% higher than beam column connection with 0.00% respectively.

iv. The displacement ductility μ δ-b of beam column connections with steel fiber volume (0.50%, 1.00%, 1.50% and 2.00%) were 6.80%, 10.20%, 18.37% and 19.73% higher than beam column connection with steel fiber volume 0.00% respectively.

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