Ultimate Capacity of Reinforced Self Compacting Concrete Columns Subjected to Transverse Cyclic Loads

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Abstract:

Experimental investigations were made to determine the ultimate capacity of tied reinforced self compacting concrete columns subjected to both axial and transverse loads. Of the twelve specimens tested, six were without steel fiber inclusions. All the specimens were tested under cyclic load until failure with the presence of constant axial load. Specimens differed in the amount of lateral reinforcement. The effects of fiber inclusion and amount of lateral reinforcement were investigated.

الخلاصة:

لقد أجريت تجارب على أثنى عشر عمودا لتحديد التحمل الأقصى للأعمدة الخرسانية المعمولة من الخرسانة ذاتية الرص والمضاف أليها الألياف الحديدية. لقد خضعت تلك النماذج الى حمل محوري أضافة الى الحمل التعاقبي المسلط على منتصف طول العمود حتى أنهيارة. تقد تم أضافة الألياف الحديدية الى نصف عدد تلك الأعمدة في حين كانت لجميعها كميات مختلفة من الحديد العرضي. أن تأثير الألياف أضافة الى المسافات بين حديد التسليح العرضي قد تمت دراستها.

Introduction:

Self compact concrete SCC,. is a material that can flow under its own weight, so that it can be easily placed in a complicated formwork, congested reinforced structural elements, and hard to reach areas without need for additional mechanical compaction[Corinaldesi V. and Moriconi G. 2003]. With similar water cement or cement binder ratio self compact concrete have a slightly higher strength compared with traditional vibrated concrete, due to the lack of vibration giving an improved interface between the aggregate and hardened paste [Nan S. et al 2001]. Adding steel fibers to concrete mixture improves many engineering properties by different deformation and failure mechanism [Haider A. 2002; Jun, Z. et al 1971 Surendra, p. S. and James I. D. 1974; Swamy R. M. and Mangat P. S. 1974].

When fibers were added to concrete mix, volume fraction Vf and the aspect ratio L/d were taken in consideration due to their effect on workability. To overcome this problem a modification of concrete mix design is recommended. Such modifications include the use of additives [Sana U.B. and John P.F. 2009].

The behavior of fiber reinforced concrete columns and beam-column connections under monotonic and cyclic axial load were tested until failure by several researchers [Ganesan N., Ramana J. V.1990; Ziad B. and Michael G. 2002 ; Hadi M. N.S. 2009; Efe E.I. and Musbau A.S.2011; Gebman M.2001; and Asad E. and Van X. 2004].

The investigations presented in this paper are aimed at examining the feasibility of combining conventional reinforcing steel with fibers in self compacting concrete columns under cyclic load because of little detailed study deals with such concrete. An effort is made in this study to observe the effect of the additives like gilenium 51and filler. This has advantages of enhancing the workability and improving the shrinkage characteristics of concrete.

The method of test can permit to interpret cases which are actually used in practice like:- a member fixed from one end and hinged in the second end(considering half of the element tested).

Research Significance:

The research results reported in this paper are intended to clarify the effect of steel fiber inclusion on the behavior of reinforced self compacting concrete tied columns under load reversals. The research clarifies the elastic-plastic characteristics of these members, and establishes the ultimate capacity and cyclic behavior of these columns subjected to bending in the presence of axial load.

Experimental Program:

All the columns were 1200mm long and 200x200 mm cross-section. The longitudinal reinforcement ratio was 0.0135 with four bars at the corners of the cross-section. The transverse steel ratio and other parameters for the twelve columns are given in Table 1. The columns were tested in the apparatus shown in Fig. 1.

Table 1- Specimens designations**, axial loads, longitudinal and transverse steel**

Columns designations	Lateral reinforcement	Fy MPa	Fu MPa	Longitudinal reinforcement	Fy MPa	Fu MPa	Fiber volume Vf%	N/Agfc′
C1	Ø6mm@200mm			4ø 12mm			O%	0.1
C2	Ø6mm@200mm			4ø 12mm			1%	0.1
C3	Ø6mm@180mm			4ø 12mm			O%	0.1
C4	Ø6mm@180mm	506	650	4ø 12mm	538	650	1%	0.1
C5	Ø6mm@160mm			4ø 12mm			O%	0.1
C6	Ø6mm@160mm			4ø 12mm			1%	0.1
C7	Ø6mm@140mm			4ø 12mm			O%	0.1
C8	Ø6mm@140mm			4ø 12mm			1%	0.1
C9	Ø6mm@120mm			4ø 12mm			O%	0.1
C10	Ø6mm@120mm			4ø 12mm			1%	0.1
C11	Ø6mm@100mm			4ø 12mm			O%	0.1
C12	Ø6mm@100mm			4ø 12mm			1%	0.1

N= axial load, Ag= gross area, fc′= concrete compressive strength, deformed bars

The transverse load (lateral load) was applied at a point in the middle height of the column according to load ratio controlled schedule shown in Fig. 3. The lateral load was applied gradually, realizing as much as possible cyclic loads. The elastic and inelastic cycles were conducted for increasing lateral load level in relation to the maximum load P, with three repeated cycles at each load level. The center to center distance between supports was 1000mm. The axial load was applied through a jack as shown in Fig. 1. Curvatures were measured by five dial gauges on each side of the column (left and right sides), perpendicular to the direction of load application through the length of the column. The measurements were taken in both the pure moment region and the shear span. During each test, the column axial load which was about 53 percent of the maximum axial load (≤ 0.1 Ag fc') remained constant. The lateral load (p) was calculated from nominal flexural strength of the section. The tie spacings, S, were calculated according to ACI code (318-08) and then modified as (S±30%) as shown in Table 1.

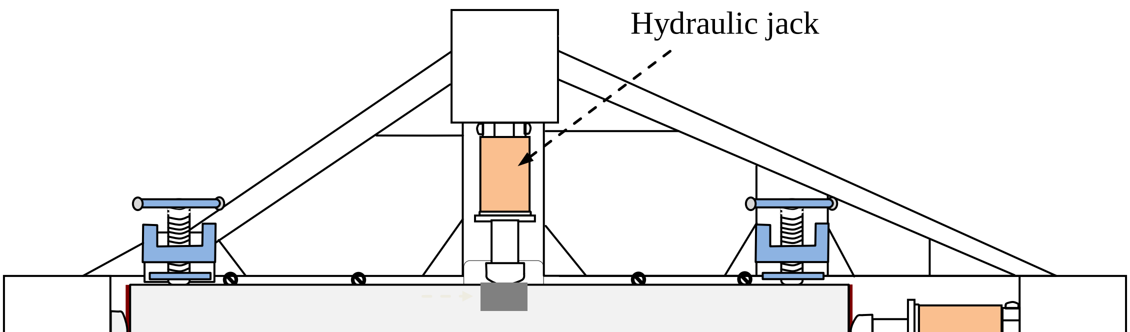


Fig. 1 Testing Apparatus

Construction of test Specimens:

- Experimental Elements

Twelve reinforced concrete columns were constructed and tested. The configuration of these specimens is shown in Fig. 2, the dimensions and reinforcement details are presented in Table 1. The tie spacings were done according to ACI Code(318-08), Nigel, and Graig. All the ties were bent with an angle of 135 degrees and anchored in concrete core to approximately 60 mm long. The ends of the columns were provided by steel plates of 4mm thickness and fixed by two anchorage bars of 80 mm length and 90 degrees hooked end, which extend in column core to 40 mm length to prevent undesired failure due to axial compressive stresses. The specimens were cast upright to simulate construction procedure in the field.

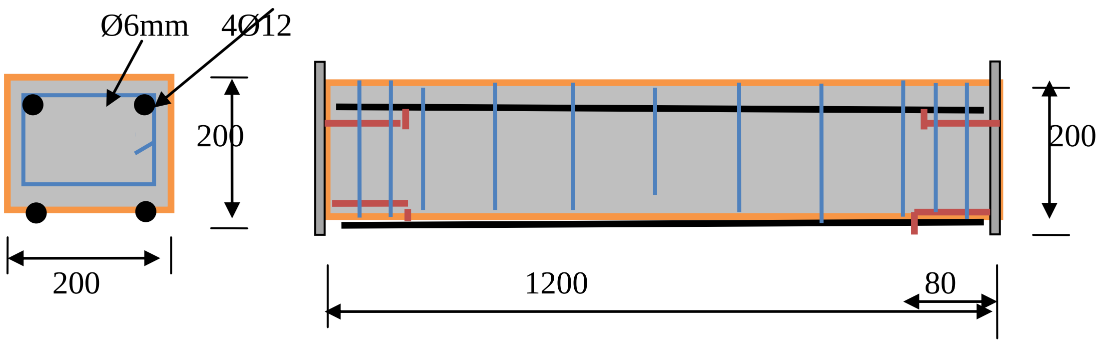


Fig. 2 Details of specimen Configurations

Six cubes of 150x150x150 mm were cast for each column. Three cubes used to determine the compressive strength after 28 days and the other three used to fix the compressive strength on the day of test. Also, six cylinders were cast three of dimensions (100x200)mm to determine the tensile strength and the other three of (150x300)mm to determine the modulus of elasticity.

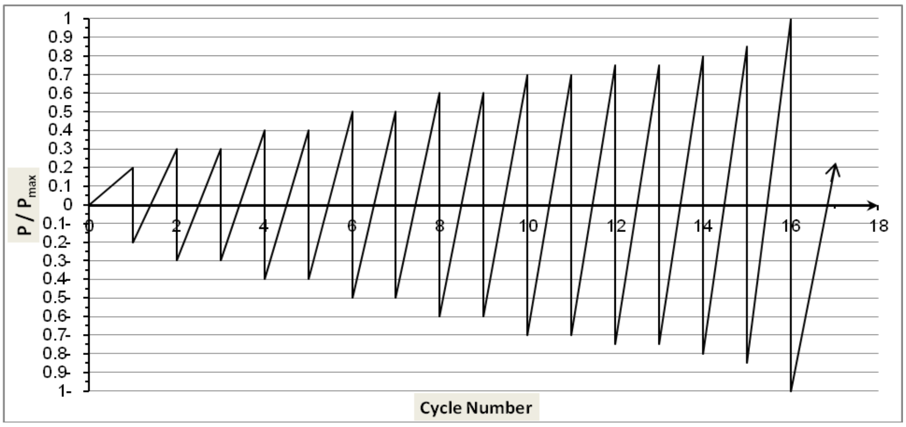


Fig.3 loading history

-Materials Used:

Standard tests according to the specifications of ASTM, and Iraqi specifications IQS-1984 were conducted to determine the properties of materials used in this work.

The cement used was ordinary Portland cement from. The fine aggregate has a fineness modulus of 2.74 and was obtained by sieving and washing by clean water several times. The coarse aggregate was rounded gravel with relative density of 2.65. The grading of these aggregates was according to IQS-1984.

The concrete used in this work was self compacting concrete (SCC) with and without steel fibers. The SCC mix was obtained from trail mix because of the sensitivity of such concrete to water –cement ratio. However, inert and active additives were used to improve and maintain the cohesion and segregation resistance and also to regulate the cement content in order to reduce the heat hydration and thermal shrinkage. A limestone filler was used to improve consistency of particle size distribution giving improved control over water demand. Gilenuim 51 was used as super plasticizer(SP). To accommodate such mix to ERMCO-2005,the trail mixes must pass the following process summarized in Table 3.

Table 3 Summary of the adopted trail mix

Test designations	Vf%		Limits according to ERMCO
	Vf 0%	Vf 1%	
V- funnel	T= 8.12 sec	T=14.49 sec	(7-27)sec
L-box (H2/H1)	0.896	0.720	≥ 0.70
Cone	T500=3.12sec	T500=4.84 sec	(2-5) sec
Cone	Dave=780 mm	Dave=700 mm	(640-800) mm

The proportional mix deduced from the previous test was chosen as shown in Table 4

Table 4 Materials used in SCC Mix

Material	Quantity Kg/m³

Cement	440
Gravel	810
Sand	760
Water	210
Gilenuim 51	5.30
Filler	115
SF	78

The average compressive strength of SCC with 0% fiber content is 23.57 MPa and equal to 25.42 MPa with 1% by volume fiber content at 28 days.

Test Results:

Plots of the applied load versus the point load displacement for all specimens are presented in appendix B. The recorded ultimate loads, first crack loads, and modes of failure of these columns are presented in Table 5. In specimens C1,C2, and C3. Inclined cracks observed. These cracks extend through both sides of the load point. Starting with second cycle of loading for columns C1 and C3 while, these cracks start at third cycle for column C2, due to the presence of steel fibers. Specimens C1 and C3 have explosive failure while C2 (1% Vf) has lenient failure. The shear capacity of column C2 was increased by 30% than column C1 due to fiber inclusion.

Table 5- Loads and failure modes of columns tested.

Column Designation	First visible crack		Ultimate load kN	No. of cycles	Mode of failure	N/Agf′ on the day of test
	Load kN	Cycle				
C1	10	2	50	12	Shear failure	0.107
C2	15	3	65	25	Shear failure	0.0976
C3	10	2	60	15	Shear failure	0.101
C4	20	4	75	28	Flexure failure	0.0942
C5	35	7	75	18	Shear failure	0.110
C6	45	9	85	33	Flexure failure	0.101
C7	40	8	85	22	Shear-flexure failure	0.109
C8	55	11	95	34	Shear flexure failure •	0.104
C9	45	9	90	27	Flexure failure	0.103
C10	55	11	100	36	Flexure failure	0.0977
C11	50	10	95	33	N Flexure failure	0.100
C12	60	12	115	42	Flexure failure	0.0984

In all specimens (C4- C12) a flexural cracks observed first. The cracks propagate towards the loading point and one main flexural crack caused the collapse of the specimens, although some shear cracks appeared during the test. In specimens C5 and C7 the flexural cracks became inert or extend slowly as the intensity of the transverse load increased, whereas the shear cracks became more large and the specimens collapse(see Table 5).Column C8 shows mixed failure which could be due to the rotation of the specimen.

Fig.4(a) to 4(c) show the monotonic envelope for the 12 specimens subjected to cyclic loading. The envelopes were formed by connecting the turning points that occurred during the first cycle of each load level. These envelopes resemble the curves in appendix B for the tested specimens.

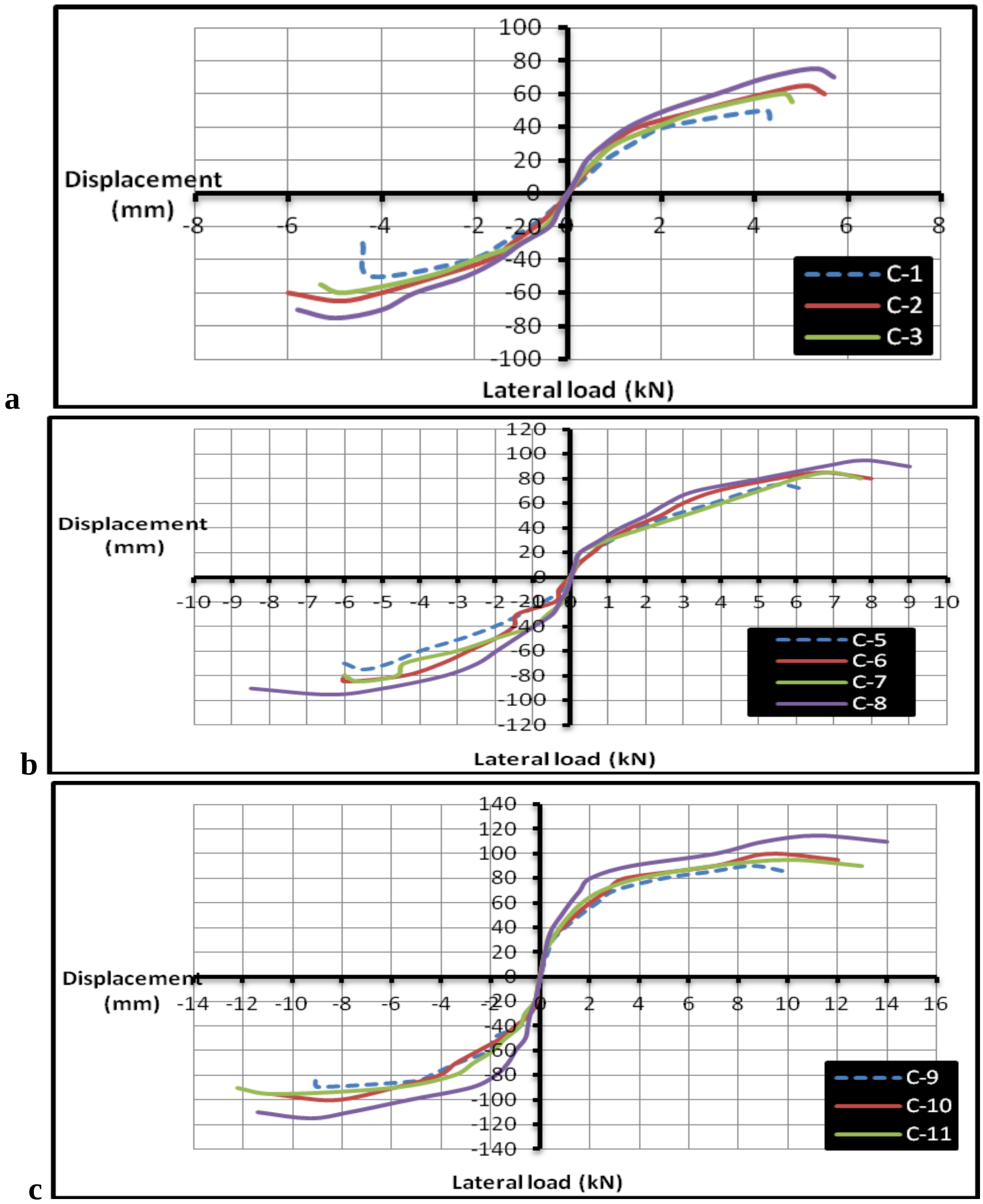


Fig. 4 Lateral load - displacement diagrams for the tested columns

Lateral Reinforcement:

The transverse reinforcement in the column is needed to resist the shear forces and provide confinement for concrete core of the column. The presence of steel fibers also enhances the confinement.

The major benefit of steel fibers is to increase the shear resistance and ductility of the members. They eliminate the sudden and brittle failure mode and transformed it to a ductile mode of failure associated with significant warning near the maximum applied load. The steel fibers not only delay the spalling of the cover concrete but also control the growth and widening of the cracks.

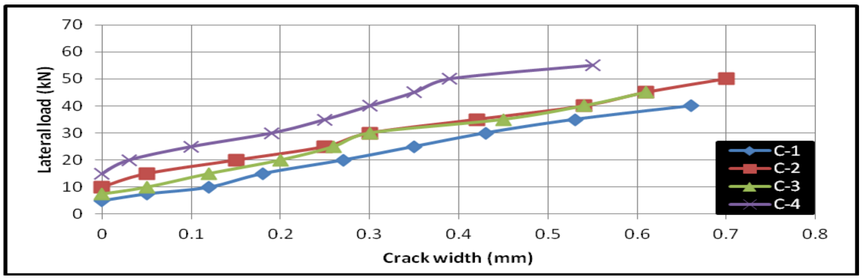
Reducing the tie spacing from 200mm in column C1 to 100mm in column C11 the shear resistance increased about 90 percent and by adding steel fibers (1% Vf) to column C12 increase the shear strength about 130%. Specimen C12 failed by the development of flexural cracks and spalling of concrete cover due to yielding of longitudinal reinforcement at 115 kN. Although, the cracks width was smaller as compared with C11but it is difficult to quantify effect of steel fibers on reducing of concrete spalling during the test.

The addition of steel fibers (1% Vf) to columns shown in Table 5 increases the shear resistance between 10 to 20 kN depending upon the volume of confined reinforcement.

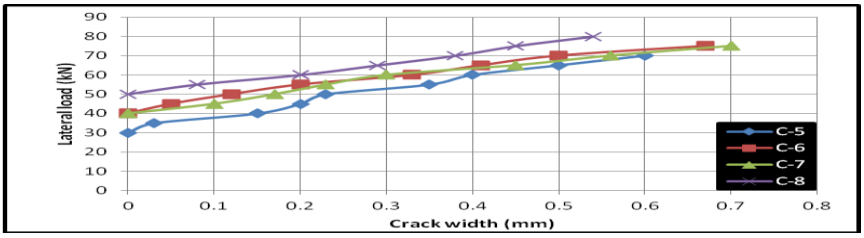
From all the test specimens can be concluded that the effect of steel fibers appear significant at low and moderate load intensities whereas this effect reduced at high loading rate. This probably that most of fibers will be pulled out at this stage.

Concrete Cracking:

The cracks were measured by using crack meter of accuracy 0.01 mm. Fig. 5(a) through 5(c) shows the development of crack width in specimens without fibers was more than in specimens with steel fibers inclusion due to the confinement effect applied by both steel fibers and ties. Moreover, reducing the distance between ties increases the confinement of concrete core and caused an enhancement in the displacement(significantly after the first cracks). Table 6 shows the values of maximum forces realized during loading of the specimens, and also the forces applied during the failure of the specimens.



5-a-



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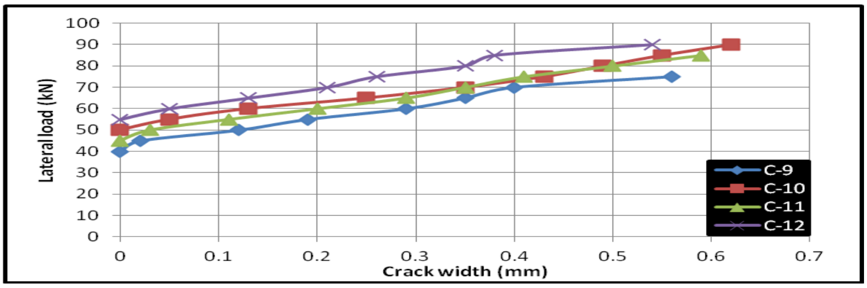

5-c-

Fig, 5 Crack width– Lateral Load displacement For columns tested

Table 6 Summery of experimental results

Col. Disg.	S mm	Vf %	FCL kN	FCW mm	UC	Pu (kN)			Δ• mm	A Joul N.m	μ	Increasing %					
						Exp.	Theo	α%				FCl	UC	Pu	Δ•	A	μ
C1	200	0	10	0.12	12	50	70	71.43	4.25	151	2.13	-----	----	---	---	---	---
C2	200	1	15	0.05	25	65	70	92.86	5.1	248	2.73	50	108	30	20	65.3	28.5
C3	180	0	10	0.05	15	60	80	75	4.8	193	2.70	0	25	20	13	27.8	27.0
C4	180	1	20	0.03	28	75	80	93.75	5.3	301	3.54	100	133	50	24.7	99.3	66.6
C5	160	0	35	0.03	18	75	93	80.65	5.6	310	3.50	----	-----	----	----	----	----
C6	160	1	45	0.05	33	85	93	91.40	6.7	491	3.83	28.6	83.3	13.3	19.6	58.4	9.4
C7	140	0	40	0.10	22	85	99	85.86	6.9	428	5.09	14.3	22.2	13.3	23.2	38.1	45
C8	140	1	55	0.08	34	95	99	95.96	7.8	623	6.24	57.1	88.9	26.7	39.3	100	78
C9	120	0	45	0.02	27	90	99	90.1	9.0	713	6.34	-----	-----	-----	----	-----	----
C10	120	1	55	0.05	36	100	99	101	9.2	962	7.63	22.2	33.3	11.1	2.2	35.9	20.3
C11	100	0	50	0.03	33	95	99	95.96	10.7	1068	10.1	11.1	22.2	5.6	19	49.8	59.3
C12	100	1	60	0.05	42	115	99	116.2	11.2	1339	11.8	33.4	55.5	27.7	24.4	87.8	86.1

Conclusions:

Based on the results of test the following conclusions can be obtained:

1. For a constant value of W/C ratio and dosage of SP, addition of steel fibers caused a reduction in value of Dave in cone test by about 11.4%.
2. Addition of steel fibers increased ductility of the specimens by (10%-28%), therefore, steel fibers was able to change the mode of failure from brittle failure to a ductile failure.
3. It is noted that strengthening of SCRC columns by steel fibers resists the crack propagation, decreases the crack width, and changed the formation of cracks.
4. The use of steel fibers has significant effect on shear resistance more than flexural; It's noticed that addition of SF increased shear resistance by about 30%, while, only 11% an increment in flexural resistance. Therefore, steel fibers were able to change the failure from shear failure to a flexural failure.
5. First crack strength was higher for columns containing fibers compared with columns strengthed by stirrups only. This can lead to the advantage of using steel fibers for repair of deffected members.
6. The spalling of concrete cover for columns containing fibers was small enough to be egnored, and the amount of cracking was reduced considerably compared with coventionally reinforced columns.
7. The use of steel fibers enhances the confinement of the member and increased the cracking cycle, cycles number, absorbed energy, and displacement by about (20%-100%), (27%-108%), (25%-64%), and (2.2%-20%) respectively.
8. From columns tested, it can be concluded that some amount of confining re-inforcement (ties) can be replaced by the addition of short, randomly oriented steel fibers. It's noticed that steel fibers could replace about (10-20) % of lateral steel reinforcement (ties).

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Appendix A Notation

A = energy absorbed

Ag = gross area of column

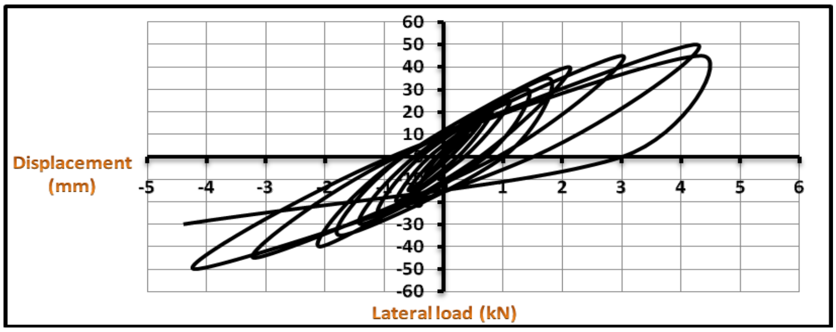
FCL = first crack load

FCW = first crack width

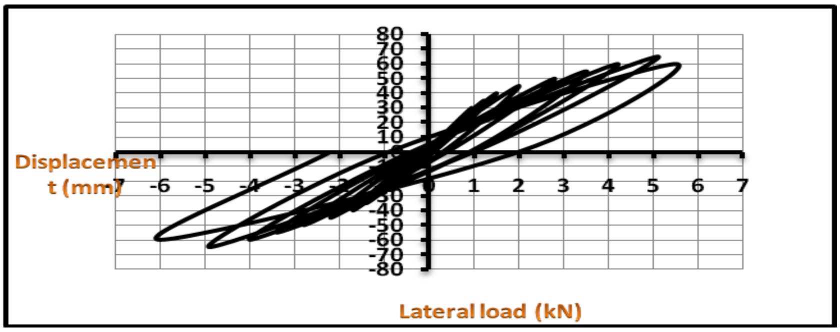
N = axial load

P = lateral cyclic load
S = tie spacing
Uc = ultimate cycle
Vf =fiber volume fraction
Δ = displacement in mm
μ = ductility of the column

Appendix B



C1



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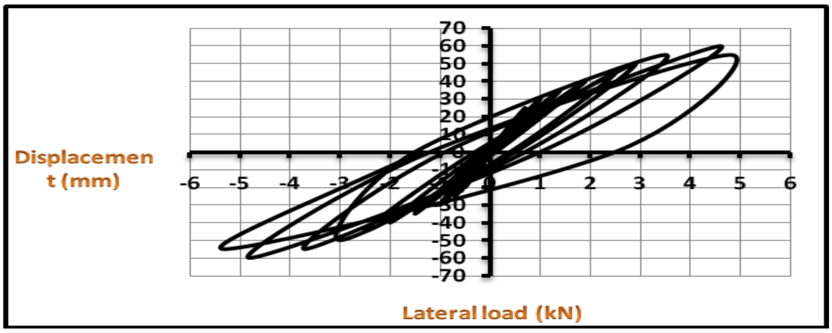

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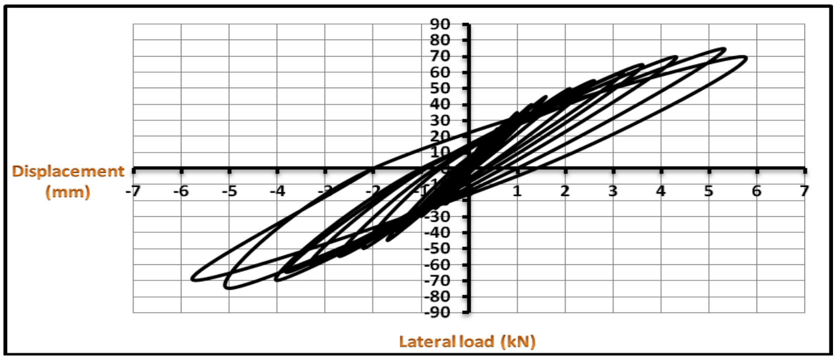

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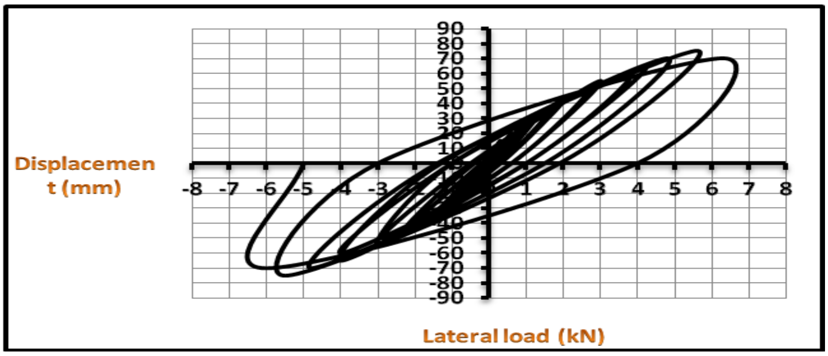
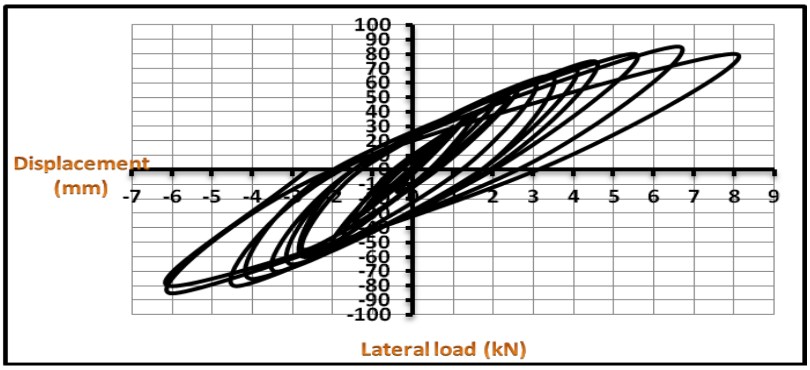
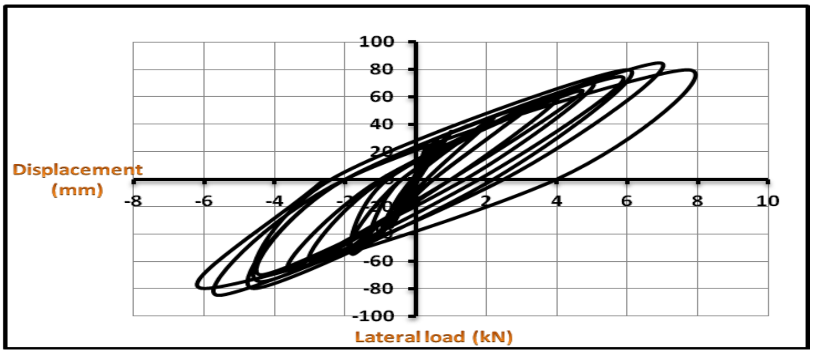

Displacemen
t (mm)
Lateral load (kN)

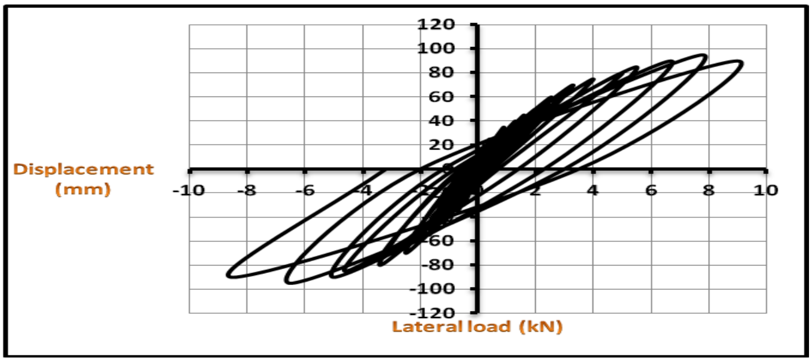
C5


Displacement
(mm)
Lateral load (kN)

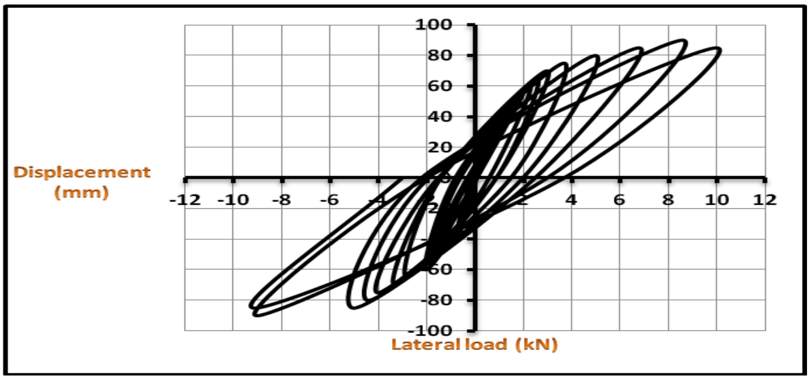
C6


Displacement
(mm)
Lateral load (kN)

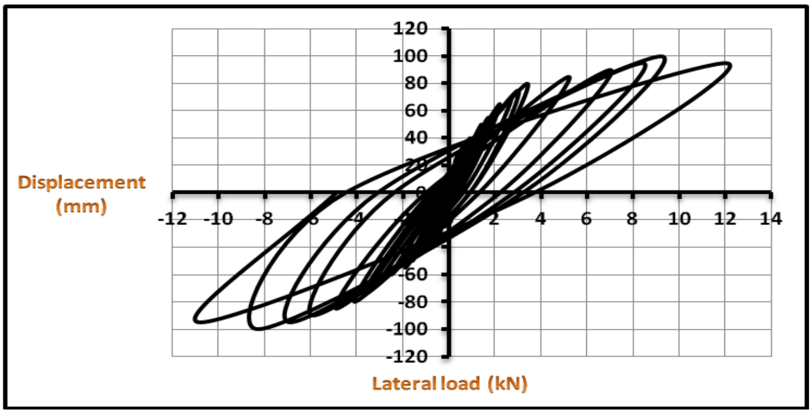
C7


Displacement
(mm)
Lateral load (kN)

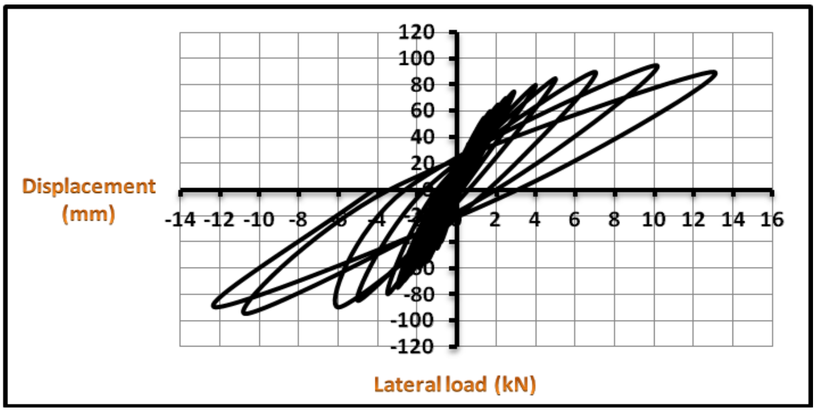
C 8



C 9



C 10



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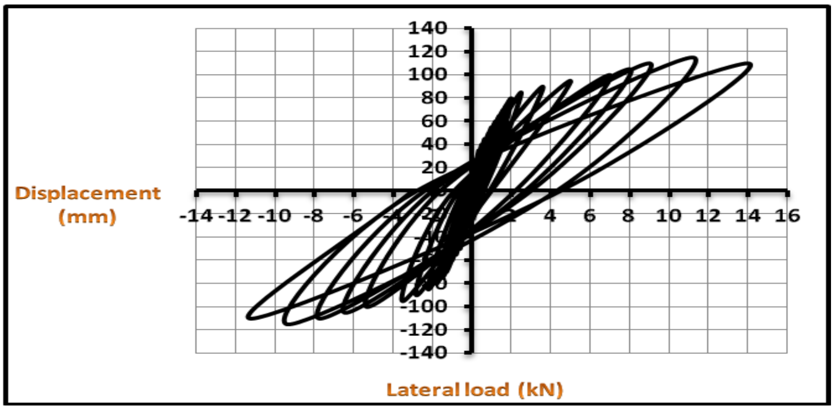

B-1 Load versus deflection response for the 12 columns tested