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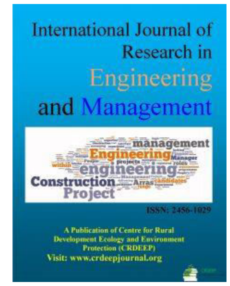

Full Length Research Paper

Effect of Cross section and various Reinforcement ratios on Flexural behavior of HSC& UHSC beams

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ABSTRACT

The main objectives of this research addressed through the experimental work described in this chapter. The main objective is to study the flexural behavior of ultra-high strength concrete beams subjected to concentrated loads. The program incorporated two phases in which specimens were designed and cast to study the desired properties. The first phase reviews a detailed tested specimen. While the second phase deals with the characteristics of the used material, mix proportions, preparation of specimens and loading system. In the current research, eight reinforced UHSC beams were cast and statically tested under the effect of two vertical concentrated loads up to failure at the reinforced concrete laboratory of housing and building research center to investigate the modes of failure, the ultimate load carrying capacity and the flexural behavior of UHSC concrete. The deformation, vertical deflection, concrete strain, longitudinal compression steel strain and tension steel strain were measured. The observed parameters and the description of the specimens and reinforcement in current work will be discussed later.

Introduction

Ultra High Strength Concrete (UHSC) is a new advanced concrete that has been transferred from laboratory research to practical applications. Based on the latest developments in concrete technology, UHSC is characterized by extra ordinary mechanical properties (high compressive and tensile strengths & high young's modulus) and has excellent durability properties. Up till now, the use of UHSC is very limited due to its unknown properties that differ from normal strength or high strength concrete. Moreover, the very low ductility of UHSC leads to brittle failure of members produced from it. The deformation behavior of the ultra-high strength matrix in comparison to normal Strength concrete is a contrary matter. Ultra High Strength Concrete (UHSC) with a high compressive strength and an improved durability marks a quantum leap in concrete technology. This high-performance material offers a variety of interesting applications. It allows the construction of sustainable and economic buildings with an extraordinary slim design. Its high strength makes it a suitable building material for bridge decks, storage halls, thin-wall shell structures and highly loaded columns. A new formulation approach by using ultra-fines materials supported by strong development of new admixtures open the way over the last twenty years to amazing progresses in concrete technology. The range of performances and characteristics that are today covered by concrete have been expanded in various directions from ordinary concrete up to ultra-high-performance concrete.

Ultra-High-Performance Concrete (UHPC) is a new class of concrete that has been developed in recent decades. When compared with high performance concrete (HPC), UHPC tends to exhibit superior properties such as advanced strength, durability, and long-term stability. The production of ultra-high strength concrete (UHSC) has been researched in the 1970s. Compressed cement paste achieved a compressive strength of 65 N/mm^2 under heat curing conditions. Intensive research on high performance concrete enhances the knowledge about the use of chemical and mineral admixture in concrete. On this basis a new concrete with a compressive strength of more than 180 N/mm^2 was developed at the beginning of the 1990s in France and Canada with the elimination of coarse aggregate [30]. Higher strength offers savings in material. Weight, or the structural dead load, is a major loading in the design of structures. Consequently, higher strength usually gives us two advantages: less material and less weight. The reduction in weight in turn reduces the demand on material because it reduces the load the structure has to carry. With strength of 200 MPa, the UHPC is almost like steel except its tensile capacity is still comparatively low so it cannot be used like steel. However, this is many folds higher than the regular concrete, the strength of which is around 50 MPa. [46]

The Experimental program

- **Objective of the Experimental Program**

The main objective of this study is to investigate flexural behavior of Ultra High Strength Concrete (UHSC) Beams. The various specific objectives are:

1. Studying the flexural behavior of UHSC beams.
2. Investigate to optimum minimum reinforcement ratio with minimum crack width UHSC beams.
3. Investigate the others cross section on UHSC
4. Recommendations for ECP for UHSC.

- **Details of Test Specimens**

The experimental program was conducted on twelve UHSC concrete beams. All beams with total span 3400 mm were simply supported with 3000 mm clear span, 200 mm projection at each end as shown as Figure.1 and tested beams detailed at Table.1

The first group: consists of four specimens. The first two specimens with cross section dimensions 100 mm wide and 200 mm height. The low longitudinal steel reinforcement with various reinforcement ratios are (1%, 2%),
The second two specimens with cross section dimensions 100 mm wide and 300 mm height. The lower longitudinal steel reinforcement with various reinforcement ratios are (1.2%, 1.8%),
while for all specimen the upper longitudinal steel reinforcement consists of 2Ø10.
The transverse reinforcement consists of closed stirrups 8 mm diameter @100 mm (2-branches) for the first three specimens,
the last one specimen has the transverse reinforcement consists of closed stirrups 10 mm diameter @100 mm (2-branches). the compressive strength of cubes is 120 Mpa. The concrete mix for all specimens is with 1% fiber reinforcement

The second group: consists of five specimens with cross section dimensions 100 mm wide and 250 mm height. The low longitudinal steel reinforcement with various reinforcement ratio are (0.7%, 1%, 1.4%, 1.8%, 2.2%), while the upper longitudinal steel reinforcement consists of 2Ø10. The transverse reinforcement consists of closed stirrups 8 mm diameter @100 mm (2-branches) the compressive strength of cubes is 120 Mpa. The concrete mix for the all specimens is with 1% fiber reinforcement

Table.1: Details of specimens.

Groups	*Specimens*	*V_f %*	*Main Steel*	*P_{sv} %*	*F_{cu}*	*b*t (mm)*
The First Group	120UH20-1%	1%	2 ϕ10	1%	120	10*20
	120UH20-2%	1%	4 ϕ10	2%	120	10*20
	120UH30-1.2%	1%	4 ϕ10	1.2%	120	10*30
	120UH30-1.8%	1%	4 ϕ12	1.8%	120	10*30
The Second Group	120UH25-0.7%	1%	2 ϕ10	0.7%	120	10*25
	120UH25-1%	1%	3 ϕ10	1%	120	10*25
	120UH25-1.4%	1%	4 ϕ10	1.4%	120	10*25
	120UH25-1.8%	1%	5 ϕ10	1.8%	120	10*25
	120UH25-2.2%	1%	6 ϕ10	2.2%	120	10*25

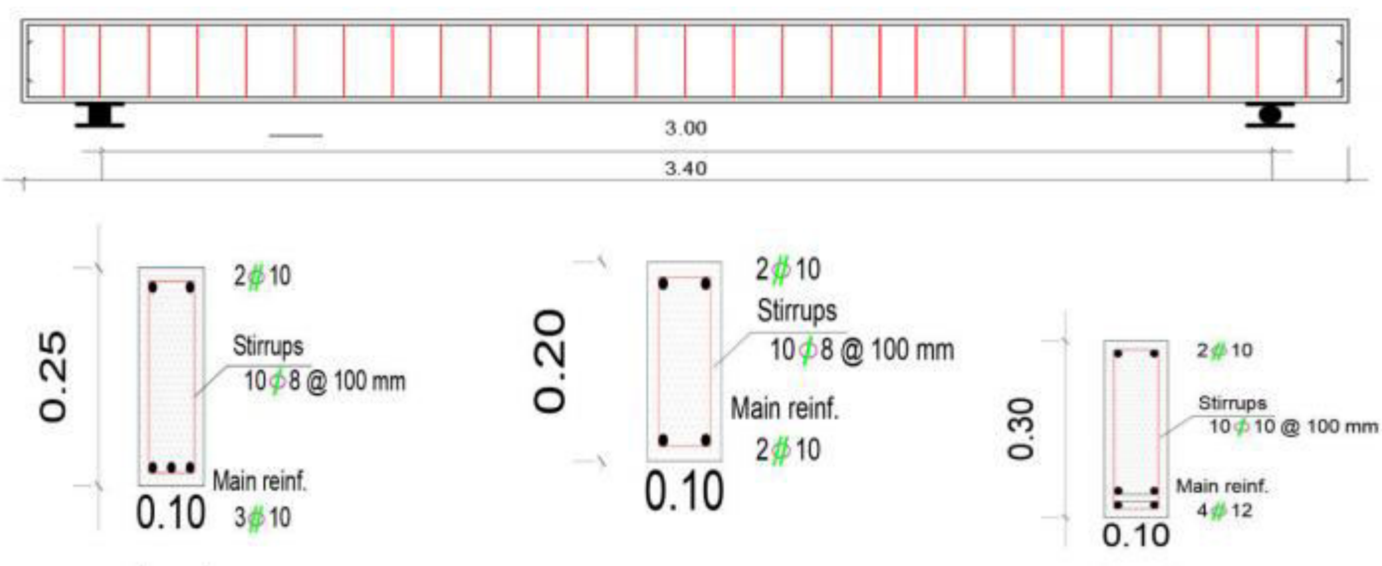

Fig. 1 Details of specimens

` Material properties

The concrete mix was ordered from a concrete mixing plant with a characteristic concrete strength of 120 MPa. Two sets of six concrete cubes and six concrete cylinders were cast alongside the beams,

Table.2 Mix proportion of Ultra-High Strength Concrete

Mix	**Cement**	**S.F.**	**Sand /Total agg.**		**Quartz powder/ Total agg**	**Coarse aggregate /Total agg.**	**W/B**	**Ad**	**St Fiber**	**Curing**
						Dolomite		%	%	
	kg/m^3	%	Siliceous	Quartz		D_5				
Mix 120	800	20	0.25		0.25	0.5	0.16	4	1	SC-7d

Experimental results and analysis

A total of nine beam specimens were tested up to failure. All of them failed in flexural before Shear capacity is reached. A summary of the test results for each tested beam specimens is presented in Table.3, includes the reinforcement ratio, depth to width ratio, moment cracking load, final and crack width, and the failure load.

Cracking behavior

Similar characteristics of crack formation were observed for all beams. The crack formation was initiated in the flexural span between the two concentrated loads where the tensile stress is highest and shear stress is zero. the cracks were perpendicular to the direction of the maximum principal tensile stress induced by pure bending, as the load increased, the numbers, and width of cracks had increased.

cracking behavior for group I

- For specimen **120UH20-1%** the specimen had experienced the formation of fine cracks vertically in the middle of beam span between the two-point load he final failure of that specimen was characterized by flexural failure in the mid-span it were small the initial crack load about 30 KN presented by FIG.2. weighted and tested on the same day of beam testing to ensure the resistance of the required concrete.
- For specimen **120UH20-2%** the specimen had experienced the formation of fine cracks vertically in the middle of beam span between the two-point load he final failure of that specimen was characterized by flexural failure in the mid-span it were small the initial crack load about 50 KN, the failure load was increased by 45% about **120UH20-1%**presented by FIG.3.
- For specimen **120UH30-1.2%** the specimen had experienced the formation of fine cracks vertically in the middle of beam span between the two-point load he final failure of that specimen was characterized by flexural failure in the mid-span it were small the initial crack load about 80 KN presented by FIG.4.
- For specimen **120UH30-1.8%** the specimen had experienced the formation of fine cracks vertically in the middle of beam span between the two-point load he final failure of that specimen was characterized by flexural failure in the mid-span and it were small the initial crack load about 95 KN the failure load was increased by 23% about **120UH30-1.2%**presented by FIG.5.

Cracking behavior for group II

- For specimen **120UH25-0.7%** the specimen had experienced the formation of fine cracks vertically in the middle of beam span between the two-point load he final failure of that specimen was characterized by flexural failure in the mid-span it were small the initial crack load about 45 KN presented by FIG.6.
- For specimen **120UH25-1%** the specimen had experienced the formation of fine cracks vertically in the middle of beam span between the two-point load he final failure of that specimen was characterized by flexural failure in the mid-span it were small the initial crack load about 60 KN the failure load was increased by 29% about **120UH25-0.7%** presented by FIG.7.
- For specimen **120UH25-1.4%** the specimen had experienced the formation of fine cracks vertically in the middle of beam span between the two-point load he final failure of that specimen was characterized by flexural failure in the mid-span it were small the initial crack load about 75 KN the failure load was increased by 58% about **120UH25-0.7%** presented by FIG.8.
- For specimen **120UH25-1.8%** the specimen had experienced the formation of fine cracks vertically in the middle of beam span between the two-point load he final failure of that specimen was characterized by flexural failure in the mid-span and it were small the initial crack load about 80 KN the failure load was increased by 80% about **120UH25-0.7%** presented by FIG.9.
- For specimen **120UH25-2.2%** the specimen had experienced the formation of fine cracks vertically in the middle of beam span between the two-point load he final failure of that specimen was characterized by flexural failure in the mid-span it were small the initial crack load about 115 KN the failure load was increased by 123% about **120UH25-0.7%** presented by FIG.10.

It was found at sample **120UH25-0.7%** designed with A_s min by ECP (201) the section had balanced failure because the $\mathbf{P_{cracking}}$ equal the $\mathbf{P_{ultimate}}$ it means that the longitudinal steel wasn't contributed with UHSC at its behavior but when designed at sample **120UH25-1.4%** by 2-times As min by ECP (201) equation it was found the contribution of longitudinal steel was founded $\mathbf{P_{cracking}}$ equal (2/3) the $\mathbf{P_{ultimate.}}$

AND;

According to calculations of $\mathbf{A_s}$ min with $\mathbf{M_{cr}}$ & $\mathbf{F_{ctr}}$ experimentally it was found the main reinforcement ratio covered the $M_{cr\ EXP}$ equal 2-times As min by ECP (201) equation; the sample **120UH25-1.4%** designed by 2-times reinforcement ratio by ECP (201); it was found the $P_{cracking}$ of sample **120UH25-1.4%** experimentally equal the $P_{cracking}$ analytically and it both of them equal (2/3) the $P_{ultimate.}$ All beams at group II by increased the reinforcement steel ratio increased the ultimate load and decreased the deflection values and crack width values

Table.3. Test results summary

Groups	Sample	ACI Design Failure Load (KN)	EXP Failure Load (KN)	P Cracking Fctr exp	P Cracking curves	Final Crack Width (mm)	Max. Deflection At Middle Span (mm)	D/W
Group.1	120UH20-1%	21.6	35	38.25	30	4	77	2
	120UH20-2%	42	64	40.5	50	3.2	69	
	120UH30-1.2%	68.5	113	89	80	7.2	55	3
	120UH30-1.8%	97.2	139	93.6	95	3.6	29	
Group.2	120UH25-0.7%	28.2	55	59	45	10	80	2.5
	120UH25-1%	42	71	61.2	60	8	75	
	120UH25-1.4%	55.25	87	62.6	75	6.7	70	
	120UH25-1.8%	68.3	99	64	80	4	50	
	120UH25-2.2%	81	123	66	95	2	35	

Fig.2 crack pattern and the failure in specimen 120UH20-1%

Fig.3 crack pattern and the failure in specimen 120UH20-2%

Fig.4 crack pattern and the failure in specimen 120UH30-1.2%

Fig.5 crack pattern and the failure in specimen 120UH30-1.8%

Fig.6 crack pattern and the failure in specimen 120UH25-0.7%

Fig.7 crack pattern and the failure in specimen 120UH25-1%

Fig.8 crack pattern and the failure in specimen 120UH25-1.4%

Fig.9 crack pattern and the failure in specimen 120UH25-1.8%

Fig.10 crack pattern and the failure in specimen 120UH25-2.2%

Crack width

Crack width of all beams was measured using linear variable displacement transducers (LVDT's) and recorded data using a data acquisition system. The crack width was measured to determine the maximum crack width through the span of each tested beam from zero up to the failure load.

The experimental load-crack width curves of the beams are shown in Figures (11) to (15) The values of maximum crack width of beam are shown in Table.3.

- **Load-Crack Width of Group I**
- The load- crack width relationship of third group with (D/W=2.0) beams showed that the crack width decreased with an increase of reinforcement steel ratio the crack width of UHSC-Beams with different longitudinal steel ratio (1%, 2%) and the same aspect ratio (D/W=2.0) decreased gradually by 20%.as shown as figure.11
- The load-deflection relationship of third group with (D/W=3.0) beams showed that the deflection decreased with an increase of reinforcement steel ratio the crack width of UHSC-Beams with different longitudinal steel ratio (1.2%, 1.8%) and the same aspect ratio (D/W=3.0) decreased gradually by 50%. For the load-crack width relationship shown in Figure.12

- **Load-Crack width of Group II**

The load- crack width relationship of second group beams showed that the crack width decreased with an increase of reinforcement steel ratio by 20%, 33%, 60%, and 80% compared to the reference beam (120UH25-0.7%). For the load-crack width relationship shown in Figure.13

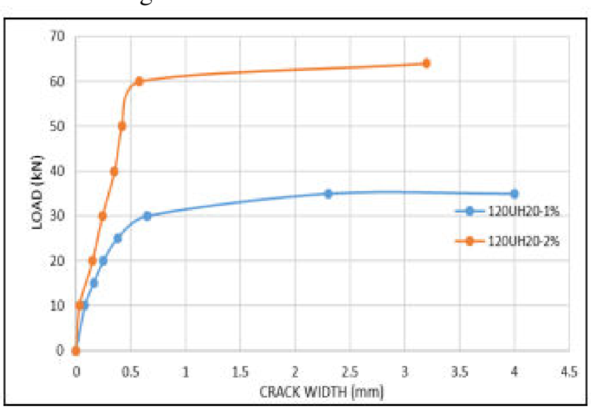

Fig.11 Load –Crack width Curve for Group I D/W=2

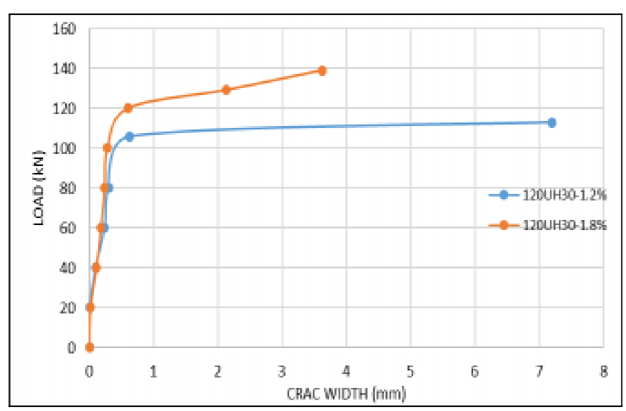

Fig.12 Load –Crack width Curve for Group.I D/W=3

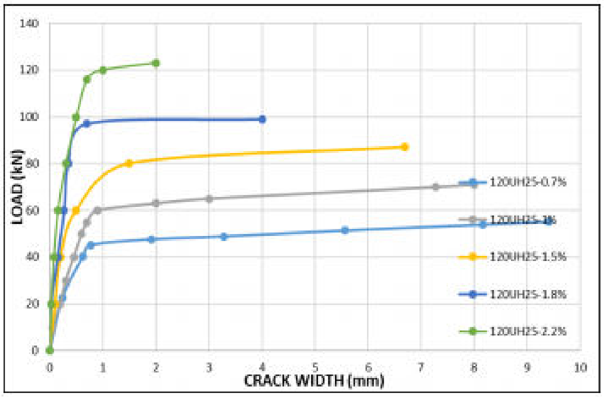

Fig.13 Load –Crack Width Curve for Group.II

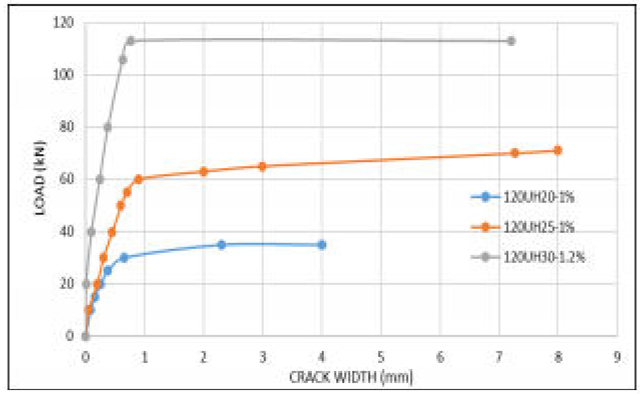

Fig 14 Effect the same reinforcement steel ratio with others cross section on load-crack width curve

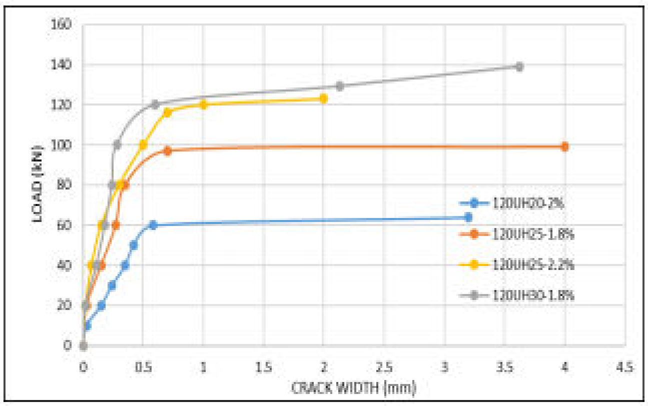

Fig15 Effect the same double reinforcement steel ratio with others cross section on load crack width curve

LOAD-DEFLECTION RESPONSE

Deflection of all beams was measured using linear variable displacement transducers (LVDT's) and recorded data using a data acquisition system. the vertical deformations were measured by three points on each beam and one point in the mid-span of the beam and other two point at first and last quarter of to predict the deflection shape of the tested beams. The experimental load-deflection curves at points of maximum deflection of the beams are shown in Figures (16) to (21). The values of maximum deflection of beam are shown in Table.3.

- **Load-Deflection Response of Group I**
- The load-deflection relationship of third group with (D/W=2.0) beams showed that the deflection decreased with an increase of reinforcement steel ratio the vertical deflection of UHSC-Beams with different longitudinal steel ratio (1%, 2%) and the same aspect ratio (D/W=2.0) decreased gradually by 11%). as shown in figure.16

- The load-deflection relationship of third group with (D/W=3.0) beams showed that the deflection decreased with an increase of reinforcement steel ratio the vertical deflection of UHSC-Beams with different longitudinal steel ratio (1.2%,1.8%) and the same aspect ratio (D/W=3.0) decreased gradually by (48%) as shown in figure.17

- **Load-Deflection Response of Group II**

The load-deflection relationship of second group beams showed that the deflection decreased with an increase of reinforcement steel ratio by 7%, 13%, 38%, and 57%) compared to the reference beam (120UH25-0. 7%).For the load-deflection relationship as shown in Figure.18

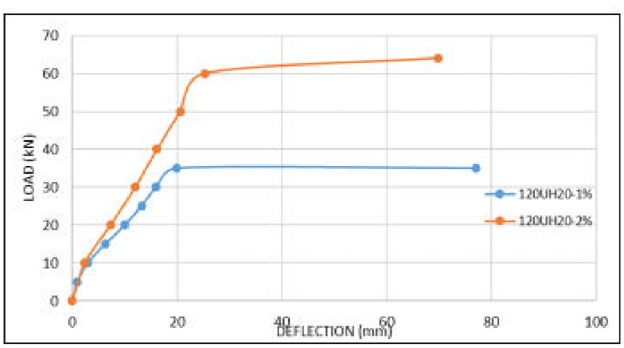

Fig.16 Load –Deflection Curve for Group I D/W=2

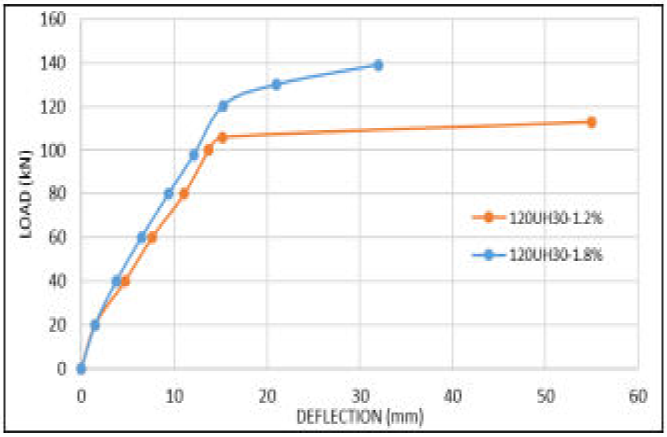

Fig.17 Load –Deflection Curve for Group I D/W=3

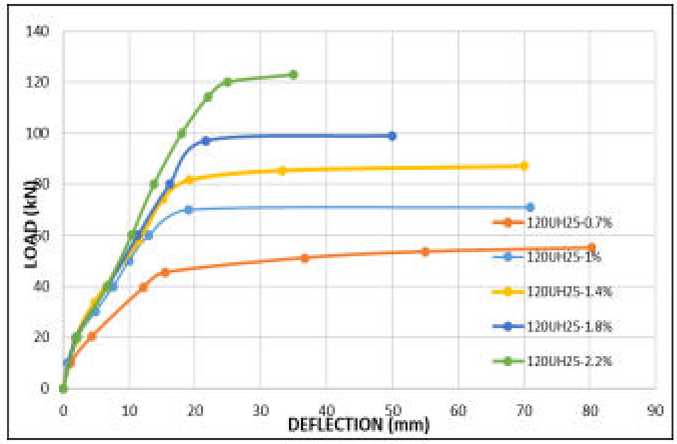

Fig.18 Load –Deflection Curve for Group.II

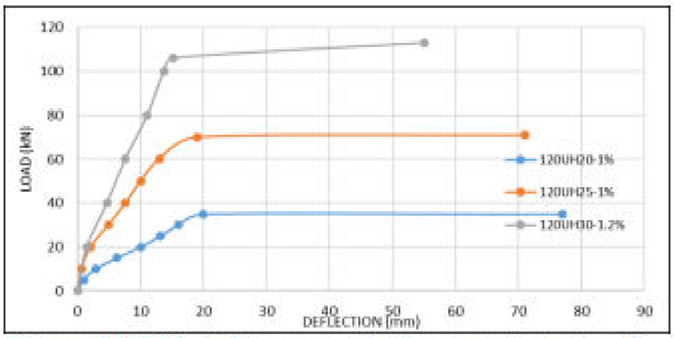

Fig.19 Effect the same reinforcement steel ratio with others cross section on load-deflection curve

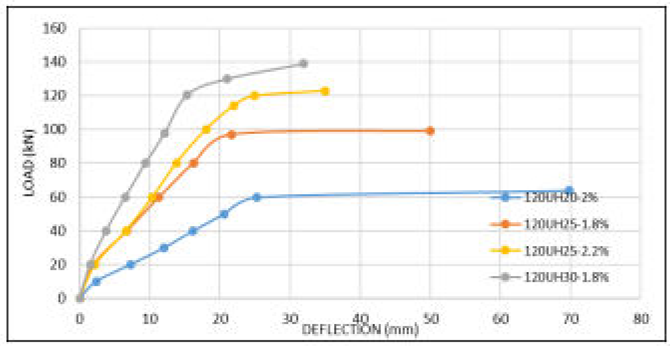

Fig.20 Effect the same double reinforcement steel ratio with others cross section on load-deflection curve

CONCRETE STRAIN

strain gauges attached on the top of the concrete surface and horizontally LVDT to measure the concrete strain at the compression zone during testing in order to ensure that the compression failure doesn't occur during load increment

Compressive Concrete Strain of Group I

- For group.III. it was found the beams with (D/W=2.00) have low value of concrete strain comparing by group II lower by 50% at reinforcement steel ratio by ECP but when double reinforcement steel ratio by ECP the concrete strain lower by 25% of group II as shown in figure.21
- And; For group.III. it was found the beams with (D/W=3.00) have low value of concrete strain comparing by group II lower by 25% at reinforcement steel ratio by ECP and double it. as shown in figure.22
- **Compressive Concrete Strain of Group II**

The load- concrete strain relationship of second group beams showed that the concrete strain increased with an increase of reinforcement steel ratio at the same load of yielding of longitudinal main steel to reach the max strain at failure =0.004.the value of concrete strain for all beams are constant at ultimate load are presented in Figure.23

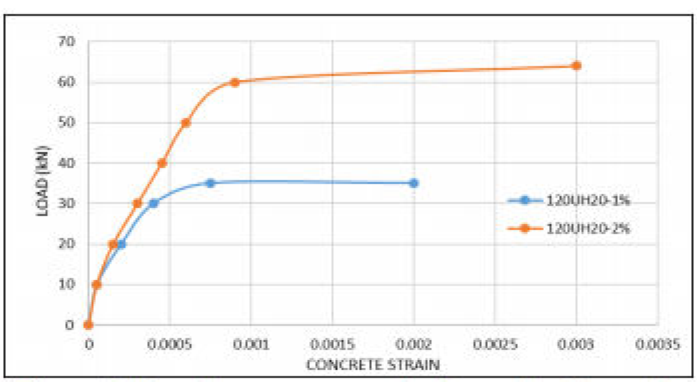

Fig.21 Load –concrete strain Curve for Group I D/W=2

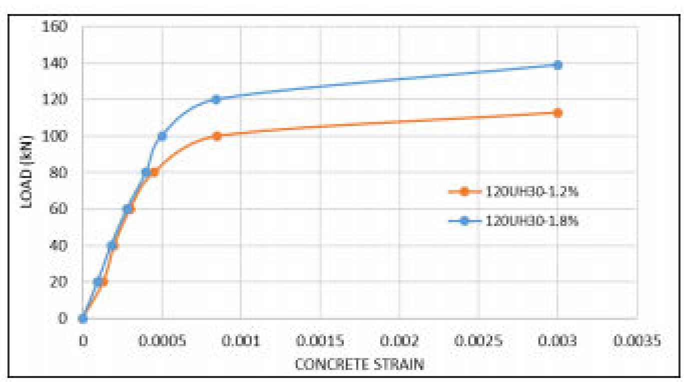

Fig.22 Load –Concrete strain Curve for Group I D/W=3

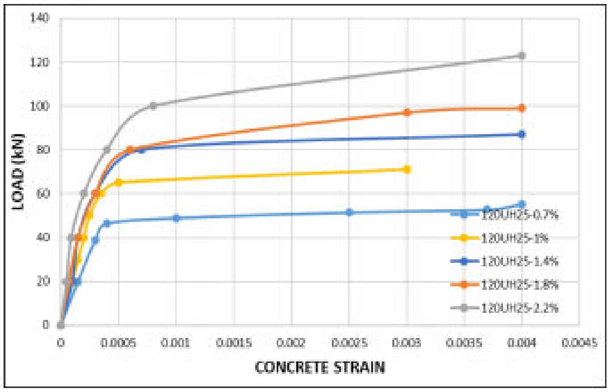

Fig.23 Load –Concrete strain Curve for Group.II

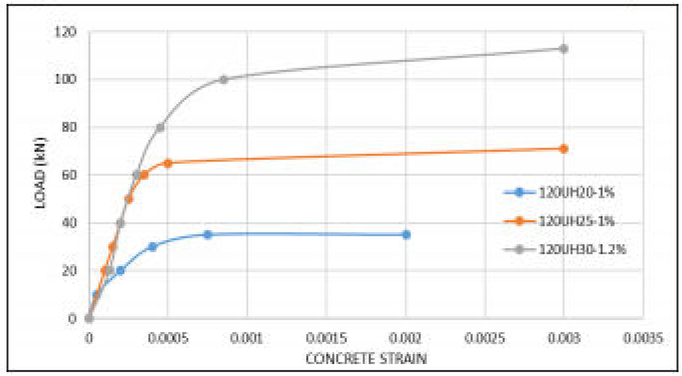

Fig.24 Effect the same reinforcement steel ratio with others cross section on load-deflection curve

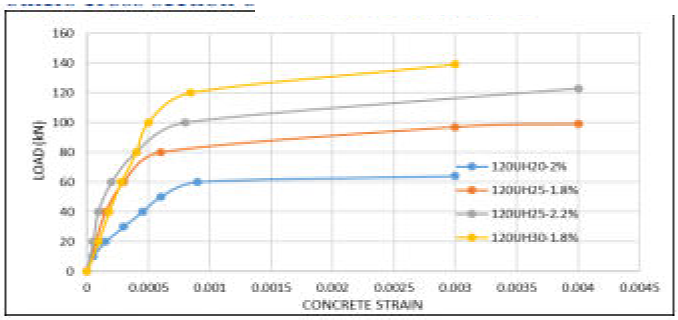

Fig.25 Effect the same double reinforcement steel ratio with others cross section on load-deflection curve

LONGITUDINAL STEEL STRAIN

Attached strain gauges at the bottom longitudinal bars were used to measure the steel reinforcement stain during testing process connected to data acquisition system.

- **Longitudinal Steel Strain of Group I**
- BUT For group.III. it was found the beams with (D/W=2.00) have high value of longitudinal steel strain by 25% comparing by group II as shown in figure.26
- And For group.III. it was found the beams with (D/W=3.00) have equal value of longitudinal steel strain comparing by group II as shown in figure.27
- **Longitudinal Steel Strain of Group II**

The load- Longitudinal Steel Strain relationship of second group beams showed that enhance the longitudinal steel strain by 60% from steel strain by 2-times minimum reinforcement ratio of the ECP for depth to width ratio 2.5 compared to (120UH25-0. 7%). For the load-longitudinal steel strain relationship as shown in Figure.28.

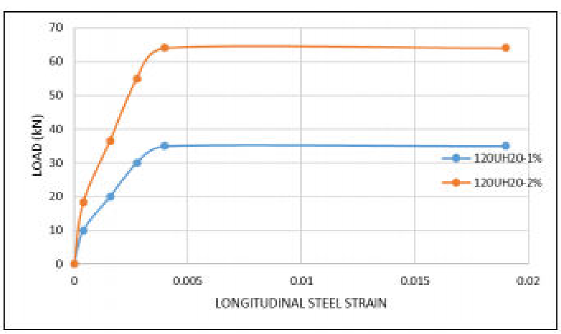

Fig.26 Load – longitudinal steel strain Curve for Group I D/W=2

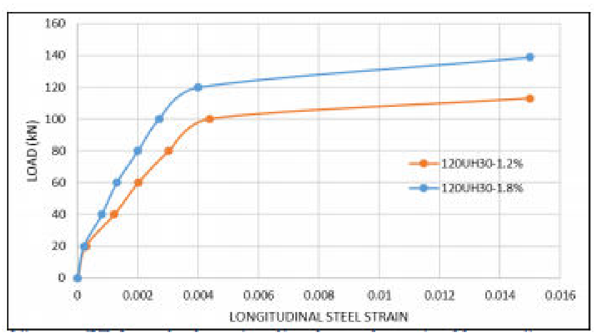

Fig.27 Load –longitudinal steel strain Curve for Group I D/W=3

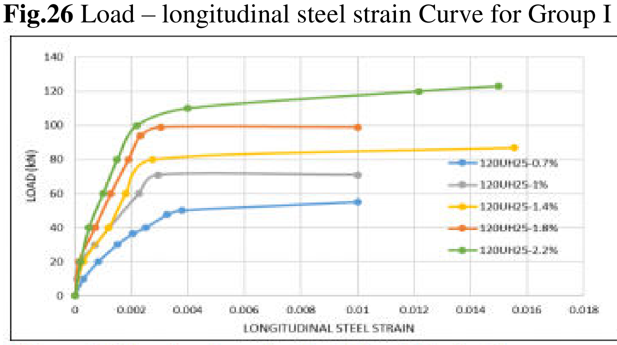

Fig.28 Load – Longitudinal Steel Strain Curve for Group.II

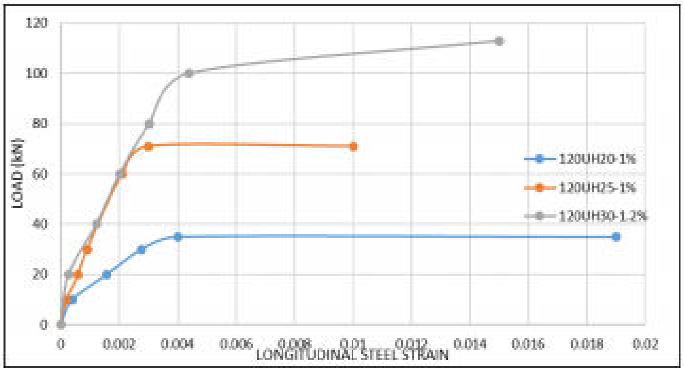

Fig.29 Effect the same reinforcement steel ratio with others cross section on load-deflection curve

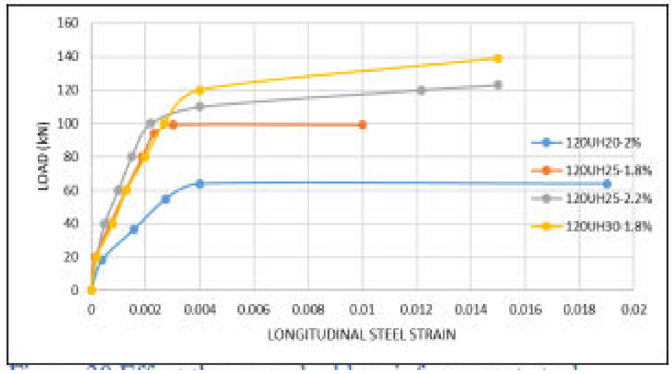

Fig.30 Effect the same double reinforcement steel ratio with others cross section on load-deflection curve

Conclusion

1- According to calculations it was found the cracking load is equal or more ultimate load for specimens have reinforcement ratio according to ECP it means that the longitudinal tension steel was not contributed with UHSC beam behavior but when increase3d the value reinforcement ratio of ECP the contribution of the longitudinal tension steelwas founded by difference between cracking load equal 2/3 ultimate load.

2- For all examined full-scale beams, it was observed that all failure modes were due to pure bending without any signs of shear failure modes. The cracks were generated mainly in the mid-span of the beam and particularly between the two points of load application. For all beams the strain of the tensile reinforcement was beyond the yield value (i.e. greater than 0.0018), simultaneously, the strain of compressive steel was less than or equal to concrete crushing strain (i.e. εc = 0.003).

3- In the experimental studies, for (G1) the ultimate load of UHSC-Beams with different longitudinal steel ratio (1.5%, and 3%) and the same (width : thickness) ratio of (1:2) increased gradually by 45%) compared to (UHB20-1.5%) the vertical deflection of UHSC-Beams with different longitudinal steel ratio (1.5%, %3%) and the same aspect ratio (1:2) decreased gradually by 11%) the crack width of UHSC-Beams with different longitudinal steel ratio (1.5%, %3%) and the same aspect ratio (1:2) decreased gradually by (20%).

4- In the experimental studies, for (G1) the ultimate load of UHSC-Beams with different longitudinal steel ratio (1.5%, %3%) and the (width : thickness) ratio of (1:3) increased gradually by (23%) the vertical deflection of UHSC-Beams with different longitudinal steel ratio (1.5%, %3%) and the same aspect ratio (1:3) decreased gradually by (48%) compared to (UHB30-1.5%) the crack width of UHSC-Beams with different longitudinal steel ratio (1.5%, %3%) and the same aspect ratio (1:3) decreased gradually by (50%).

5- Experimentally, for G2 (Fcu=120 MPa, and same (width: thickness) ratio of (1:2.5)), the achieved ultimate load for beams with different longitudinal tensile reinforcement ratio of (1%, 1.5%, 2%, 2.5%, and 3%) was increased gradually by (29%, 58%, 80%, and 123%), respectively, over the reference beam (UHB25-1%). Whereas, the final vertical deflection at mid-span of these beams decreased gradually by (7%, 13%, 38%, and 57%) compared to the reference beam (UHB25-1%..). Finally, the crack width of the same beams decreased gradually by (20%, 33%, 60%, and 80%), respectively, with reference to beam (UHB25-1%).

6- It was found experimentally & analytically that a ratio equal to double the minimum reinforcement percentage stated by ECP (201) can be considered as the minimum ratio for UHSC. It was noted that values lesser the stated could end-up with significant sudden increase in the tensile reinforcement strain as well as unacceptable deflection values. (as abeam 120UH25-1.4%.).

7-the reinforcement ratio of ECP is not suitable with UHSC beam with different d/w ratio because the reinforcement ratio of ECP caused very high deflection and crack width with low steel strain but 2-times the reinforcement ratio of ECP according to calculations and experimental studies was found controlled the deflection by 13% and crack width by 33%

8-it was found the best percentage of reinforcement steel ratio =2-times ECP because the tested beam with 1.4% steel have greater initial crack load than beam with 1% steel by 4-times

9-For beams with thickness to width ratio equal (2), it was clear that using the correct minimum reinforcement percentage (i.e. double that stated with the code) act upon reducing the crack width by 20% compared to beams having the minimum reinforcement ratio stated by the ECP. In addition, the deflection was also decreased 11% at concrete crushing strain (0.003). the 2-times minimum reinforcement ratio of the ECP decreased the crack width about 20% (AS beam UHB25-1% steel ratio by code with 120UH25-1.4% by double steel ratio by code) and decrease the deflection by (11%) at 0.003 concrete strain by enhance the longitudinal steel strain from steel strain by minimum reinforcement ratio of the ECP for depth to width ratio 2.

10-the 2-times minimum reinforcement ratio of the ECP decreased the crack width about 33% and decrease the deflection by (13%) at 0.003 concrete strain by enhance the longitudinal steel strain by 60% from steel strain by minimum reinforcement ratio of the ECP for depth to width ratio 2.5.

11-the 2-times minimum reinforcement ratio of the ECP is decrease the crack width about 50% and decrease the deflection by (47%) at 0.003 concrete strain by enhance the longitudinal steel strain from steel strain by minimum reinforcement ratio of the ECP for depth to width ratio 3.

12- According to experimental, it was found the Asmin covered the Mcr &Fctr for UHSC from lab by EXP. was 2-times the Asmin equation by ECP 201 for difference D/W= 2,2.5,3

13-According to calculations of Mcr by ECP 201 were found the Pcracking for sampls designed by Asmin ECP201 have a balanced section (brittle failure) and equal or less than by little percent as Pultimate but when samples were designed by 2-times Asmin have under reinforced sedction (ductile failure) ECP 201were found the Pcracking equal (50%-67%) Pultimate and when increase the reinforcement ratio increased Pultimate .

References

Acker P. and Behloul M., (2004), "Ductal Technology: a Large Spectrum of Properties, a Wide Range of Applications," Proceedings of the International Symposium on Ultra High Performance Concrete, Kassel, Germany, September 13-15, 2004, pp 11-24.

Alves M.F., Cremonini R. A. and Dal Molin D. C. C. (2004) "A comparison of mix proportioning methods for high-strength concrete" Cement and Concrete Composites Vol. 26, pp. 613-621.

Bornemann R. and Faber S. (2004) "UHPC with Steel- and Non-Corroding High-Strength Polymer Fibres under Static and Cyclic Loading" Proceedings of the International Symposium on Ultra High Performance Concrete, Kassel, Germany, September 13-15, 2004, PP. 673-682.

Bouzoubaa, N., Zhang M. H., Bilodeau A. and Malhotra V. M. (1998), "Laboratory-Produced High-Volume Fly Ash Blended Cements: Physical Roperties and Compressive Strength of Mortars", Cement & Concrete Research, Issue No. 11, Vol. 28, pp. 1555-1569.

Buitelaar P., (2004) "Heavy Reinforced Ultra High Performance Concrete" Proceedings of the International Symposium on Ultra High Performance Concrete, Kassel, Germany, September 13-15, 2004, PP. 25-35.

Cwirzen A., Penttala V. and Vornanen C., (2008) "Reactive Powder Based Concretes: Mechanical Properties, Durability and Hybrid Use with OPC", Cement & Concrete Research, Issue No. 10, Volume 38, pp. 1217-1226.

El-Dieb A. S., (2009) "Mechanical, Durability and Microstructural Characteristics of Ultra-High-Strength Self-Compacting Concrete Incorporating Steel Fibers" Materials and Design, Issue No. 10, Volume 30, pp. 4286-4292.

Elfgren L., Jonasson J.-E. and Ronin V., (2004) "High Performance Concretes with Energetically Modified Cement (EMC)", Proceedings of The International Symposium on Ultra High Performance Concrete, Kassel, Germany, September 13-15, 2004, pp. 93-102.

Federal Highway Administration (FHWA), (2006) "Materials Property Characterization of Ultra-High Performance Concrete" Publication No. FHWA-HRT-06-103, August 2006.

Fehling E., Bunje K., Schmidt, M. and Schreiber W., (2004) "Ultra High Performance Concrete Bridge across the River Fulda in Kassel – Conceptual Design, Design Calculations and Invitation to Tender" Proceedings of the International Symposium on Ultra High Performance Concrete, Kassel, Germany, September 13-15, 2004, pp. 69-76.