Structural Behavior of High Strength Self – Compacting Concrete Beams

A. A. Elsayed

Abstract:- This research presents an experimental and theoretical studies on the structural behavior of high strength self-compacted concrete I-beams. The main objective is to arrive the mode of failure of I beams which reinforced with and without web reinforcement. The experimental program presents the obtained results of tested eight high strength self compacting concrete I- beams specially reinforced to ensure a shear failure. All beams were tested simply supported along span 2400mm and subjected to four lines loadings until failure. The main variables were web thickness. The presence of web reinforcement in concrete beams increases its shear capacity and improves the ductility of the beam. Increasing the loading span to depth ratio decreased the failure load of concrete beams with web reinforcement, increasing the loading span to depth ratio decreased the failure load of concrete beams with web reinforcement, increasing (a/d) ratio from 2.4 to 2.9 led to a decrease in failure load by 33.3%. The effect of the studied variables are presented and discussed.

Keywords- High strength concrete; Self compacting concrete; Beams; Shear failure; Stirrups; Cracking.

I. INTRODUCTION

The use of high strength concrete (HSC) has increased considerably during the last decade, since it can be produced reliably using low water-cement ratios by using high-range water reducing admixtures. HSC is frequently used in beams, columns and precast elements and in structures where durability is an important design parameter. In Japan in the year 1988, self-compacting concrete (SCC) emerged on the scene and it has been the subject to numerous investigations in order to adapt it to modern concrete production [1]. Self compacting concrete (SCC) is an innovative concrete that does not require vibration for placing and compaction. It is able to flow under its own weight, completely filling formwork and achieving full compaction, even in the presence of congested reinforcement. Utilization of high strength self compacting concrete (HSSCC) is becoming widespread in the construction industry so that we need to examine the characteristic properties of HSSCC; accordingly the shear behaviour of HSSCC beams is presented in this paper. Previous research investigations have been reviewed [2-12], it was concluded that shear failure is difficult to predict accurately and in spite of many decades of experimental research and the use of highly sophisticated analytical tools, it is not yet fully understood. Furthermore, if a beam without properly designed shear reinforcement is overloaded to failure, shear collapse is likely to occur suddenly, with no advance warning of distress.

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A.A. Elsayed, Associate Professor, Department of Civil Engineering, Modern Academy for Engineering and Technology, Abd El-Aziz Hasan, Qism El-Khalifa, Cairo Governorate, Egypt.

This is in strong contrast with the nature of flexural failure. For typically under reinforced beams, flexural failure is initialed by gradual yielding of the tension steel, accompanied by obvious cracking of the concrete and large deflections, giving warning and providing the opportunity to take corrective measures. Most of the current shear procedures are based on tests carried on beams with compressive strength lower than 40 Mpa [13], code equations must be re-evaluated for this increase in compressive strength that high strength self compacting concretes achieve.

II. EXPERIMENTAL PROGRAM

This part presents a detailed description of the testing program carried out to meet the study objectives. The objectives of the program were:

a- To evaluate the behavior of HSSCC beams with various amounts of shear reinforcement.

b- To gain understanding on the effect of using self compacting concrete on the shear capacity of HSC beam type specimens.

c- To study the effect of loading the beams with variable span to depth ratios.

In the experimental program carried out at the Concrete Research Laboratory at Ain Shams University in Cairo, testing to failure of HSSCC beams with and without shear reinforcement was carried out. The evaluation of the shear capacity and behavior of these beams were done through the testing of eight (I-shaped) beams till failure occurred. The concrete compressive strength measured at test date was 54 MPa. All test specimens had some common characteristics for the purpose of establishing needed comparisons. The properties of the materials used in the construction of the test specimens are presented together with construction details, test setup, instrumentation, and test procedure.

2.1 MATERIAL PROPERTIES

The materials used in casting of the reinforced concrete specimens are Ordinary Portland cement, natural clean sand; coarse aggregate, Silica fume and a super-plasticizer. clean water was used for producing self compacting concrete and the water to powder ratio was 0.30 by weight in order to produce high strength concrete. Two types of steel reinforcement were used, the first type is mild steel of grade 240/350 MPa. The yield strength is 240 MPa and ultimate strength is 350 MPa. Mild steel was used mainly in stirrups. The second type of reinforcement is high grade steel reinforcement 360/520 having yield strength is 360 MPa and the ultimate strength is 520 MPa. The super plasticizer used in the concrete mix is GLENIUM C315 which is a product of BASF [14] chemical company is a unique third generation super plasticizer which has been developed primarily for the use in producing concrete with high durability and high performance. GLENIUM C315 complies with BS-5057 part 3 and EN934-2 and is compatible with all types of cement. The concrete mix proportions were chosen

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based on the results of several trial mixes, Table (1) shows the mix proportions used to produce high strength self compacting concrete.

2.2 PREPARATION OF TEST SPECIMENS

The dry coarse aggregate, cement and sand were first mixed for about one minute before adding half of the mixing water. After two minutes of mixing, the remaining mixing water and super plasticizer were added, Mixing was continued for another minute to achieve uniform distribution throughout the concrete mix. Before pouring of concrete a slump test (ASTM C 143-05) was carried out to verify the consistency of the mix and to determine its flow ability. The measured flow diameter was 710 mm. which indicates that this concrete mix is self compacting concrete according to the European guidelines for self compacting concrete [15].

Then concrete was casted in the custom made wooden moulds. Beams were demolded after 48 hrs from casting, covered with wet burlap, and stored under laboratory conditions for 28 days. In addition, twelve 150-mm cubes were cast from the concrete mix and tested for compressive strength after a water-curing period of 7&28 days, three 100 x 100 x 500 mm. prisms and three standard cylinders (300 x 150 mm) were cast with the main beams as control specimens and they were demolded the next day after casting. Custom designed wooden formwork was used to cast the specimens. Figure (1-a) shows the preparation of form work and insertion of reinforcing steel, while figure (1-b) shows pouring of concrete.

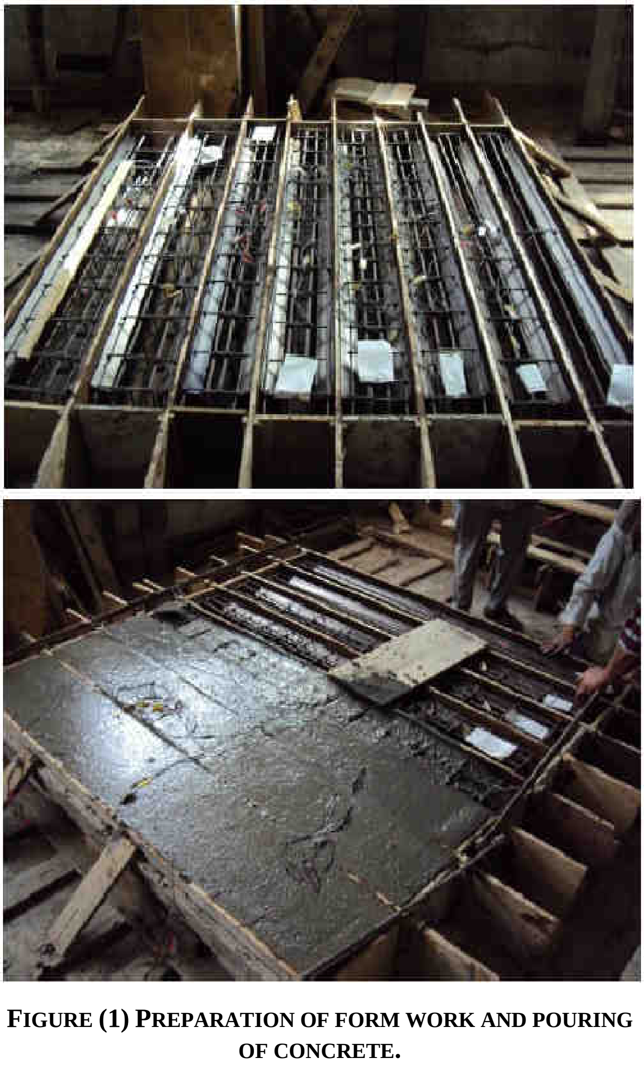


FIGURE (1) PREPARATION OF FORM WORK AND POURING OF CONCRETE.

TABLE (1) CONCRETE MIX PROPORTIONS TO PRODUCE HIGH STRENGTH SELF COMPACTING CONCRETE

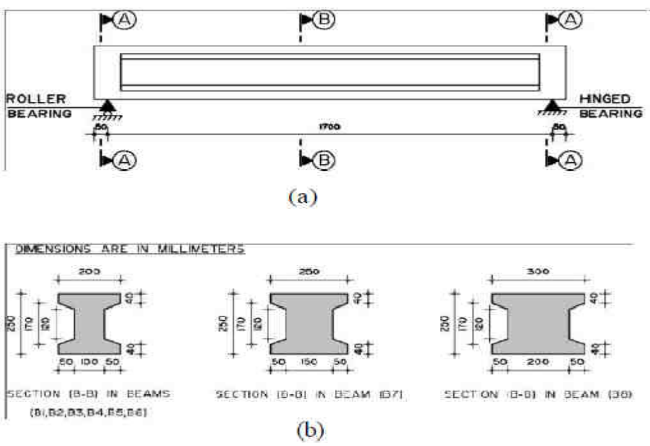
Proportions, Kg/m^3					
Cement	Sand	Dolomite	Silica Fume	Water	Super Plasticizer Lit/m^3
540	750	750	70	185	12

Figure (2) above shows the reinforced concrete beams before the testing phase after removing the wooden formwork, while Table (2) shows the details of beam specimens. The concrete dimensions of beam specimens are shown in Figure (3). All the longitudinal top and bottom reinforcement for all beams was identical; Figure (4) shows one of the studied beams showing the distribution of longitudinal bars throughout the beam cross section, while the spacing of stirrups for each beam is shown in Table (2).

TABLE (2) DETAILS OF BEAM SPECIMENS.

Beam No.	Web Reinforcement /m	Web width (cm.)	loading span/depth ratio
B1	-------	10	2.4
B2	6Ø6	10	2.9
B3	7Ø6	10	2.4
B4	6Ø6	10	2.4
B5	8Ø6	10	2.4
B6	-------	10	3.4
B7	6Ø6	15	2.4
B8	6Ø6	20	2.4

FIGURE (2) REINFORCED CONCRETE SPECIMENS AFTER REMOVING FORMWORK.



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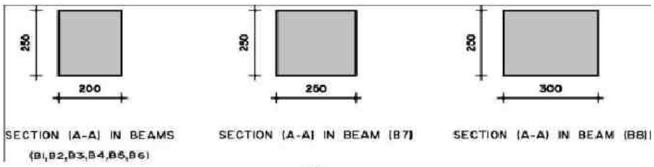


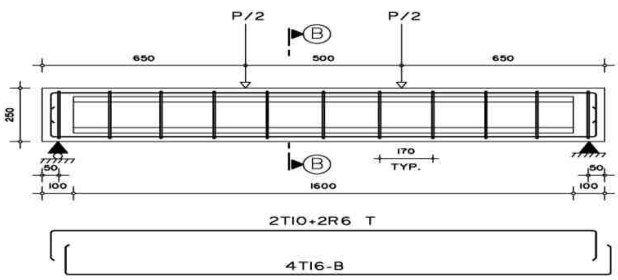
(C)

FIGURE (3) CONCRETE DIMENSIONS OF BEAM SPECIMENS

2.3 INSTRUMENTATION AND TESTING PROCEDURE

The strain in the reinforcing steel were measured using TML electrical strain gauges from Tokyo Sokki Kenkyujo Co., Ltd. type FLA-6-11-1L, with gauge resistance of 120 ohms and a gauge length of 6 mm. All beams were loaded to failure by means of vertical hydraulic jacks using a steel distribution beam with special bearing assemblies on the top face of the specimen. Linear Variable Differential Transducer (LVDT) was used to measure deflection through a computer-controlled data acquisition system. This system was used to record measurements at fixed time intervals. Measurements included load from the load cell, deflection from LVDT and the strains at bottom bars, top bars and stirrups from the electrical strain gauges, concrete strain measurements were done by using dimic points as shown in Figure (5) by measuring the initial distance between points and the distance between points at each load increment.

Before each test the beam specimen was placed on the supporting frame and the locations of applied loads and locations of LVDT's were adjusted as shown in Figure (4) and (5) except that the shear span shown in Figure (4) was variable according to the beam number. The LVDT's and the strain gauges were attached to the data acquisition system and their initial readings were recorded before loading of the specimen. The cracks at each incremental load were traced using a permanent marker and widths of selected cracks were measured when the load reached its steady state. At each load step dial gauge measurements were recorded and photographs of the crack propagation were taken, and the test was terminated when the beam was fractured or when extensive deformation was observed.



Beam B (7)

Section B-B

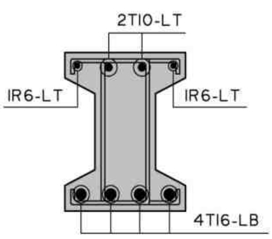


FIGURE (4) ELEVATION IN CONCRETE

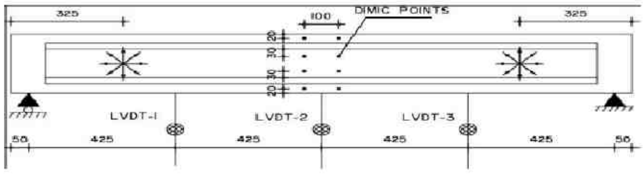


FIGURE (5) LOCATION OF DIMIC POINTS AND LVDT IN A TYPICAL BEAM.

FIGURE (6) TEST SETUP.

III. FAILURE MODES

Beams B1 and B5 in group (G1) failed in a shear compression failure, while beams B3 and B4 failed by a shear failure. Failure of beams in this group is considered brittle failures but in general beam B1 without web reinforcement showed a more brittle failure than beams with web reinforcement. In group (G2) beams B1 and B6 in this group both failed in a brittle failure. Beam B1 failed in a shear compression failure while beam B6 failed by a shear failure. Although both beams did not have web reinforcement it was noticed that beam B6 failed in an extremely brittle failure than beam B1 due to the increased (a/d) ratio. In group (G3) beams B2 and B4 failed by a shear failure and no crushing of the compression zone was noticed for beams in this group. Web cracks in beam B2 were wider than those in beam B4 and beam B2 showed a more brittle failure than beam B4. Beams B4, B7 and B8 in group (G4) were studied. Beam B4 failed in a shear failure while beams B7 and B8 failed in a shear compression failure and it was noticed in beam failures in this group that beams with a wider web width failed by having wider web cracks than the other beams. All beams in this group were characterized by having a less brittle failure than beams of other groups and beams that had a wider web width did have a more ductile failure than other beams.

In general failure of all of the studied beams were characterized by having a very smooth surface of failure which indicated brittle failure and most of failure cracks propagated through the aggregates in the cement paste and not around it which indicates that the cement paste is as strong as the coarse aggregate zone and this is a typical failure in high strength concrete. The crack patterns of the tested beams are shown in Figure (7).

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FIGURE (7) CRACK PATTERNS IN TESTED BEAMS.

IV. TEST RESULTS AND DISCUSSION

The tested beams in this study were divided into four groups as shown in Table (3), group (G1) was for studying beams with variable web reinforcement, groups (G2) and (G3) study the shear loading span (a/d) for beams with and without web reinforcement, while group (G4) is for studying the effect of variable web width on the shear behaviour of beams.

TABLE (3) STUDIED FACTORS

Group	Factors		Beam No.
G1	Web RFT. (ρ_{st})		B1&B3&B4&B5
G2	Loading span to depth ratio	Beams without web rft.	B1 & B6
G3		Beams with web rft.	B2 & B4
G4	Web Width (bw)		B4&B7&B8

TABLE (4) PROPERTIES OF STUDIED BEAMS

Beam no.	f_{cu} (MPa)	F_y stirrups (MPa)	b_w (mm.)	d (mm.)	Flange width (mm.)	Web rft. %	Long. rft. %
B1	54	235	100	220	200	0	3.21
B2	54	235	100	220	200	0.33	3.21
B3	54	235	100	220	200	0.40	3.21
B4	54	235	100	220	200	0.33	3.21
B5	54	235	100	220	200	0.47	3.21
B6	54	235	100	220	200	0	3.21
B7	54	235	150	220	250	0.22	2.14
B8	54	235	200	220	300	0.16	1.60

Table (4) consists of properties of the tested beams including the compressive strength at the age of 28 days, the width and depth of the beams, flange width, transverse (web) reinforcement ratio, and the longitudinal bottom reinforcement ratio.

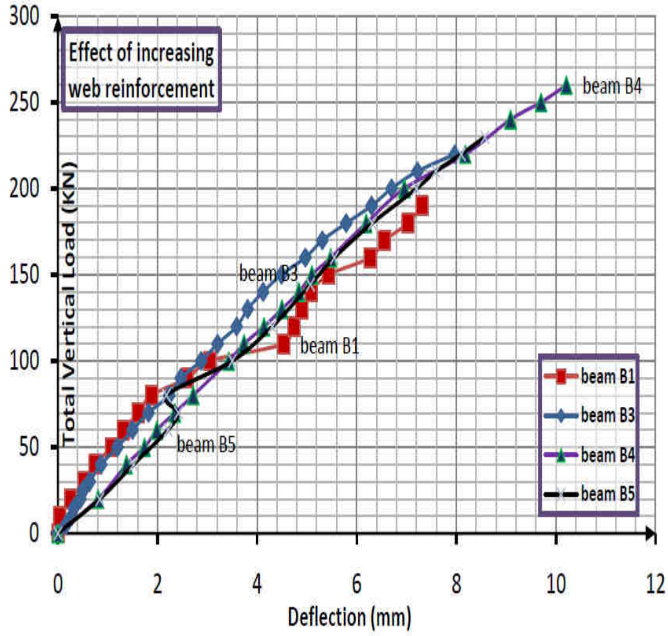

FIGURE (8) LOAD DEFLECTION CURVES FOR GROUP G1

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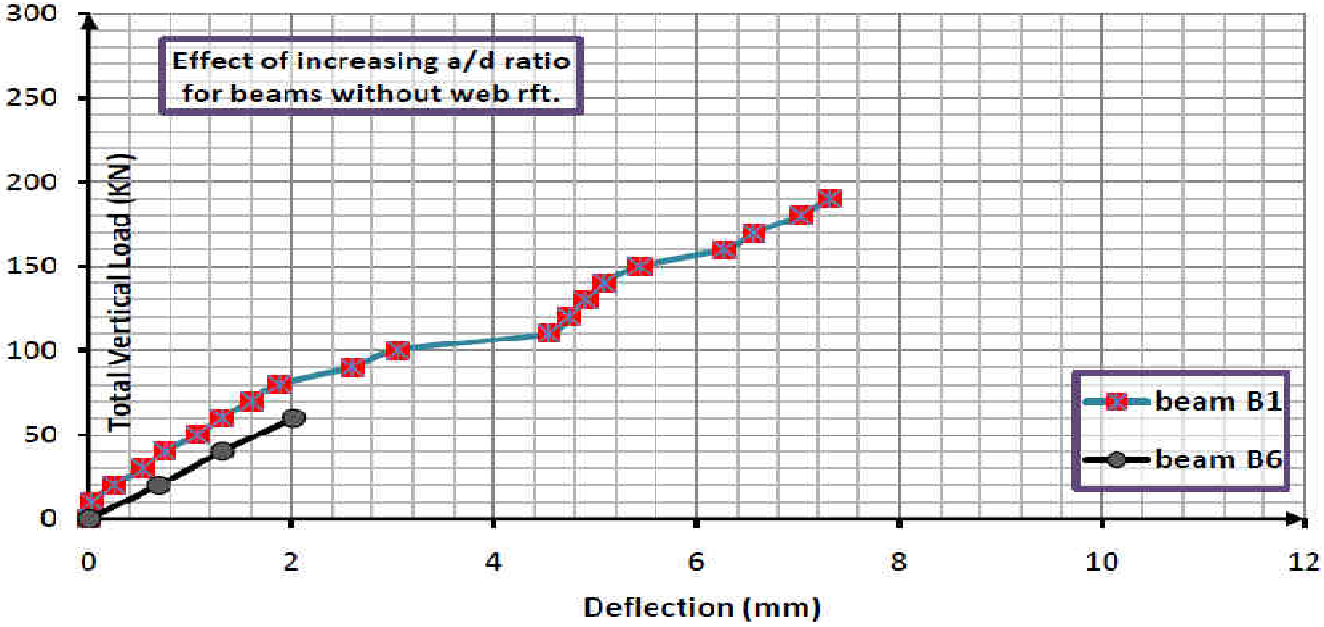

FIGURE (9) LOAD DEFLECTION CURVES FOR GROUP G2

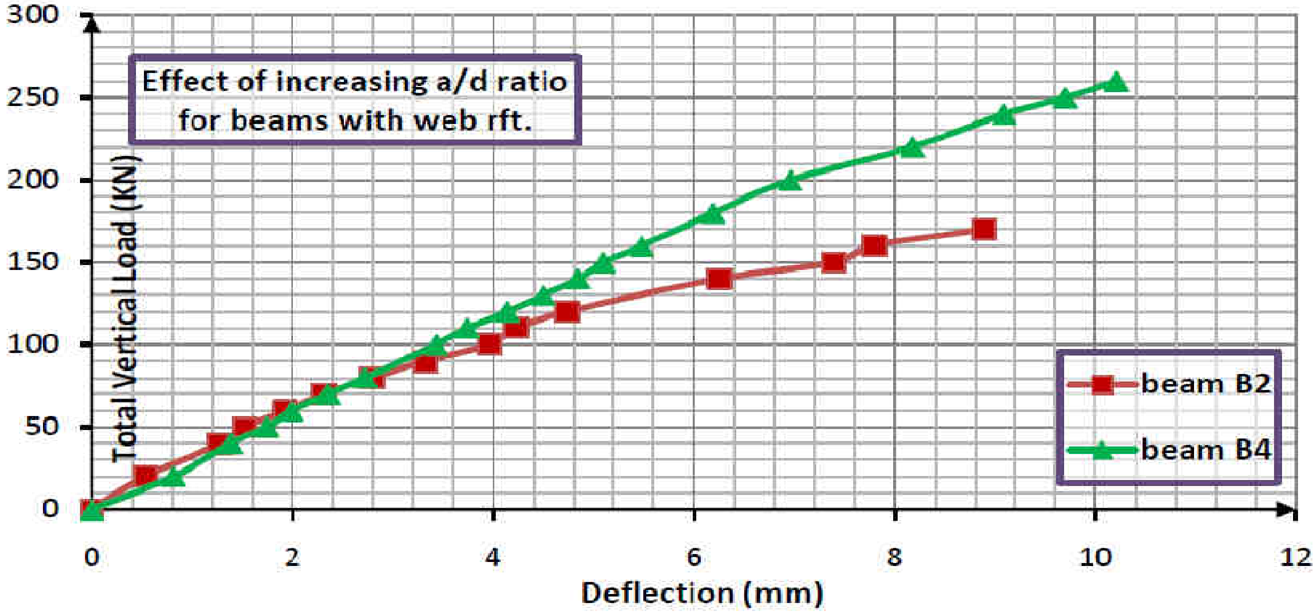

FIGURE (10) LOAD DEFLECTION CURVES FOR GROUP G3

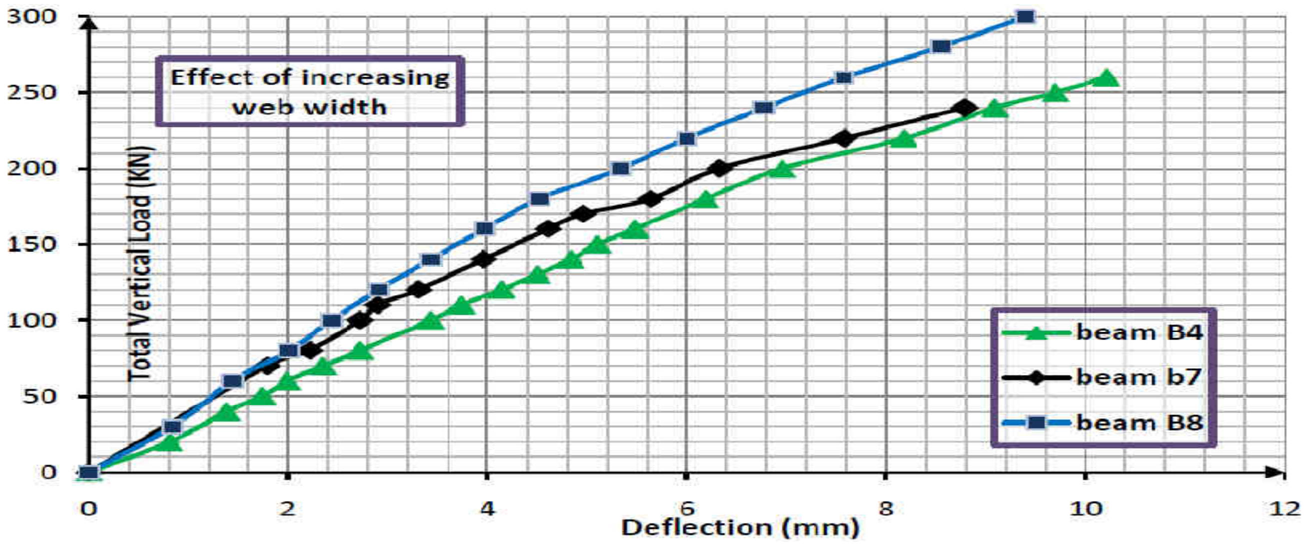

FIGURE (11) LOAD DEFLECTION CURVES FOR GROUP G4

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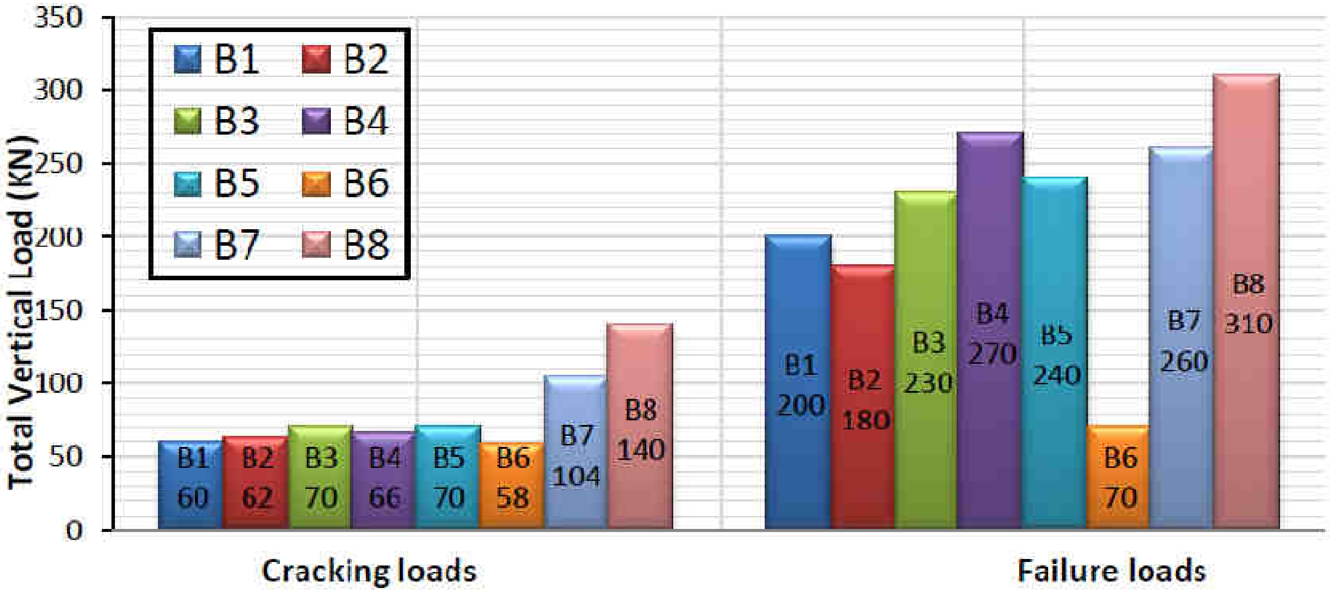

FIGURE (12) COMPARISON BETWEEN CRACKING AND ULTIMATE LOADS OF TESTED BEAMS

Figures (8) to (11) represent the load versus deflection curves for the studied groups; the studied beams in each group and the point of comparison are indicated on each graph. Figure (12) shows the cracking loads and failure loads of each of the studied beams, these loads are the readings from the load cell before being divided by means of the distributing beam into two concentrated load. Comparing the results of beams in group (G1); B1 (ρw = 0%), beam B3 (ρw = 0.40%), beam B4 (ρw = 0.33%) and beam B5 (ρw = 0.47%) show that by increasing the amount of web reinforcement leads to an increase in the angle (slope) of the crack propagation. It was also found that increasing amount of web rft. led to a decrease in the difference between stiffness at the cracking stage and ultimate stage. The cracking loads of the beams were not affected significantly with the increase in web reinforcement while the failure load was affected by the increase in web reinforcement; increasing web rft. led to an increase in failure load. From the load-deflection curves of beams in Group 1, it can be seen that there was an improvement in the stiffness of beam B3, B4 and B5 due to the presence of web reinforcement in these beams. Beam B1 had lower deflection values than the other beams in this group until reaching the cracking stage than it started to show larger deflection values than the other beams at the same loading values. Beams B4 and B5 nearly had the same load-deflection curve except at the point of failure where beam B4 failed after the failure of beam B5, the load-deflection curves for beams in this group are shown in Figure (8).

Comparing the results of beams in group (G2); beam B1 (a/d=2.4) and beam B6 (a/d=3.4) show that by increasing the loading span to depth ratio leads to a decrease in the failure load and increases the brittle behaviour of the beam, as the failure of the beam is more sudden and without any warning. It is noticed from Figure (9) that beam B6 showed larger deflection than beam B1 up to failure, and the deflection of beam B6 was greater than beam B1 at cracking stage. The deflection of beam B6 could not be measured due to the quick sudden failure of this beam.

From the results of beams B4 (a/d=2.4) and beam B2 (a/d=2.9) in group (G3) show that by increasing the loading span to depth ratio leads to a decrease in the failure load and increases the brittle behaviour of the beam, but the difference in this type of failure and that of beams without web reinforcement is that the presence of shear reinforcement prevents the beam from sudden failure thus increasing the shear strength of the beam.

Studying the cracking patterns between B2 and B4 in group (G3), beam B2 showed more flexural and shear cracks than those in beam B4, the flexural cracks extended higher than those of beam B4 due to the increased loading span to depth ratio of beam B2. Both of the tested beams nearly cracked at the same load accordingly the cracking loads were not much affected by the change in the loading span to depth ratio, Figure (10) represents the loading –deflection curves for beams in group (G3).

For beams in group (G4) beam B8 showed the largest deflection at cracking stage, while beam B4 showed the least deflection value at cracking stage. At failure stage beam B4 showed the largest deflection, while beam B7 showed the least value at failure load. As an overall behaviour it can be noticed from Figure (11) that beam B4 showed greater deflection than beam B7, while beam B7 showed greater deflection than beam B8 throughout the loading of the beams after the cracking stage.

The deflection of beams in this group shows logical results, beam B8 had the least deflection after the cracking stage due to the increased moment of inertia of the section as this beam had a web width of 200 mm., while beam B7 showed higher deflection values than those of beam B8 due to the decrease in web width of this beam which is 150 mm. while beam B4 with the smallest web width showed the highest deflection during loading after the cracking stage.

V. COMPARISON OF TEST RESULTS WITH DIFFERENT DESIGN APPROACHES.

In order to check the accuracy of the current code provisions, three design codes are reviewed; the Egyptian Code of Practice (ECP-203-2007)[16], the American Concrete Institute Building Code Requirements for Structural Concrete and Commentary (ACI Committee 318, 2008)[17] and the British Standard Code of Practice for Design and Construction (BS 8110 – 97)[18]. A comparison between shear values obtained from the experimental testing

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of beams and the shear values calculated from code equations of the studied codes.

VI. CONCLUSIONS AND SUMMARY OF RESULTS.

- High strength self compacting concrete beams without web reinforcement showed an extremely brittle behaviour when failing in shear, although beams with web reinforcement had also a brittle failure but the presence of stirrups adjusted the failure mode of the beams to show excessive cracking as a warning before failure.
- Increasing the loading span to depth ratio decreased the shear capacity of the concrete beams without web reinforcement and increased the brittle behaviour of this type of concrete accordingly failure of beam with high loading span to depth ratio was sudden and accompanied with a loud sound and usually was formed of one failure crack, while failure of beams with smaller (a/d) ratios was more explosive due to failure in the compression strut formed between point of loading and the support. Accordingly increasing the loading span to depth ratio from 2.4 to 3.4 led to a decrease in the failure load of the beam by 65 %.
- Increasing the loading span to depth ratio decreased the failure load of concrete beams with web reinforcement, increasing (a/d) ratio from 2.4 to 2.9 led to a decrease in failure load by 33.33%.
- Increasing the web width in the I-beams increased the shear strength of high strength self compacting concrete beams with web reinforcement. In this study it was found that increasing the web width by 50 to 100 mm. led to an increase in failure load by 12.90 to 16.12%, while the cracking loads of these beams increased by 25.70 to 36.53%.
- It was found that the stiffness of beams without web reinforcement varied greatly from the cracking stage to the failure stage and this difference in stiffness reached 40.29%, while providing web reinforcement led to improving the this variance in stiffness between the two stages in beam B5 in the study this difference reached 3.19%.
- From the theoretical study, it was found that all studied design codes; ACI [318-2008], ECP [203-2007] and BS [8110-97] were conservative in calculating the shear capacity of high strength self compacting concrete beams.
- ECP [203-2007] code equations showed better prediction than those of the ACI [318-2008] code equations, where the mean value of the experimental result to the predicted one in the ACI equation was 3.16 and a standard deviation of 0.94, while the equations stated in the Egyptian code [ECP] gave a mean value of 3.62 and a standard deviation equals to 1.24.

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