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Shear Behavior of Self Compacting Concrete Deep Beams

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Abstract :

This study deals with shear behavior of self compacting concrete (SCC) deep beams. The experimental work includes testing eight reinforced concrete simply supported deep beams cast using SCC. All tested beams have dimensions of 100×330×1050 mm and have been subjected to two point loads. The parameters considered are shear span to effective depth ratio (a/d), concrete compressive strength (f'_c) and vertical web reinforcement ratio (r_v). Test results indicated that the increase in (a/d) ratio from 0.6 to 1 leads to decreases in cracking and ultimate shear strengths by average ratios of 28.6 % and 23.3 % respectively. Increasing (f'_c) from 32.84 MPa to 64.65 MPa leads to increases in the cracking and ultimate shear strengths by average ratios of 11.7 % and 38.8 % respectively. Increasing the vertical web reinforcement ratio (r_v) from 0.25 % to 0.57 % leads to an increase in the ultimate shear strength by average ratio of 10.1 %. The analytical work includes derivation of new method for predicting ultimate shear strength of SCC deep beams depending on modified strut and tie model with adoption of a circular failure interaction relation. The new method gives high agreement with the experimental results by comparison with the ACI-2011 Code method.

Keywords: self compacting concrete, shear, deep beams, steel fibers, high strength.

سلوك القص لعتبات الخرسانة ذاتية الرص العميقة

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الخلاصة :

تتناول هذه الدراسة سلوك القص للعتبات العميقة المصنوعة من الخرسانة ذاتية الرص. يتضمن البرنامج فحص ثمانية من العتبات الخرسانية المسلحة العميقة ذات الاستناد البسيط المصبوبة باستخدام الخرسانة ذاتية الرص. جميع العتبات كانت بإبعاد 100×330 × 1050 ملم ومعرضة لحملين مركزين. المتغيرات المعتبرة في هذه الدراسة هي نسبة فضاء القص الى العمق الفعال (a/d)، مقاومة انضغاط الخرسانة (f'_c) ونسبة تسليح القص العمودي (r_v). نتائج الفحص اظهرت ان الزيادة في نسبة (a/d) من 0.6 الى 1 تؤدي إلى نقصان في مقاومة التشقق والمقاومة

القصوى بمعدل نسب % 28.6 و % 23.3 على التوالي. الزيادة في (f'_c) من 32.84 MPa الى 64.65 MPa يؤدي إلى الزيادة في مقاومة التشقق والمقاومة القصوى بمعدل نسب % 11.7 و % 38.8 على التوالي. زيادة نسبة تسليح القص العمودي (r_v) من % 0.25 إلى % 0.57 يؤدي إلى زيادة في مقاومة القص القصوى بمعدل نسب % 10.1. الجزء التحليلي من هذه الدراسة تضمن اشتقاق طريقة جديدة لاحتساب مقاومة القص القصوى لهذا النوع من العتبات بناءاً على الطريقة المطورة لتقنية (strut and tie model) باعتماد علاقة خط الفشل الدائري. الطريقة الجديدة تعطي توافق عالي مع النتائج العملية بالمقارنة مع الطريقة المعتمدة من قبل (ACI-2011 Code).

1. Introduction

Reinforced concrete deep beams are structural members having depth much greater than normal in relation to their span, while the thickness in the perpendicular direction is much smaller than either span or depth[1]. These members are used in many structural applications such as diaphragms, water tanks, foundations, bunkers, shear walls, girders used in multi story buildings to provide column offsets, and floor slabs under horizontal loads[1,2]. Usually, deep beams have narrow width and contain congested shear reinforcement. Therefore, the conventional concrete does not flow well when it travels to the web and does not completely fill the bottom part. This results in many problems in concrete such as, voids, segregation, weak bond with reinforcement bars and holes in its surface. Therefore, the self compacting concrete (SCC) is very appropriate type for casting these members.

Self compacting concrete, provides distinct advantages over conventional vibrated concrete due to liquid nature such as: elimination of above mentioned problems, low noise level in construction, faster construction and improving quality and durability, no need to vibration where it is able to fill all spaces in the formwork and passes through reinforcing bars by its own weight [3,4].

The difference in some properties between the conventional vibrated concrete and the self compacting concrete requires necessity to investigate the behavior and capacity of structural members constructed using this type of concrete. Therefore, the behavior of deep beams made using SCC is experimentally investigated in this research work. Because of the lesser amount and smaller maximum size of coarse aggregate used in SCC compared with conventional vibrated concrete, one can expect that the shear strength of deep beams made by SCC is lesser than that carried out by deep beams made using conventional vibrated concrete, where the interlock mechanism of coarse aggregate is weaker which represents an important part of the total shear strength parts for these members. But the well self compaction and regularity of microstructure of this type of concrete reduce the weaken positions in it and may lead to an increase in its efficiency to resist the shear stresses.

2. Experimental Program

The experimental program consists of testing eight simply supported deep beams constructed using self compacting concrete. All beams have the same dimensions and flexural reinforcement. They have an overall length of 1050 mm, a width of 100 mm and a height of

330 mm. In this study three parameters are considered: shear span to effective depth ratio (a/d) , concrete compressive strength (f′c) and vertical shear reinforcement (ρ_v).

The specimens are divided into four groups (A , B , C , and D). These groups are classified according to shear span to effective depth ratio (a/d) and concrete compressive strength (f′c) values. Group (A) relates to small value of shear span to effective depth ratio (a/d = 0.6) and normal concrete compressive strength (f′c < 41 MPa) according to ACI 363R[5]. Group (B) relates to larger value of shear span to effective depth ratio (a/d = 1) and normal concrete compressive strength. Group (C) relates to a/d = 0.6 and high concrete compressive strength (f′c > 41 MPa)[5] while group (D) relates to a/d = 1 and high concrete compressive strength. Each group involves two different beams, where the first beam of each group have approximately minimum vertical reinforcement ratio according to ACI318M-2011[6] provisions (ρ_v = 0.0025). The second beam of each group have a high vertical reinforcement ratio (ρ_v = 0.0057). **Table (1)** shows details of all eight beams with their related parameters. The normal strength SCC will be denoted by (NSCC) and high strength SCC will be denoted (HSCC)

Table (1) Details of tested beams and research parameters

Group	Beam design-ation	Total Length mm	Cross Section Dimensions mm	Conc. Type	a/d	Long. Reinf.	Vertical shear reinf.
A	A1	1050	100 × 330	NSCC	0.6	2ϕ16+ 2ϕ10	ϕ4 / 100
	A2	1050	100 × 330	NSCC	0.6	2ϕ16+ 2ϕ10	ϕ6 / 100
B	B1	1050	100 × 330	NSCC	1	2ϕ16+ 2ϕ10	ϕ4 / 100
	B2	1050	100 × 330	NSCC	1	2ϕ16+ 2ϕ10	ϕ6 / 100
C	C1	1050	100 × 330	HSCC	0.6	2ϕ16+ 2ϕ10	ϕ4 / 100
	C2	1050	100 × 330	HSCC	0.6	2ϕ16+ 2ϕ10	ϕ6 / 100
D	D1	1050	100 × 330	HSCC	1	2ϕ16+ 2ϕ10	ϕ4 / 100
	D2	1050	100 × 330	HSCC	1	2ϕ16+ 2ϕ10	ϕ6 / 100

3. Materials:

3.1 Cement

Ordinary Portland cement (type I) of Tasluja Factory is used in the present study. Test results of chemical composition and physical properties of the used cement tested by National Center for Construction Laboratories and Researches in Baghdad comply with the requirements of I.Q.S. No.5, 1984[7].

3.2 Fine Aggregate

Al-Ukhaider natural sand is used in concrete mix. Before using it, the sieve analysis is performed at Material Laboratory in Engineering College of Al- Mustansiriya University to ensure its validity for mixing. The fineness modulus , depending on this analysis, is 2.78. The sieve analysis results of the sand comply with the limits of the Iraqi Specification No.45/1984[8].

3.3 Coarse Aggregate (Gravel)

Crushed gravel of maximum size 10 mm brought from Al-Niba'ee region is used. Before using it, the sieve analysis is performed at Material Laboratory in Engineering College of Al-Mustansiriya University to ensure its validity for mixing and choosing the primary proportions of mix materials. The grading of this aggregate conforms to the Iraqi specification No.45/1984[8].

3.4 Limestone Powder

Limestone powder is locally named “Al-Gubra” brought from Al-Mousel district and has been used as a filler for concrete production for many years. The particle size of the limestone powder is less than 0.125 mm, which satisfies EFNARC 2002[9] recommendations.

3.5 Super Plasticizer

In this work, the super plasticizer used is known commercially as "GLENIUM51". It is a new generation of modified polycarboxylic ether. It is compatible with all Portland cements that meet recognized international standards. Super plasticized concrete exhibits a large increase in slump without segregation. However, this provides enough period after mixing for casting and finishing the concrete surface.

3.6 Steel Reinforcing Bars

Deformed steel bars are used in this work with nominal diameters of 16 mm and 10 mm for longitudinal reinforcement in tension side (bottom side) and plain bars of diameter 4 mm are used for longitudinal reinforcement in compression side (top side) while deformed bars of 4 mm and 6 mm are used as vertical shear reinforcement. Tensile tests of steel reinforcement are carried out at the laboratory of Materials at the College of Engineering in AL-Mustansiriya University to determine the average yield stress and the ultimate stress. The test results are listed in **Table (2).** Steel reinforcing cages are shown in **Figure (1).**

Table (2) Properties of reinforcing steel bars

Nominal bar diameter (mm)	Bar area (mm^2)	Yield stress (MPa)	Ultimate stress (MPa)	Elongation at ultimate stress (%)
16	201	671	831	6.6
10	78.5	650	807	9.7
4	12.6	406	534	3.4
6	28.3	394	558	3.7
4 plain	12.6	413	521	3.1

Fig .(1) Steel reinforcement cage used for tested beams

4. Concrete Mix Proportions

To determine mix proportions for different types of concrete adopted in this study, the tables of mix proportion suggested by Al-jadiri**[10]** in her research carried out in 2008 is adopted with some modifications after performing many trial mixes. **Table (3)** gives the final quantities by weight of materials used in preparation of self compacting concrete per cubic meter for the two mixes adopted in this work.

Table (3) Proportions of SCC mixes per cubic meter

Mix type	Mix name	Cement (kg)	Limestone powder (LSP) (kg)	Water (liter)	Sand (kg)	Gravel (kg)	Super plasticizer (liter)
Normal strength concrete	NSCC	400	170	190	797	767	7.5
High strength concrete	HSCC	550	50	165	855	767	20

5. Tests on Fresh Self Compacting Concrete

In this work, consideration of concrete mix as a self compacting concrete is verified by three standard tests: Slump flow, $T_{50\ cm}$ slump flow and L-box as shown in **Figures (2)** and **(3).**

Fig .(2) Spreading concrete in Slump Flow test of SCC

Fig .(3) Flowing of concrete in horizontal section in L-box test of SC

6. Mixing

The procedure of mixing is stated as follows:

1. The fine aggregate is added to the mixer with 1/3 quantity of water and mixed for 1 minute.
2. The cement and limestone powder are added with another 1/3 quantity of water. Then, the mixture is mixed for 1 minute.
3. The coarse aggregate is added with the last 1/3 quantity of water and 1/3 dosage of super plasticizer, and the mixing time lasts for 1½ minutes then the mixer is left for 1/2 minute to rest.
4. Then, the 2/3 of the leftover of the dosage of super plasticizer is added and mixed for 1½ minutes.
5. The concrete is then discharged for performing fresh properties and casting.

7. Tests and Measurements of Deep Beams

All beams were tested using a hydraulically universal testing machine of 3000 kN capacity under monotonic loads up to ultimate load at the Structural Laboratory of the College of Engineering of Al-Mustansiriya University. Vertical deflections are measured at deep beam midspan using digital gauge of (0.01 mm) accuracy. Loading was applied at increments of 10 kN. At each load stage the deflection readings at the midspan of beam were recorded. When the first crack appeared, the load corresponding it was recorded.

8. Fresh SCC Properties Results

Table (4) illustrates the results of these three tests that carried out on SCC mixes and the comparisons with the standard limitations are also presented.

Fig .(4) Digital gauge position

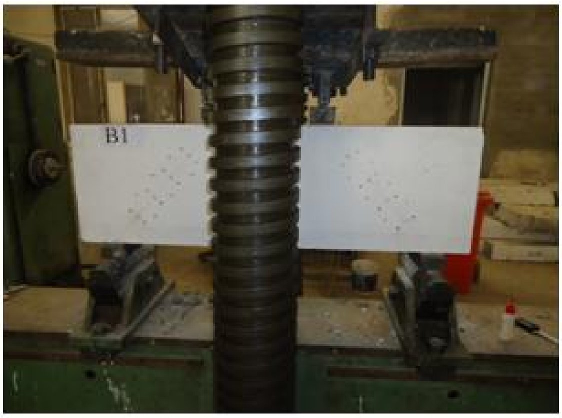

Fig .(5) Deep beam inside testing Machine

Table (4) Tests results of fresh properties for SCC

Mix name	Slump flow (mm)	$T_{50\ cm}$ (sec)	L – box (H_2/H_1)
NSCC	770	2.5	1
HSCC	730	4	0.92
Limits of EFNARC[9]	650-800	2-5	0.8-1

9. Hardened SCC Mechanical Properties Results

Table (5) shows test results of mechanical properties obtained for the two mixes. These properties are concrete compressive strength (f'_c), splitting tensile strength (f_t), modulus of rupture (f_r) and modulus of elasticity (E_c). Each value presented in this table represents the average value of three specimens.

Table (5) Tests results of mechanical properties for hardened SCC

Mix name	f'_c (MPa)	f_t (MPa)	f_r (MPa)	E_c (MPa)
NSCC	32.84	3.12	4.41	24897
HSCC	64.65	4.56	6.80	35287

The concrete mixes give normal compressive strength about 32.84 MPa while high compressive strength about 64.65 MPa. This means that good design of mixes can achieve the requirements of both types of concrete: NSCC and HSCC. The increase in (f'_c) from 32.84 MPa to 64.65 MPa (97% increase) leads to increases in (f_t) from 3.12 MPa to 4.56 MPa (46% increase), in (f_r) from 4.41 MPa to 6.8 MPa (54% increase) and in (E_c) from 24897 MPa to 35287 MPa (42% increase).

10. Test Results of SCC Deep Beams

Table (6) summarizes the results of first cracking load (P_{cr}) and ultimate load (P_u) for all tested beams together with their modes of failure.

Table (6) Tests results of SCC deep beams

Beam name	a/d	f′c (MPa)	% ρ_v	P_{cr} kN	P_u kN	Mode of shear failure
A1	0.6	32.84	0.25	165	485	Diagonal splitting
A2	0.6	32.84	0.57	180	535	Diagonal compression
B1	1	32.84	0.25	125	370	Diagonal splitting with crushing of nodal zone
B2	1	32.84	0.57	135	425	Diagonal splitting
C1	0.6	64.65	0.25	195	695	Diagonal splitting
C2	0.6	64.65	0.57	190	740	Diagonal splitting
D1	1	64.65	0.25	140	520	Diagonal splitting with crushing of nodal zone
D2	1	64.65	0.57	120	565	Diagonal splitting

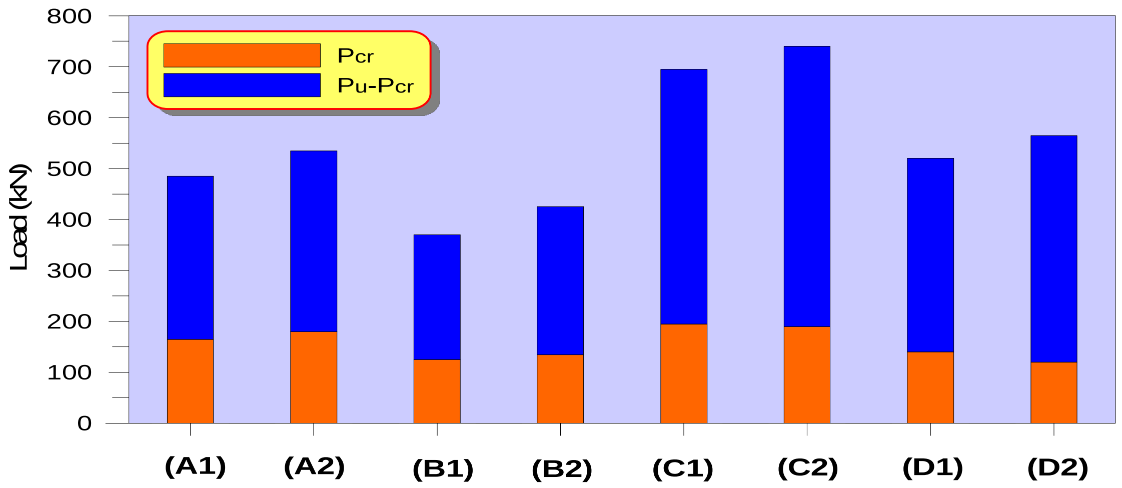

Fig .(6) Cracking and ultimate loads for all SCC beams

11. Behavior of SCC Deep Beams

Figures (7), (8) and **(9)** show the crack patterns for some SCC deep beams after testing . At low load levels, all the tested beams behaved in an elastic manner where no defects in their structure and the cracks did not appear at any place and the deflections at midspan are small and proportional to the applied load. Generally, the first diagonal crack (shear crack) appears at the middle third of the diagonal region bounded by load and support positions at a loading level ranges between 21% and 34 % of the ultimate load. The first flexure crack is observed in the lower part of the beam at the middle region between load positions. As the load is further increased, the inclined cracks expand and extend toward the support and load positions, also new cracks form parallel to the first crack and new cracks form near support.

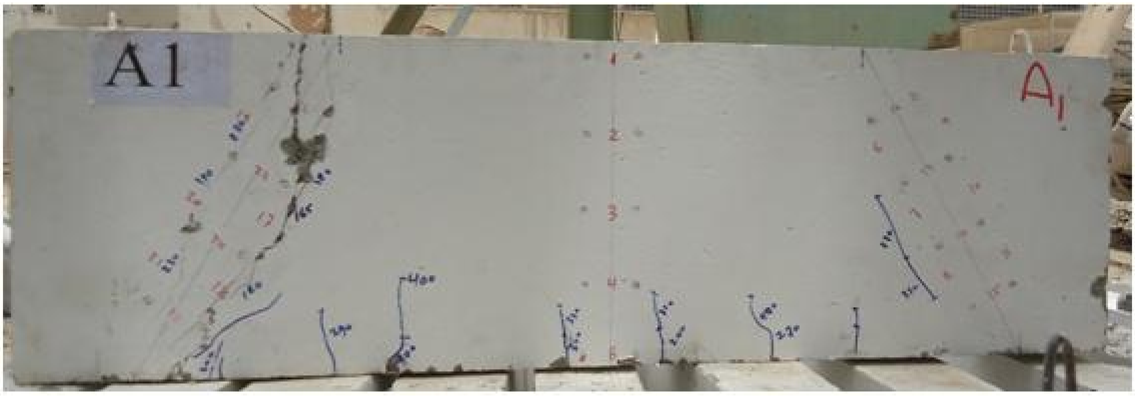

Fig .(7) Crack pattern for beam (A1)

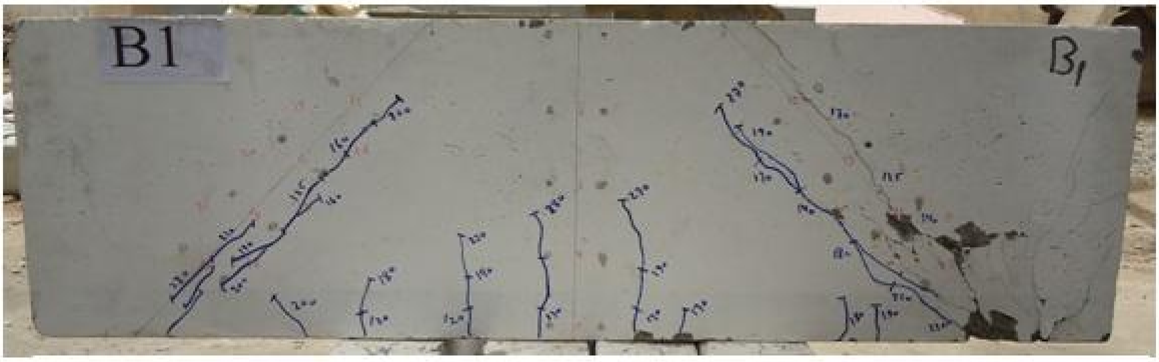

Fig .(8) Crack pattern for beam (B1)

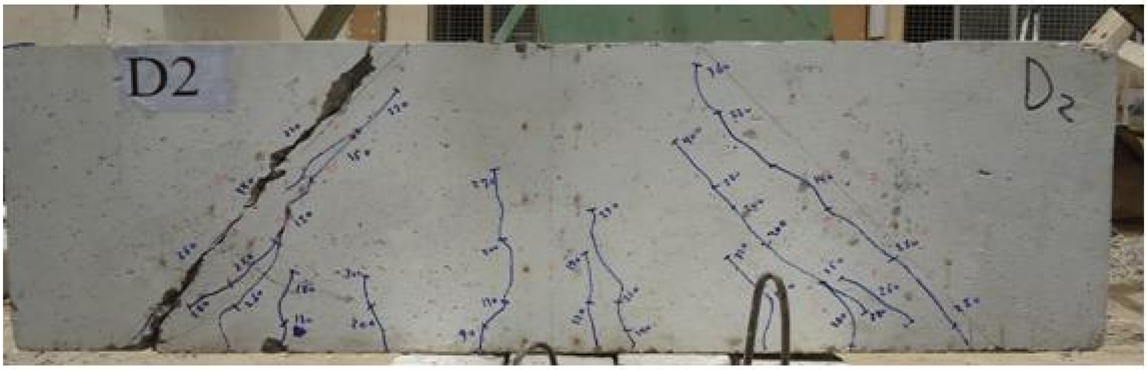

Fig .(9) Crack pattern for beam (D2)

Eventually, the diagonal cracks are many and one or more of these cracks might penetrate into the compression zone at the loading position. Failure occurs by splitting the beam into two parts approximately along the line joining the edge steel blocks at the support and loading positions (diagonal splitting mode). Also the stirrups intersecting the splitting line are yielded or ruptured. In some beams such as beam (A2) the failure takes place before the main cracks penetrate into the compression zone by crushing of concrete between these cracks at the strut joining the load and support positions (diagonal compression failure) because of high compression stresses in this strut.

The different mode of failure of beam (A2) was due to the use of 6 mm diameter bars of stirrups instead of 4 mm bars, where the inclined cracks do not penetrate into the compression zone due to presence of these stiff bars. Also there is a simple difference in mode of failure of beam (B1) where its splitting was accompanied by crushing of nodal zone at support region.

The splitting lines for beams of groups B and D are more pronounced than those of beams of group A and C because of higher tension stresses due to higher value of (a/d) ratio.

At failure, number and length of flexural cracks occurring in beams of group B and D are more than these in beams of group A and C because the higher bending moment due to higher value of (a/d).

12. Effect of Shear Span to Effective Depth Ratio (a/d)

Effect of (a/d) on cracking and ultimate loads and the ratio between them for all tested beams are detailed in **Table (7).** The reduction in cracking load due to increasing (a/d) ratio ranges from 24.2 % to 36.8 % (average of reduction is 28.6 %). The reduction occurring in HSCC beams is slightly larger than that occurring in NSCC beams. The reduction becomes larger as (ρ_v) increases. The reduction in the ultimate load due to increasing the (a/d) ratio ranges from 20.6 % to 25.2 % (average of reduction is 23.3 %). The reduction occurring in HSCC beams is slightly larger than that occurring in NSCC beams. Also, the reduction becomes smaller as (ρ_v) increase. The ratio between cracking and ultimate loads ranges from 0.26 to 0.34 for a/d = 0.6 while it ranges from 0.21 to 0.34 for a/d = 1, i.e., generally this ratio decreases as the (a/d) ratio increases.

Table (7) Effect of (a/d) ratio on cracking and ultimate loads

Strength type	r_v %	a / d = 0.6			a / d = 1			% Variation due to increasing (a/d)	
		P_{cr} kN	P_u kN	P_{cr} /P_u	P_{cr} kN	P_u kN	P_{cr} /P_u	DP_{cr} %	DP_u %
Normal Strength	0.25	**165**	**485**	**0.34**	**125**	**370**	**0.34**	**-24.2**	**-23.7**
	0. 57	**180**	**535**	**0.34**	**135**	**425**	**0.32**	**-25**	**-20.6**
High Strength	0.25	**195**	**695**	**0.28**	**140**	**520**	**0.27**	**-28.2**	**-25.2**
	0. 57	**190**	**740**	**0.26**	**120**	**565**	**0.21**	**-36.8**	**-23.6**

From the **Figure (10),** it is clear that the increase in the (a/d) ratio significantly increases the deflection value for all load stages. This increase becomes larger as the applied load increases. The increase is more pronounced for HSCC beams.

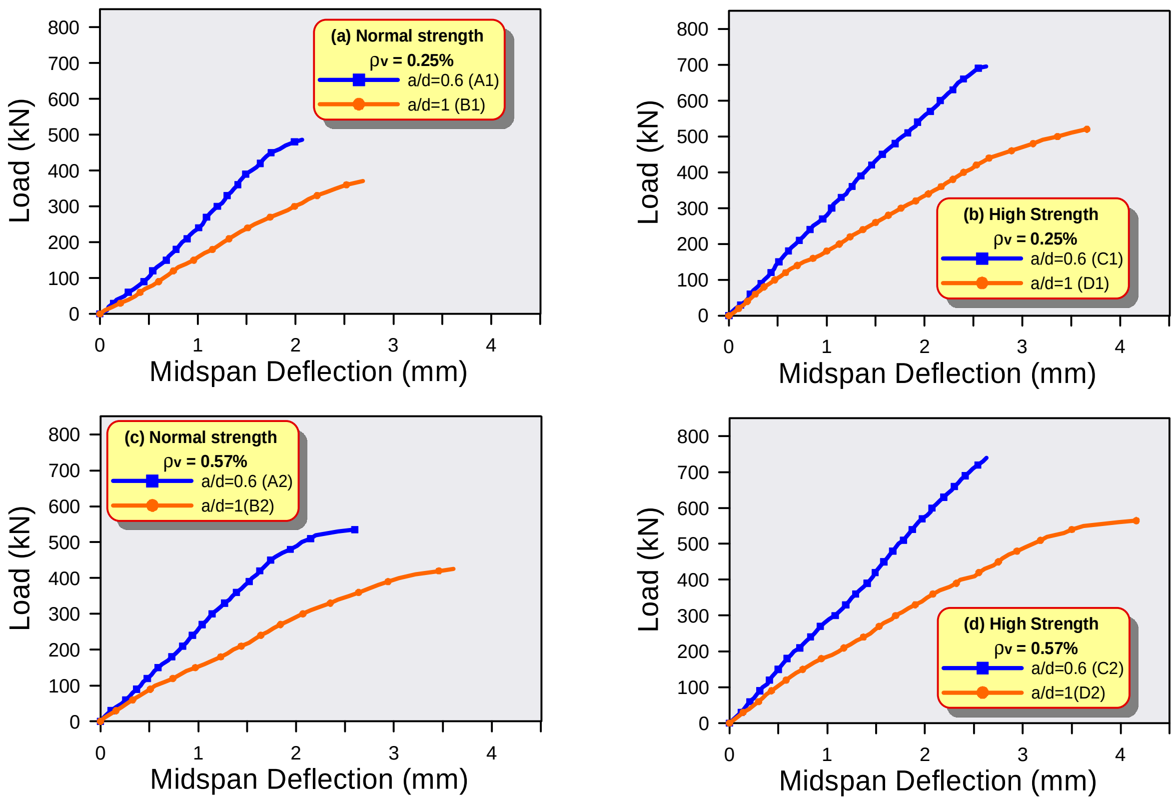

Fig. (10) Effect of (a/d) ratio on load – midspan deflection plot

13. Effect of Concrete Compressive Strength (f'_c)

Effect of (f'_c) on cracking and ultimate loads and the ratio between them for all tested beams are detailed in **Table (8).** The improving in ultimate load due to doubling the (f'_c) value ranges from 32.9 % to 43.3 % (average increase is 38.8 %). This improvement is larger as (a/d) ratio decreases. Also, this improvement is smaller as (ρ_v) increases. The improvement in cracking load due to doubling the (f'_c) value ranges from 5.6 % to 18.2 % (average increase is 11.7 %). This improvement in cracking load becomes smaller as the (a/d) ratio increases. Also generally, the improvement reduces as (ρ_v) increase. The ratio between the cracking and ultimate loads ranges from 0.32 to 0.34 for NSCC beams while it ranges from 0.21 to 0.28 for HSCC beams, i.e., this ratio decreases as (f'_c) increases.

From the **Figure (11),** it is clear that the increase in (f'_c) value reduces the deflection for all load stages. The reduction in deflection as a result of rising (f'_c) is insignificant. The increase in (f'_c) value results in higher modulus of elasticity then result in higher flexural rigidity (EI), therefore, the deflection is smaller (positive action). But This increase in (f'_c) value results in decreasing the compression zone depth because of rising the neutral axis according to equilibrium of internal forces. This leads to smaller moment of inertia for beam section, thereby this leads to a reduction in flexural rigidity (EI) and an increase in the deflection (negative action). The two contradictory actions of (f'_c) lead to insignificant effect on deflection value.

Table (8) Effect of (f'_c) on cracking and ultimate loads

	r_v %	Normal Strength			High Strength			% Variation due to increasing (f'_c)	
		P_{cr} kN	P_u kN	P_{cr} / P_u	P_{cr} kN	P_u kN	P_{cr} / P_u	DP_{cr} %	DP_u %
a / d = 0.6	0.25	165	485	0.34	195	695	0.28	+18.2	+43.3
	0. 57	180	535	0.34	190	740	0.26	+5.6	+38.3
a / d = 1	0.25	125	370	0.34	140	520	0.27	+12	+40.5
	0. 57	135	425	0.32	120	565	0.21	-11.1	+32.9

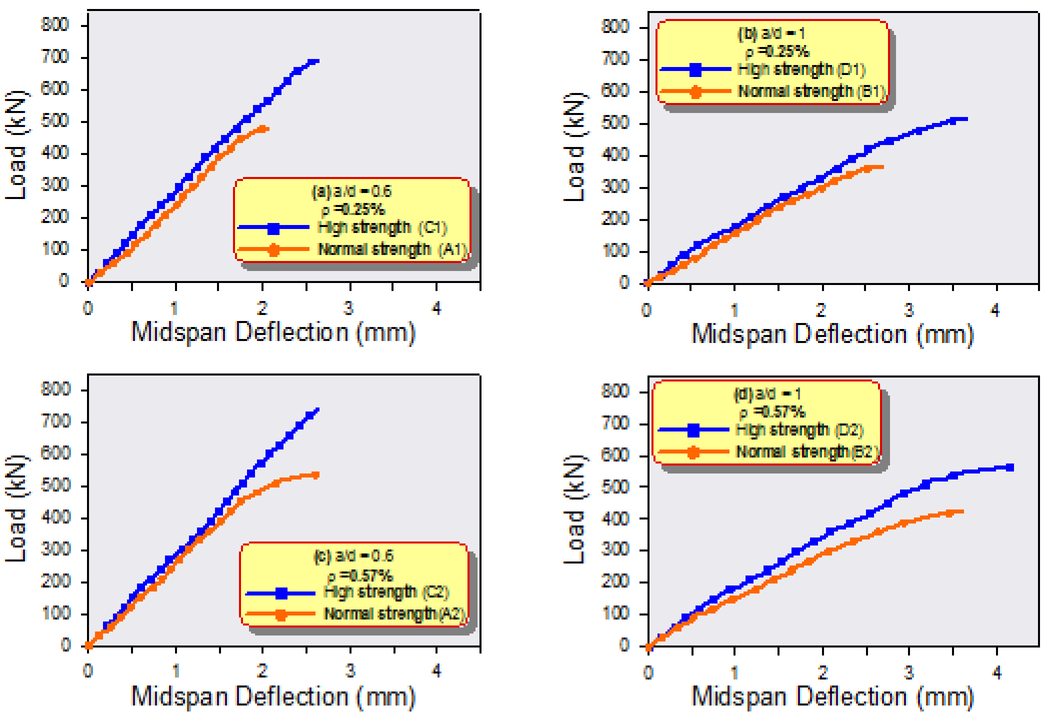

Fig .(11) Effect of (f'_c) ratio on load – midspan deflection plot

14. Effect of Vertical Shear Reinforcement Ratio (ρ_v)

Effect of (ρ_v) on cracking and ultimate loads and the ratio of them for all tested beams are detailed and **Table (9).** When (ρ_v) increases from 0.25 % to 0.57 %, the ultimate load increases by percentages ranging from 6.5 % to 14.9 % (the average of increase is 10.1 %). Effect of (ρ_v) is larger as the (a/d) ratio increases. Also its effect in NSCC beams is larger than its effect in HSCC beams. Effect of increasing (ρ_v) values on cracking load seems to be unclear and random where it leads to a range varying from -14.3 % to +9.1 %. The ratio between cracking and ultimate loads decreases with increasing (ρ_v) value where it ranges from 0.27 to 0.34 with average value of 0.31 for (ρ_v) = 0.25 %. While it ranges from 0.21 to 0.34 with average value of 0.28 for (ρ_v) = 0.57%.

Figure (12) shows that the increase in shear reinforcement causes small and random variations in the deflection values. Therefore each two beams which are only different by the shear reinforcement ratio have convergent load-deflection plots for all load stages. This is because the fact that the shear reinforcement has no substantial effect on flexural rigidity.

Table (9) Effect of (r_v) ratio on cracking and ultimate loads

	Strength type	r_v = 0.25 %			r_v = 0.57 %			% Variation due to increasing (% r_v)	
		P_{cr} kN	P_u kN	P_{cr} / P_u	P_{cr} kN	P_u kN	P_{cr} / P_u	DP_{cr} %	DP_u %
a / d = 0.6	NSCC	165	485	0.34	180	535	0.34	+9.1	+10.3
	HSCC	195	695	0.28	190	740	0.26	-2.6	+6.5
a / d = 1	NSCC	125	370	0.34	135	425	0.32	+8	+14.9
	HSCC	140	520	0.27	120	565	0.21	-14.3	+8.7

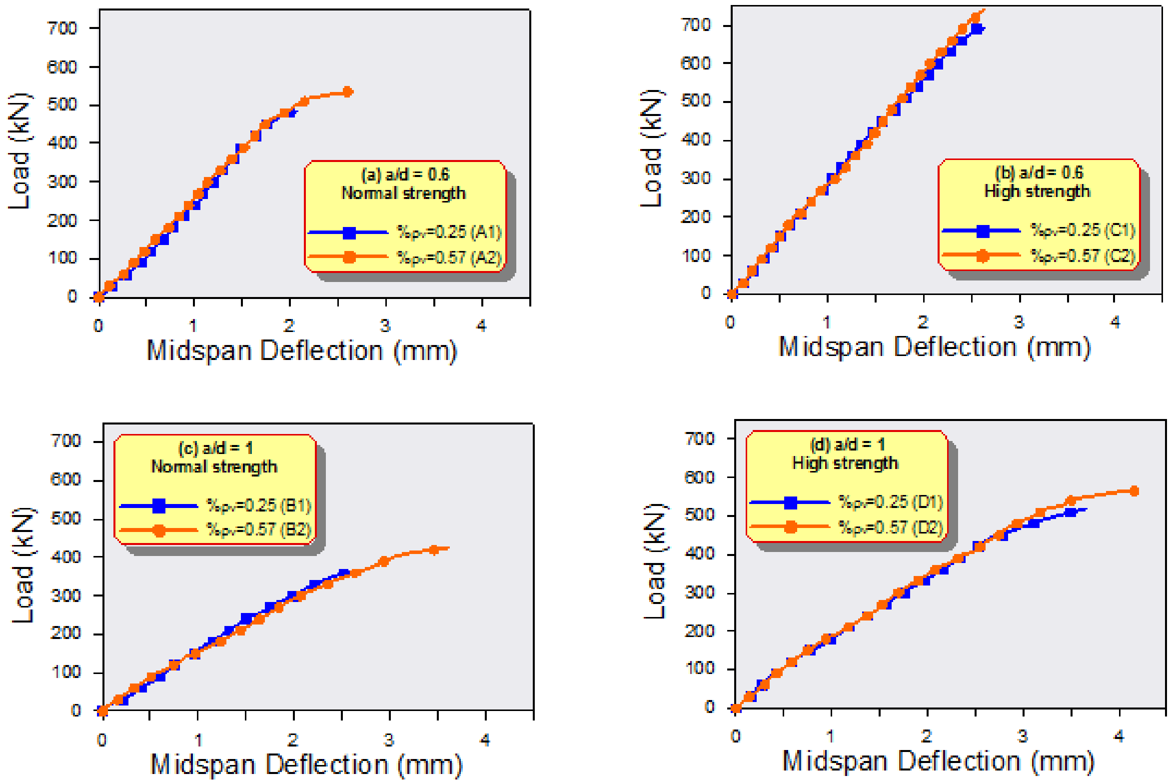

Fig .(12) Effect of (r_v) ratio on load – midspan deflection plot

15. Proposed Method for Analysis of SCC Deep Beams

The present study involves an attempt to derive new methods based on STM technique for predicting ultimate shear capacity of SCC deep beams that reflects the behavior of SCC deep beams accurately. This proposal adopts the same model of truss adopted by the ACI Code and most of the researchers which is shown in **Figure (13).** But this proposal assumes that the adopted truss model is suitable for predicting concrete shear strength only because the truss structure consists of concrete and longitudinal reinforcement only where the web reinforcement is not part of this structure. Therefore, no real role for web reinforcement is present in this model and the value of shear predicted by this model represents the concrete shear strength (V_c) rather than total shear strength (V_n).

The concentrated loads applied on deep beams cause two types of stress. The first is compression stress in direction of strut axis, while the second is the tension stress in transverse direction of strut. In this model of truss, the compression stresses are resisted by concrete of the strut while the tension stresses are resisted by reinforcement of the tie.

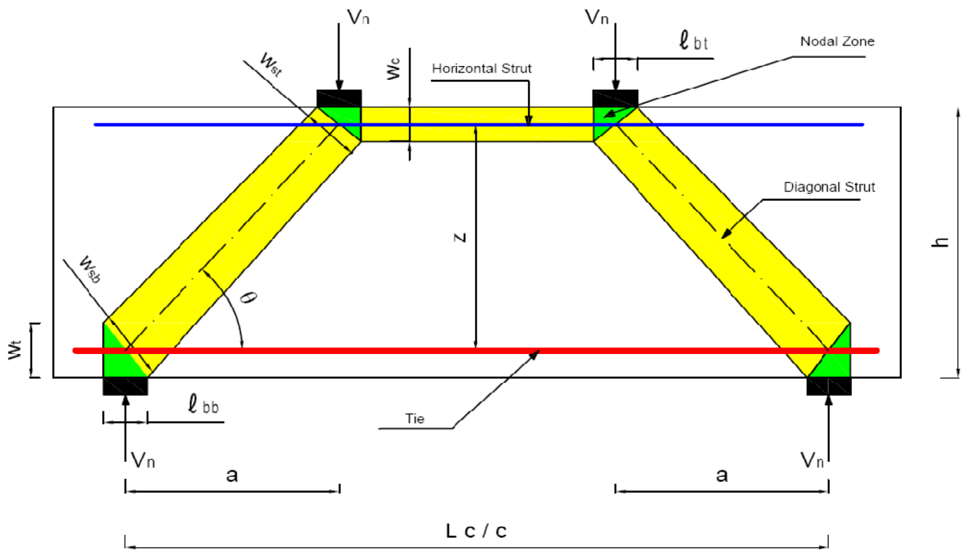

Fig .(13) Analogy of deep beam by STM

If the diagonal strut resists the applied load only, failure will occur when the force in the diagonal strut reaches the ultimate compressive capacity of this strut according to the equation:

$$\frac{V_c}{V_{cc}} = 1 \qquad (1)$$

If the tie resists the applied load only, the failure will occur when the force in tie reaches the ultimate tensile capacity of the tie at yielding of longitudinal reinforcement according to the equation:

$$\frac{V_c}{V_{ct}} = 1 \qquad (2)$$

where

V_c = applied shear force

V_{cc} = shear force resulting from reaching compression stress of diagonal strut to its ultimate value, which is calculated by forces equilibrium as follows:

$$V_{cc} = max.stress * A_{cs}\, sin\theta \qquad (3)$$

Maximum compressive stress in concrete is taken as ($0.85f_c^{'}$). If this value is substituted in Eq. (3), the following equation will be obtained:

$$V_{cc} = 0.85f_c^{'}A_{cs}\, sin\theta \qquad (4)$$

where:

$$A_{cs} = b(w_t \cos\theta + \ell_{bb} \sin\theta) \qquad (5)$$

$$\theta = \tan^{-1} \frac{d-d'}{a}$$

V_{ct} = shear force resulting from reaching longitudinal reinforcement strength to its ultimate value, which is calculated by forces equilibrium as follows:

$$V_{ct} = A_{ts} f_y \tan\theta \qquad (6)$$

When both diagonal strut and tie resist the applied load through the truss structure, the failure criteria will be combination of compression failure criterion and tension failure criterion as expressed in the following equation:

$$\frac{V_c}{V_{cc}} + \frac{V_c}{V_{ct}} = 1 \qquad (7)$$

This equation is based on Mohr-Columb failure criterion which adopts the linear interaction between these two components through normalized expression.

In this proposal, the nonlinear interaction will be adopted by raising terms of this equation to a certain power which is assumed as a constant value and will be denoted as (c_1). This constant will be determined by calibration with test data. Based on this assumption Eq. (7) becomes as follows:

$$\left(\frac{V_c}{V_{cc}}\right)^{c1} + \left(\frac{V_c}{V_{ct}}\right)^{c1} = 1 \qquad (8)$$

Eq. (8) can be arranged to give the applied shear force (V_c) which represents the concrete shear strength of deep beams, as follows:

$$V_c = \frac{V_{cc}\, V_{ct}}{\sqrt[c_1]{(V_{cc})^{c_1} + (V_{ct})^{c_1}}} \qquad (9)$$

The reinforcement shear strength (V_s) must be calculated separately and added to the concrete shear strength (V_c). For this purpose, the following formula can be proposed for calculating reinforcement shear strength:

$$V_s = \left(C_2 \frac{a}{d} \rho_v f_{yv} + (C_3 - \frac{a}{d}) \rho_h f_{yh}\right) bd \qquad (10)$$

Where the total shear strength (V_n) becomes as follows:

$$V_n = V_c + V_s \qquad (11)$$

Experimental results of eight beams tested in this study and another fourteen beams available in literature[11] were adopted for regression analysis by Data Fit program for determination of coefficients (C_1, C_2, and C_3). The coefficient (C_1) which gives the higher agreement with test results is 1.93. This value is approximated to 2 for simplification and familiarity purposes, while the coefficients C_2 and C_3 are about 1 and 3. Thus, the formula of total shear strength can be expressed as:

$$\mathbf{V_n} = \frac{\mathbf{V_{cc}\,V_{ct}}}{\sqrt{(\mathbf{V_{cc}})^2+(\mathbf{V_{ct}})^2}} + \left(\frac{a}{d}\rho_v f_{yv} + (3-\frac{a}{d})\rho_h f_{yh}\right) bd \qquad (12)$$

Use $C_1 = 2$ means that the best interaction for combination between normalized compression stresses of diagonal strut and normalized tension stresses of the tie for adopted model truss of SCC deep beams is a circular curve.

16. ACI 318 Code-2011 method[6] for Analysis of SCC Deep Beams

The nominal compressive strength of a strut is calculated according to the formula:

$$F_{ns} = f_{ce} A_{cs} \qquad (13)$$

$$f_{ce} = 0.85 \beta_s f_c' \qquad (14)$$

β_s value is taken as $= 0.75$ for struts located such that the width of the midsection of the strut is larger than the width at the nodes (bottle-shaped struts) with reinforcement satisfying the sufficient transverse reinforcement while $\beta_s = 0.6$ for bottle-shaped struts with reinforcement that is not satisfying the sufficient transverse reinforcement.

The Code considers the transverse reinforcement requirement to be satisfied if the strut is crossed by layers of reinforcement that satisfying the following formula:

$$\sum \frac{A_{si}}{b_s s_i} \sin \alpha_i \geq 0.003 \qquad (15)$$

$$A_{cs} = b w_s \qquad (16)$$

w_s = width of strut which is equal to w_c for horizontal strut and taken as the smaller of w_{st} or w_{sb} for diagonal strut as shown in **Figure (13).**

$$w_{sb} = w_t \cos\theta + \ell_{bb} \sin\theta \qquad (17)$$

$$w_{st} = w_c \cos\theta + \ell_{bt} \sin\theta \qquad (18)$$

The nominal strength of non prestressed tie shall be taken as:

$$F_{nt} = A_{ts} f_y \tag{19}$$

The nominal compressive strength of a nodal zone shall be:

$$F_{nn} = f_{ce} A_{nz} \tag{20}$$

Where A_{nz} = area of nodal zone face

$$f_{ce} = 0.85\, \beta_n f_c' \tag{21}$$

β_n = effectiveness factor for a nodal zone which is taken as $\beta_n = 1$ for nodal zones bounded by struts or bearing areas or both (C-C-C nodes) while $\beta_n = 0.8$ for nodal zones anchoring one tie (C-C-T nodes).

Ultimate shear strength (V_n) of deep beam is determined by equilibrium equations of truss modeled as in **Figure (13),** and taken as the smaller of the following values:

- From diagonal strut strength:

$$V_n = F_{ns} \sin\theta \tag{22}$$

- From horizontal strut strength:

$$V_n = F_{ns} \tan\theta \tag{23}$$

- From tie strength:

$$V_n = F_{nt} \tan\theta \tag{24}$$

- From nodal zone strength:

$$V_n = F_{nn} \sin\theta \quad \text{for face perpendicular to diagonal strut} \tag{25}$$

$$V_n = F_{nn} \tan\theta \quad \text{for face perpendicular to tie} \tag{26}$$

$$V_n = F_{nn} \quad \text{for face perpendicular to the applied loads} \tag{27}$$

17. Results of analysis

Experimental results of eight beams tested in this study and another fourteen beams available in literature[11] are adopted for checking the validity of derived method. **Table (10)** gives ratios of experimental to predicted results of shear strength values obtained by the proposed and the ACI- Code methods. The proposed method gives ratios that are close to unity for all beams. This means that the analytical results of the proposed method have a good correlation with experimental results. This reflects the accuracy and rationality of the proposed method. The results show that the ACI Code method is conservative. This means that the ACI Code method significantly underestimate the ultimate shear capacity. The ACI Code method is less accurate than the proposed method. This is evident through the statistical results listed at bottom of **Table (10).**

Also, **Figure (14)** shows a comparison between experimental and predicted ultimate shear strengths (V_n) for the proposed and the ACI Code methods. This figure shows the good correlation between the experimental and analytical results obtained for proposed method in comparison with the ACI Code method where its data points are less dispersant and closer to the 45^o line than data points of the ACI Code method. Also, the figure shows underestimation or conservation of the ACI Code method where the fit line of their data lies away from the 45^o line.

Table (10) Result of analysis by proposed method and ACI-2011 method

Beam Name	$V_{n\ Exp.}$ (kN)	$V_{n\ Exp.} / V_{n\ Predicted}$		Beam Name	$V_{n\ Exp.}$ (kN)	$V_{n\ Exp.} / V_{n\ Predicted}$	
		ACI 2011	Proposed Method			ACI 2011	Proposed Method
A1	242.5	2.095	1.096	B4	300	1.608	1.006
A2	267.5	1.996	1.102	B5	265	1.420	0.954
B1	185	1.952	0.965	B6	250	1.340	0.97
B2	212.5	1.794	0.934	B7	235	1.259	0.99
C1	347.5	1.795	0.972	B8	280	1.501	1.008
C2	370	1.911	0.976	B9	265	1.420	1.002
D1	260	1.394	0.943	B10	253.5	1.359	0.984
D2	345	1.802	1.056	B11	248	1.329	0.978
B1	335	1.795	0.989	B12	245	1.313	0.976
B2	375	1.806	0.999	B13	315	1.688	1.033
B3	444	1.940	1.055	B14	265	1.420	0.977
Avg.		**1.618**	**0.992**				
S.D.		**0.258**	**0.046**				
C.O.V.		**15.93 %**	**4.62 %**				
C.C.		**0.684**	**0.976**				

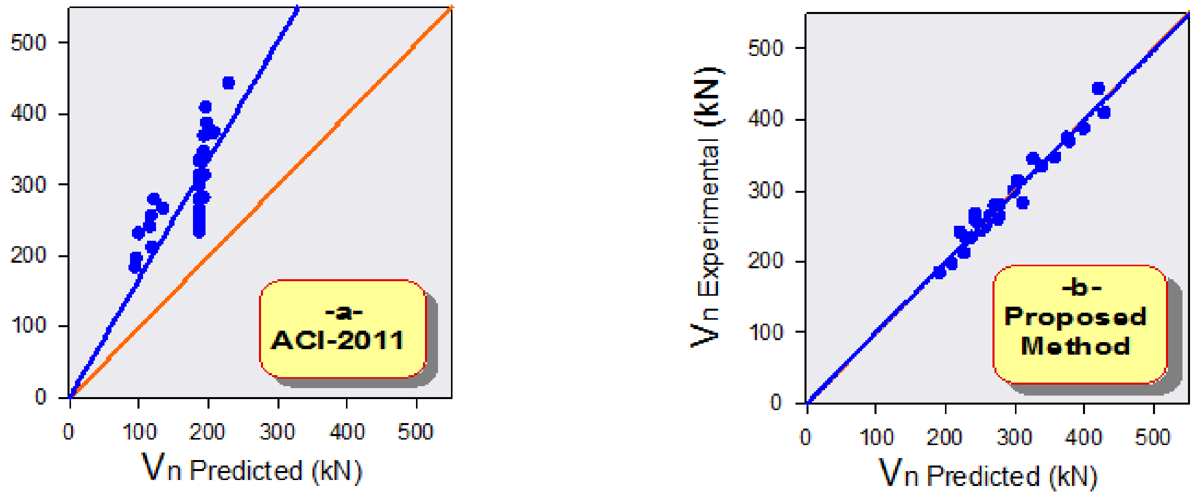

Fig .(14) Comparison between experimental and predicted ultimate shear strengths (V_n)

18. Conclusions

1. All tested SCC deep beams were failed by shear. The shear failure took place by diagonal splitting mode for all tested beams except beam (A2) where its shear failure took place by diagonal compression mode.
2. It was found that for all tested beams the increase in the shear span to effective depth ratio (a/d) from 0.6 to 1 reduces the cracking load by a range of 24.2 % to 36.8 % (average of reduction is 28.6 %). This reduction becomes larger in high strength concrete (HSCC) beams when compared with the normal strength concrete (NSCC) beams.
3. The increase in the (a/d) ratio from 0.6 to 1 reduces the ultimate load with a range of 20.6 % to 25.2 % for all tested beams (average of reduction is 23.3 %). This reduction becomes slightly larger in HSCC beams.
4. For all tested beams, the increase in the concrete compressive strength (f'_c) from 32.84 MPa to 64.65 improves the cracking load by a range of 5.6 % to 18.2 % (average increase is 11.7 %). This improvement decreases as (a/d) ratio increases and as (ρ_v) increases.
5. By increasing (f'_c) from 32.84 MPa to 64.65, the ultimate load improves with a range of 32.9 % to 43.3 % for all beams (average increase is 38.8 %). This improvement decreases as (a/d) ratio increases and as (ρ_v) increases.
6. The increase in the vertical web reinforcement ratio (ρ_v) from 0.25 % to 0.57 % leads to increases in ultimate load in the range from 6.5 % to 14.9 for all cases (the average of increase is 10.1 %). These increases become larger as (a/d) ratio increases and as the (f'_c) value decreases.

7. Values of cracking to ultimate load ratio range from 0.21 to 0.34 for all beams. This ratio is slightly decreased with increasing (a/d) ratio. Also, this ratio decreases with increasing (f'_c).
8. The load- deflection response of SCC deep beams is significantly affected by (a/d) ratio. The response becomes appreciably nonlinear as the (a/d) ratio increases. Load- deflection response is slightly affected by the compressive strength of concrete (f'_c). It was found that the response is slightly stiffer as (f'_c) increases. Also, it was noticed that the shear reinforcement has no obvious effect on load-deflection response.
9. An analytical method to predict (V_n) is derived based on a modified strut and tie model with adopting a circular failure interaction relation between tension and compression stresses in members of represented simple truss model. It was found that the proposed method give results that very close to test results of 22 SCC deep beams by comparison with results of the ACI-2011 Code method which is conservative. The proposed method gives Avg. = 0.992 and C.O.V. = 4.62 % While the ACI Code method gives Avg. = 1.618 and C.O.V. = 15.93 %.

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