On the Contribution of Shear Reinforcement in Shear Strength of Shallow Wide Beams

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Abstract: One of the common floor structural systems used in the Middle East is reinforced concrete hollow block slab with shallow wide beams (hidden beams). Most of the building codes in the middle east; the current Egyptian Code of practice (ECP 203-2007) for example, require that the applied one way shear stress in the shallow wide beams be less than the concrete shear strength without any shear reinforcement contribution, and the shear strength provided by concrete equals two thirds of concrete shear strength of shallow slender beams. As a consequence; a large cross-sectional areas of concrete shall be provided for these members to resist one-way shear demands which results in a conservative uneconomic design provision. The above mentioned requirements by some building codes in the Middle East were not found in most of other recognized international codes or standards. An experimental program was carried out to investigate the contribution of web shear reinforcement to shear strength of shallow wide beams. The main parameters considered in this investigation were: concrete compressive strengths and vertical stirrups; with varying amount, configuration and spacing. The experimental program consisted of twelve simply-supported reinforced concrete wide beams subjected to two concentrated loads at third points. The specimens were divided into 5 groups. All specimens were typically proportioned so that shear failure would preclude flexural failure. Shear strengths at failure recorded in this experimental program are compared to the analytical strengths calculated according to some international codes. Test results clearly demonstrate the significance of the web reinforcement in improving the shear capacity the ductility of the shallow wide beams which is consistent with the recognized international codes and standards provisions.

[Mohamed M. Hanafy, Hatem M. Mohamed and Nabil A.B. Yehia. **On the Contribution of Shear Reinforcement in Shear Strength of Shallow Wide Beams.** *Life Sci J* 2012;9(3):484-498] (ISSN:1097-8135). http://www.lifesciencesite.com. 68

Keywords: shear strength, shallow wide beams, stirrups, normal strength concrete, high strength concrete, modified compression field theory.

1. Introduction

In design of buildings, modern architectural constraints are pushing the designers to provide longer clear spans at a reasonable cost. At the same time, there is a need to minimize the overall structural slab depth to achieve more floor clear height, which can be achieved through the use of either shallow wide beams (Hidden Beams) or flat plate slabs.

According to the majority of building code in the middle east; Egyptian Code of practice (ECP 203-2007) [1] for example; the shear stress in shallow wide beams must be less than the concrete shear strength with no consideration of the contribution of shear reinforcement. Moreover, according to the same code; the shear strength provided by concrete for shallow wide beams equals 67% of the concrete shear strength for shallow slender beams. As a consequence, large cross-sectional areas of concrete shall be provided to meet one-way shear demands. In other words, while the code neglects the web reinforcement contribution in shear strength, it persists on providing specified minimum web reinforcement, and moreover, reduces the concrete shear strength. These three conjugate requirements of the code lead to a very conservative, yet uneconomic, shear design of shallow wide beams. In the same stream, the code requires the stirrups to be arranged so that the distance between stirrup branches across the beam section not to exceed 250 mm.

High-strength concrete has gained an increased interest in reinforced concrete structures in last ten years as it generally leads to the design of smaller sections. This in turn reduces the dead weight, allowing longer spans and more usable area of building. However an increase in the concrete strength produces an increase in its brittleness and smoothness of shear failure surfaces, leading to some concerns about the application of high strength concrete. In the last few years the development of concrete technology and practice has led to a significant change of what high strength concrete is, and subsequently, the definition of high strength concrete has changed over the time. For instance in the 1950s, concrete with compressive strength ($f_c^{'}$) of 35 N/mm^2 was considered to be high strength concrete. Currently, a number of construction projects have used concrete with 28-day compressive strengths ($f_c^{'}$) in the range of 65 to70 N/mm^2. American Concrete Institute ACI 363 [2] defines the high-strength concrete as a concrete with a minimum 28-day cylinder compressive strength ($f_c^{'}$) of 41 N/mm^2.

Recently, few experimental and analytical investigators directed their attention to study the shear behavior of shallow wide beams. Most of the current shear procedures are based on tests carried out on beams with a concrete compressive strength ($f_c^{'}$) lower than 70 N/mm^2. In addition, the mechanism of shear failure is not fully understood due to the lack of research in high-strength concrete shallow wide beam.

Khalil,[3] carried out an experimental study to investigate the shear behavior of hidden beams (wide shallow beams) in hollow block slabs. His experimental investigation included nine medium-scales simply supported hidden beams and five full-scale hollow block one way slabs with normal concrete strength. The results showed that the capacity of specimens with shear reinforcement reached as high as 300% of those without shear reinforcement. Lubell *et al.,* [4] carried out an experimental study to investigate the shear behavior of the wide beams and thick slabs as well as the influence of member width. In their study they tested five specimens of normal strength concrete with a nominal thickness of 470 mm and varied in width from 250 to 3005 mm. The study demonstrated that the failure shear stresses of narrow beams, wide beams, and slabs are all very similar. It is worth mentioning that the basic expression for one-way shear in ACI 318-02 [5] is the same for narrow beams and wide beams. Dino Angelakos, *et al.,* [6] investigated the effect of concrete strength and minimum stirrups on shear strength of large members in more details. They conducted an experimental program of twelve 1000 mm deep beams with concrete strengths ($f_c^{'}$) varying from 21 to 80 N/mm^2. The beams were loaded by a point load applied at the middle of a 5400 mm simply supported span. Their tests revealed that even large lightly reinforced members containing minimum level of stirrups can fail at approximately 70% of the ACI 318-02 [5] predicted shear strength. They also concluded that changing the concrete strength by a factor of 4 had almost no influence on the shear strength of these large beams while changing the longitudinal reinforcement ratio from 0.5 to 2.09% increased the observed shear strength by 62%. James and James [7] investigated the shear behavior of reinforced concrete exterior wide beam-column-slab connections subjected to lateral earthquake loading. An experimental program of three reinforced concrete exterior wide beam-column-slab specimens were designed, constructed, instrumented and tested. The three specimens were all two-thirds scale, and had 300 mm deep wide beam. The width of the wide beams varied from 865 to 940 mm. The wide beams were constructed with concrete strengths varying from 29 to 34.5 N/mm^2. Upon examining the beams after failure, they observed that the wide beams never exhibited any inclined cracking that could be characterized as related to shear. Observed cracks were narrow, vertical flexural cracks that opened very little. Stirrups strain gages never measured strains in the stirrups vertical legs greater than one-third of the yield strain, hence, they concluded that the wide beams performed well in the shear. Lubell *et al.,* [8] investigated the influence of the shear reinforcement spacing on the one-way shear capacity of wide reinforced concrete members. A series of 13 normal strength concrete specimens were designed and tested. Shear reinforcement spacing was a primary test variable. The specimens contained web reinforcement ratios close to ACI 318-02 [5] minimum requirements. The study concluded that the effectiveness of the shear reinforcement decreases as the spacing of web reinforcement legs across the width of a member increases, the use of few web reinforcement legs, even when widely spaced up to a distance of approximately 2d, has been shown to decrease the brittleness of the failure mode compared with a geometrically similar member without web reinforcement. To ensure that the shear capacity of all members with web reinforcement are adequate when designed according to ACI 318-02 [5], the study recommended that the transverse spacing of web reinforcement should be limited to the lesser of both the effective member depth and 600 mm.

The objective of this research program is to determine the effect of the following parameters on the shallow wide beam shear resistance: (i) concrete compressive strength, (ii) existence of vertical stirrups as web reinforcement, (iii) volumetric ratio of vertical stirrups (iv) spacing between vertical stirrups, and (v) number of vertical stirrups branches in section . A comparison between test results and the prediction of different building codes such as (ECP 203-2007) [1], ACI 318-02 [5], EN1992 [9], ASHTO-LRFD [10] and CSA 2004 [11] is also presented. A similar comparison is made between the experimental test results and analytical results obtained through the application of the windows based computer program *"Response 2000"* which employs the modified compression field theory (MCFT) [12]. CSA 2004 [10] prediction was obtained using the computer program "*Response 2000*" [13] since the modified compression field theory forms much of the basis of the Canadian design code.

Codes' Review For Shear Of Shallow Wide Beams

Egyptian Code of practice (ECP 203-2007) [1]

The current Egyptian Code of practice (ECP 203-2007) determines the shear resistance of shallow wide beams as following:

$$q_u \le q_{cu} \quad (1)$$

$$q_{cu} = 0.16\sqrt{\frac{f_{cu}}{\gamma_c}}\; b_w d \quad (2)$$

Where q_{cu} is the concrete shear capacity (N/mm^2), f_{cu} is the concrete characteristic cube strength (N/mm^2), γ_c is concrete partial safety factor equals 1.50, b_w is the width of the web (mm) and d is the effective depth of the section (mm). The code neglects the web reinforcement contribution in shear strength of shallow wide beams, while stressing the need to provide specified minimum web reinforcement, and at the same time reduces the concrete shear strength for shallow wide beams.

American Concrete Institute (ACI 318-02) [5]

According to ACI 318-02; the nominal shear strength, V_n, of non-prestressed members is the sum of the concrete contribution; V_c, and shear reinforcement contribution; V_s. Thus,

$$\phi V_n \geq V_u \quad (3)$$

$$V_n = V_c + V_s \quad (4)$$

Where V_u is the factored shear force at the section, the concrete contribution term, V_c , can be calculated by either of the following two equations:

$$V_c = 0.17\sqrt{f_c'}\, b_w d \quad (5)$$

$$V_c = \left[0.16\sqrt{f'_c} + 17\rho_w \frac{V_u d}{M_u}\right] b_w d \leq 0.3\sqrt{f_c'}\, b_w d \quad (6)$$

When the factored shear force V_u exceeds the shear strength provided by concrete; ϕV_c, shear reinforcement must be provided to carry the excess shear and its contribution is calculated as:

$$V_s = \frac{A_v f_y d}{s} \leq 0.66\sqrt{f_c'}\, b_w d \quad (7)$$

Where: V_u = factored shear force at the section (N), V_c = nominal shear strength provided by concrete (N), V_s = nominal shear strength provided by shear reinforcement (N), V_n = nominal shear strength (N), M_u = factored flexural moment at section (N.mm), ϕ = strength reduction factor = 0.75, $\rho_w = A_s/b_w d$, As = area of longitudinal reinforcement (mm^2), A_v = area of shear reinforcement (mm^2), b_w= web width of section (mm), d= distance from the extreme compression fiber to the centroidal axis of the longitudinal reinforcement (mm), s = spacing of the transverse reinforcement (mm), f_c'= concrete compressive cylinder strength (MPa), f_y = yield strength of the transverse reinforcement (MPa).

The ACI prediction gives un-conservative results for large lightly reinforced members without shear reinforcement. A minimum area of shear reinforcement, $A_{v,min}$, shall be provided in all reinforced concrete flexural members where V_u exceeds $0.5\varphi Vc$, except beams with h not greater than the largest of 250mm, 2.5 times thickness of flange, or 0.5 the width of web (i.e. shallow wide beams) because there is a possibility of load sharing between weak and strong areas.

Eurocode (EN1992) [9]

Members Not Requiring Shear Reinforcement

The design value for the shear resistance $V_{Rd,c}$ is given by:

$$V_{Rd,c} = [(0.18/\gamma_c)k(100\rho_l f_{ck})^{1/3}]b_w d \quad \mathbf{(8)}$$

$$k = 1 + \sqrt{\frac{0.20}{d}} \leq 2.0 \quad \mathbf{(9)}$$

$$\rho_l = \frac{A_{sl}}{b_w d} \leq 0.02 \quad \mathbf{(10)}$$

Where f_{ck} = characteristic concrete cube strength (MPa), A_{sl} =the area of the tensile reinforcement (mm^2), b_w = the smallest width of the cross-section in the tensile area (mm), γ_c is concrete partial safety factor equals *1.50.*

Members Requiring Shear Reinforcement

The code neglects the concrete contribution in this case $V_{Rd,c}= 0$. The design of members with shear reinforcement is based on a truss model, whereby the values for the angle θ of the inclined struts in the web are limited as follows:

$$1 \leq \cot\theta \leq 2.5 \quad (11)$$

For members with vertical shear reinforcement, the shear resistance, $V_{Rd,s}$, is given by:

$$V_{Rd,s} = (A_{sw}/s) z f_{ywd} \cot\theta \quad (12)$$

Where:

A_{sw} = cross-sectional area of the shear reinforcement (mm^2), s = spacing of the stirrups (mm), f_{ywd} = yield strength of the shear reinforcement (MPa), θ = the angle between inclined concrete struts and the main tension chord, z = the inner lever arm for a member with constant depth (mm)

The Eurocode EN1992 [9] is applicable up to concrete strengths of f_{ck} =90 MPa, which corresponds to f_c'=91.6 MPa. The characteristic value f_{ck} for the cylinder strength is defined as a 5% fractile. By contrast f_c' is a 9% fractile, and the relation between the two quantities is $f_{ck} = f_c' - 1.6$ (MPa).

AASHTO LRFD Bridge Design Specifications (2005) [10]

AASHTO LRFD Section Design Model for Shear is a hand-based shear design procedure derived from the Modified Compression Field Theory (MCFT). The nominal shear resistance; Vn, can be computed by:

$$V_n = V_c + V_s \quad (13)$$

$$V_C = 0.083\beta\sqrt{f_c'}\, b_v d_v \quad (14)$$

$$V_s = \frac{A_v f_y d_v(\cot\theta + \cot\alpha)\sin\alpha}{s} \quad (15)$$

Where:

b_v = effective web width, taken as the minimum web width within the depth (mm), d_v = effective shear depth = the greater of ($0.9d$ or $0.72h$) (mm), β = factor indicating the ability of diagonally cracked concrete to transmit tension, θ = Angle of inclination of the diagonal compressive struts, α = angle of inclined stirrups to longitudinal axis. All other notation are identical to those indicated before.

Canadian Standards Association (CSA A23.3-04) [11]

The MCFT is the basis for the general shear provisions of the CSA [11]. In order to overcome concerns by practicing engineers over difficulties in using the LRFD [10] specifications, the CSA [11] specifications presented below were developed to provide a simpler way to obtain θ and β . In this proposed method, for design purposes θ is taken equal to 30° for evaluating the demand of shear on the longitudinal reinforcement. In this approach, the nominal strength is defined as:

$$V_n = V_c + V_s \leq 0.25 f_c^{'} b_w d_v \quad (16)$$

$$V_C = 0.083\beta \sqrt{f_c}' \; b_w d_v \quad (17)$$

$$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \quad (18)$$

Where: $b_w = b_v$

It should be noted that ACI-318-02, the AASHTO LRFD and CSA do not permit the use of a concrete compressive strength; $f_c^{'}$, greater than 70 MPa for shear strength calculations.

2. EXPERIMENTAL WORK:

In order to investigate effect of the above mentioned parameters on the shear resistance of the shallow wide beams, an experimental program was carried out to test twelve simply-supported reinforced concrete beams, six beams are made of normal concrete compressive strength of f_{cu} =40 N/mm^2, and the remaining six are made of high concrete compressive strength of f_{cu} =90 N/mm^2. Detailed description of the specimens, the material properties, test set-up, instrumentation, test procedure, and measurements are presented in this section.

Test Specimens:

In the experimental program, tests were carried out on twelve concrete beams named (NB1 to NB6) and (HB1 to HB6) where "NB" refers to normal strength concrete beams and "HB" refers to high strength concrete beams. The width/depth ratio is limited to 2 in all specimens.

All tested beams are 500mm x 250mm in cross-section with 800mm flange width along a length of 1100 mm centered in span to ensure that shear failure would preclude flexural failure. All tested beams have 2000mm clear span and the same flexural longitudinal top and bottom reinforcement (6T25+5T22 Bottom and 6T12 Top). The beams were simply supported and subjected to two concentrated static loads (four-point bending). The details of the tested beams are shown in table (1) and Figs. 1, 2 and 3. The test specimens were divided into 5 groups.

Group No. (1): This group consists of two specimens (NB1) and (HB1), each specimen represents the reference specimen for normal and high strength concrete respectively with no web shear reinforcement.

Groups No. (2 & 4): Each group consists of three specimens: (NB2, NB3 and NB4) in group (2) and (HB2, HB3 and HB4) in group (4). All specimens in these two groups are reinforced with Minimum web shear reinforcement ratio (= 0.167%) according to ECP203-2007 [1].

Groups No. (3 & 5): Each group consists of two specimens; (NB5 and NB6) in group (3) and (HB5 and HB6) in group (5), all specimens in these two groups are reinforced with (0.46%) web shear reinforcement ratio (more than minimum stirrups).

Materials:

Trial mixes were conducted in the Concrete Research Laboratory at Cairo University to reach the target cubic compressive strength of 40 N/mm^2 and 90 N/mm^2 after 28 days. Table (2) shows mix proportions by weight of the quantities needed for one cubic meter of concrete to achieve the target cube compressive strength

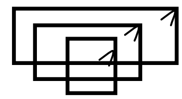
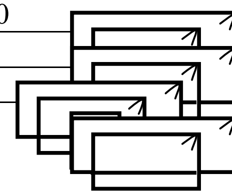
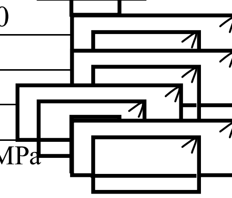
Test Procedure:

The specimens were placed in the testing machine between the jack head and the steel frame and supported on two hinged supports. The strain gages, load cell and linear voltage displacement transducer (LVDT) are all connected to the data acquisition system attached to the computer.

All beams were subjected to two concentrated loads; each load was applied at 750 mm from the support where shear span-to-depth ratio (*a/d*) = 3.57. The load was monitored by a load cell of 2500 KN capacity and transmitted to the reinforced concrete beam through two transversal steel I-beams resting on steel pads to provide uniform bearing surfaces. Figs. (4) and (5) show the testing setup. The load was applied gradually with constant rate of loading, about 50 KN load increments, during the test.

The data acquisition system continuously recorded readings of the electrical load cell; the two LVDTs that measure the beam deflection at mid span and the stirrups strain gages.

Table 1 : Tested beams details

Group	*Specimen*	f_{cu} *(N/mm²)*	*Longitudinal RFT** *Bottom*	*Longitudinal RFT** *Top*	*Web Shear RFT.* (Vertical Stirrups)*
G1	NB1	40	6T25 + 5T22	6T12	____
G2	NB2				3Y6@200 
	NB3				Y8+Y6@200 
	NB4				2Y6@135
G3	NB5				3T10@200
	NB6				2T10@135
G1	HB1	90	6T25 + 5T22	6T12	____
G4	HB2				3Y6@200 
	HB3				Y8+Y6@200 
	HB4				2Y6@135
G5	HB5				3T10@200
	HB6				2T10@135

*T: High Strength steel reinforcement; f_y=420MPa, Y: Mild steel reinforcement; f_y =300MPa

Table 2 Mix Design of Normal and High Strength Concrete

	Compressive target strength (N/mm²)	*Cement (KN)*	*Silica Fume (KN)*	*Crushed Dolomite (KN)*	*Sand (KN)*	*Water (liter)*	*Super-plasticizer (liter)*
NSC	40	4.00	----	12.8	6.4	200	2.5
HSC	90	5.60	1.20	11.20	5.60	145	20

Table 3 Summary of experimental results.

Normal Strength Concrete; F_{cu}=40 MPa					High Strength Concrete; F_{cu}=90 MPa				
Group/ Specimen	Test Results (KN) Cracking Load* Flexural	Test Results (KN) Cracking Load* Shear	Test Results (KN) Failure Load	Failure Mode**	Group/ Specimen	Test Results (KN) Cracking Load* Flexural	Test Results (KN) Cracking Load* Shear	Test Results (KN) Failure Load	Failure Mode**
G1/NB1	270.0	450.0	490.0	SC	G1/HB1	200.0	500.0	590.0	SC
G2/NB2	100.0	450.0	700.0	ST	G4/HB2	150.0	600.0	795.0	ST
G2/NB3	200.0	500.0	600.0	SC	G4/HB3	200.0	500.0	680.0	ST
G2/NB4	150.0	550.0	610.0	ST	G4/HB4	150.0	600.0	700.0	SC
G3/NB5	150.0	550.0	990.0	SC	G5/HB5	150.0	550.0	1190.0	ST
G3/NB6	100.0	500.0	1100.0	SC	G5/HB6	200.0	600.0	1220.0	ST

* cracking load values are approximate values with ± 10.00 KN tolerance.
** Failure mode; SC: shear-compression & ST: shear tension.

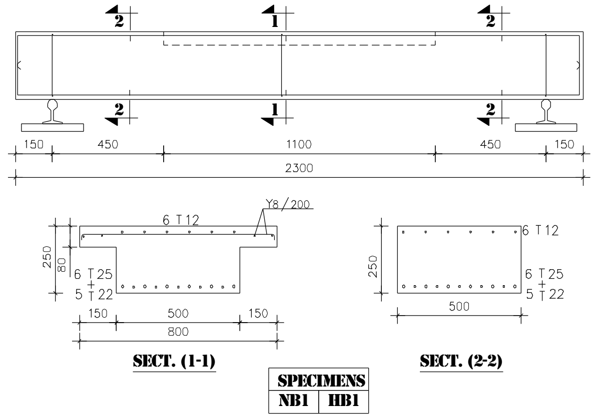


Fig. 1, Details of group No. (1)

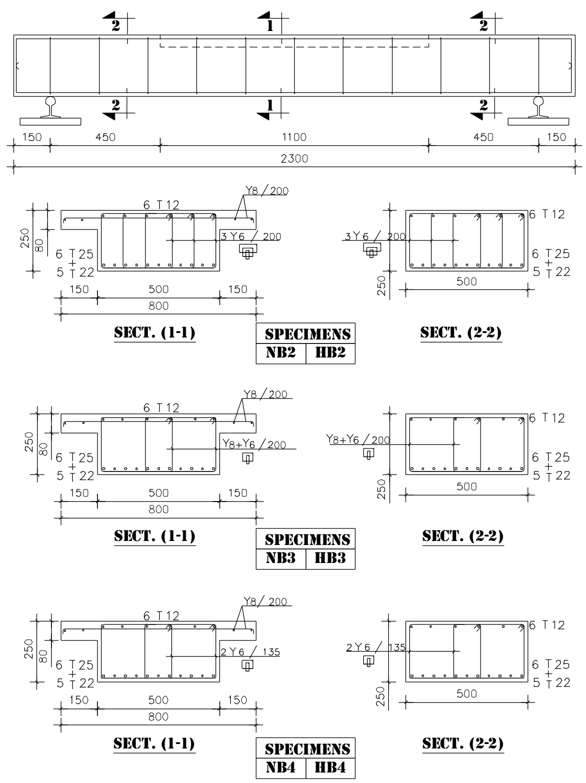


Fig. 2, Details of groups No. (2&4)

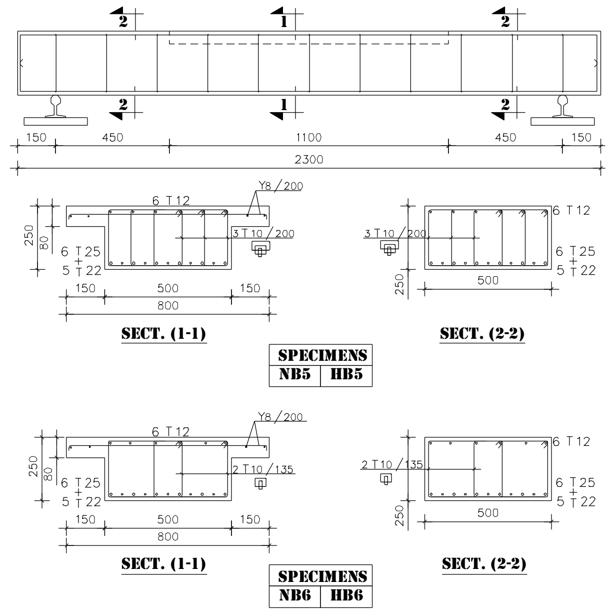


Fig. 3, Details of groups No. (3&5)

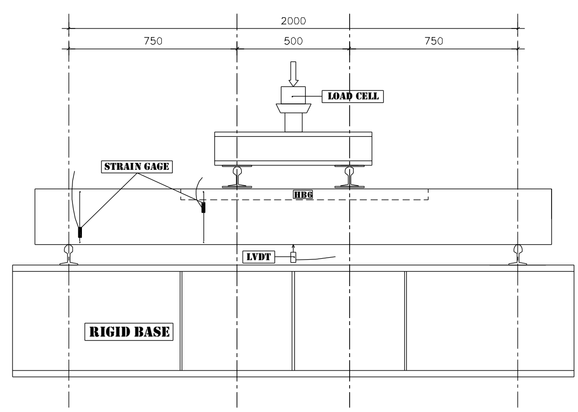


Fig. 4, Schematic Test Arrangement

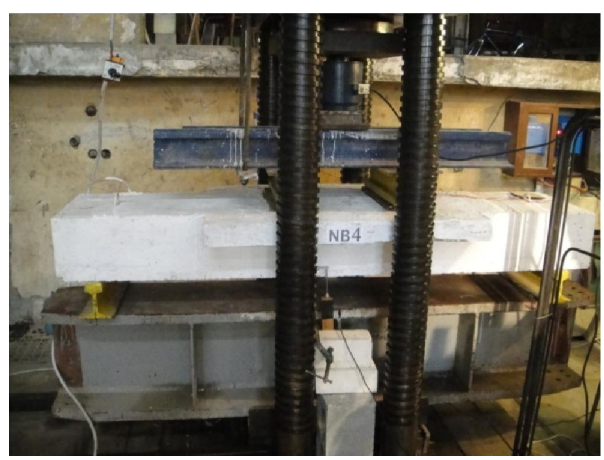


Fig. 5, Test Arrangement

Test Result:

Experimental test results of the twelve specimens are concluded in cracking pattern, load - deflection, and load - stirrups strains for each test specimen.

The windows-based computer program "Response 2000" [13] was used in the current investigation to predict the response of the tested specimens using modified compression field theory (MCFT) [12].The program outputs the ultimate load at failure, the load deflection graphs and the failure crack pattern. A comparison between test results and Response 2000 [13] outputs is also presented in this section.

A comparison between test results of the failure load and the prediction using different building codes such as ECP 203-2007 [1], ACI 318-02 [5], EN1992 [9], ASHTO-LRFD [10] and CSA 2004 [11] is also presented in this section.

Cracking Pattern and Mode of Failure:

For all specimens, the first crack development, crack propagation, and plane of failure were observed during the test. As stated before; all tested specimens were designed to fail in one way shear. This presumption was investigated for all tested specimens.

The general behavior of all tested specimens was relatively similar and the crack development followed a similar pattern in all tested specimens. All beam specimens failed in shear and shear cracks crossed the compression zone of beam section.

It was observed that the first batch of cracks was vertical flexural cracks occurred in the specimens mid span and near mid span section. No crack has been witnessed at ends of beam along 450 mm of each side - outside flange zone. Upon increasing the applied load, new series of flexural cracks was formed at the bottom in the shear span region and gradually propagated towards the compression flange then rotated towards the two loading points while no crack had been witnessed at beam ends. By increasing the applied load and at intermediate loading stages, a new series of flexural cracks was formed in the shear span region then rotated to form flexural – shear cracks; joining the loading and supporting points.

During subsequent loading stages, additional diagonal shear cracks appeared and developed through a substantial depth of the specimen section, and propagated towards the top compression flange. Cracks continued propagating horizontally just underneath the flanges towards the loading points as shown in Fig. (6).

Generally, there were two phases after flexural cracking, starting from the first shear crack up to the shear failure: the first phase is the cracking formation phase; in which new shear cracks occur, and the second phase is the stabilizing cracking phase; in which only the shear cracks widen until reaching shear failure due to significant widening of shear cracks.

Table 3 summarizes the results of the twelve tested beam specimens. The table gives the main characteristics of each specimen, its flexural cracking load, shear cracking load, and the failure shear load.

Fig. 6, Shear Cracks propagation

Figs. 7 and 8 show the experimental and the predicted failure cracking patterns for all specimens. It should be noted that in experimental results; the load is recorded along cracks to show crack propagation history. However, the failure cracking pattern as output by *Response 2000* program shows the crack width (mm) along the crack at failure stage. So, the comparison between the two crack patterns; the experimental and the analytical, can be carried out only on the context of the general distribution and extension of cracks.

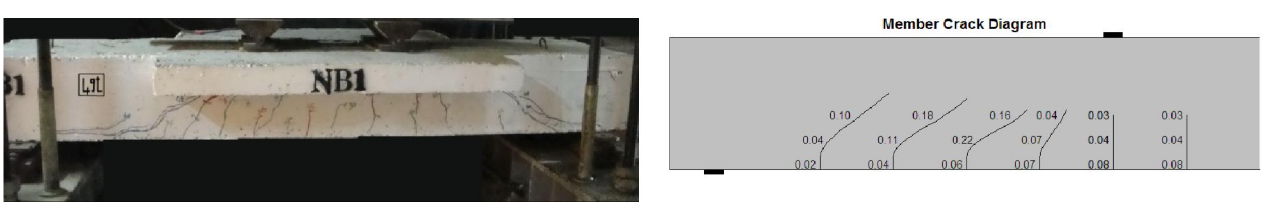


Fig. 7a, Experimental (left) and Analytical (right) crack pattern for Specimen NB1.

Fig. 7b, Experimental (left) and Analytical (right) crack pattern for Specimen NB2.

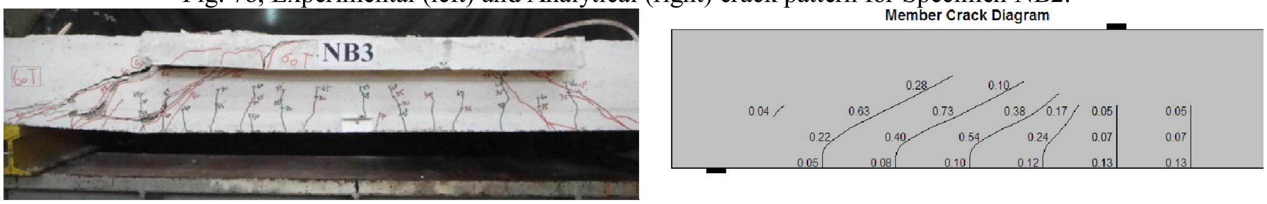


Fig. 7c, Experimental (left) and Analytical (right) crack pattern for Specimen NB3.

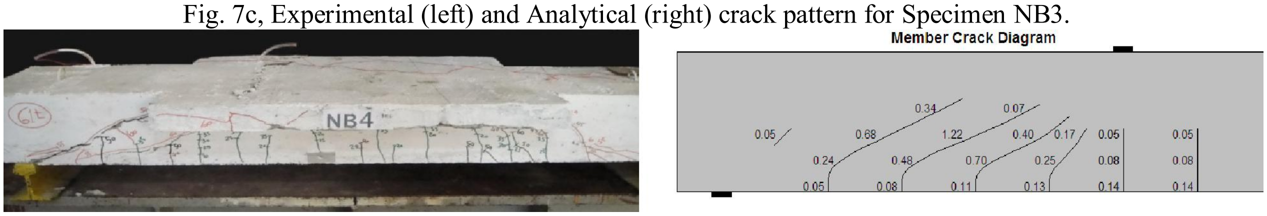


Fig. 7d, Experimental (left) and Analytical (right) crack pattern for Specimen NB4.

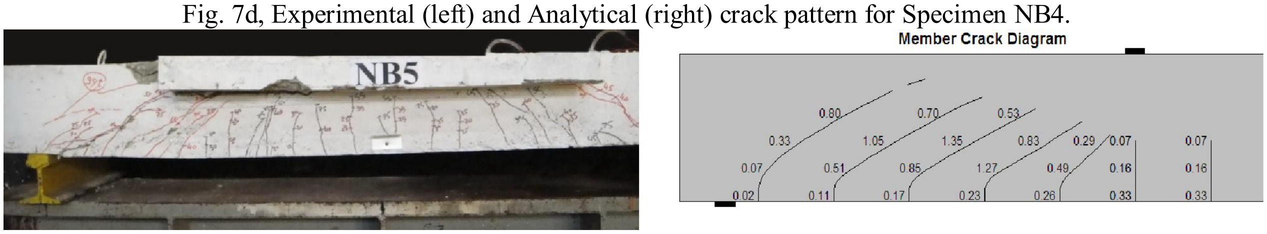


Fig. 7e, Experimental (left) and Analytical (right) crack pattern for Specimen NB5.

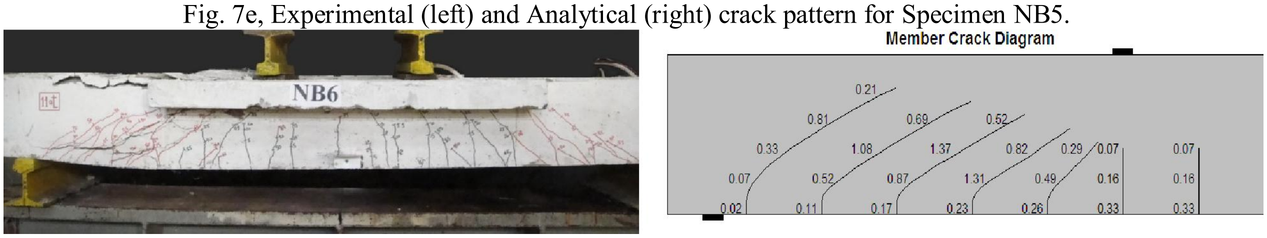


Fig. 7f, Experimental (left) and Analytical (right) crack pattern for Specimen NB6.

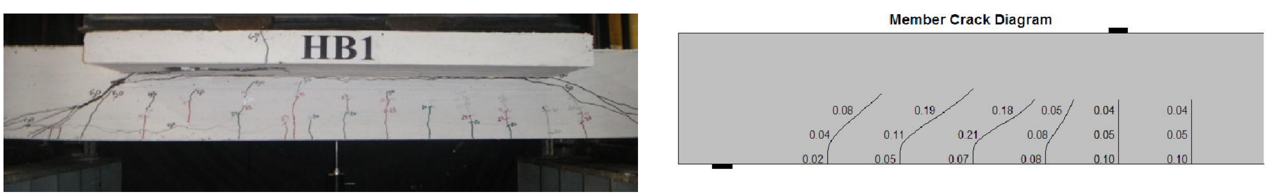


Fig. 8a, Experimental (left) and Analytical (right) crack pattern for Specimen HB1.

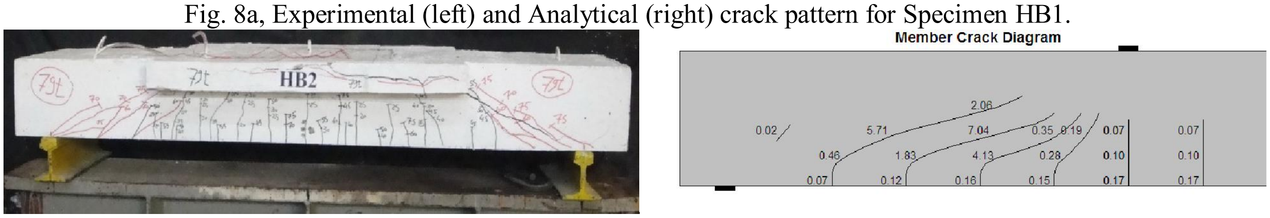


Fig. 8b, Experimental (left) and Analytical (right) crack pattern for Specimen HB2.

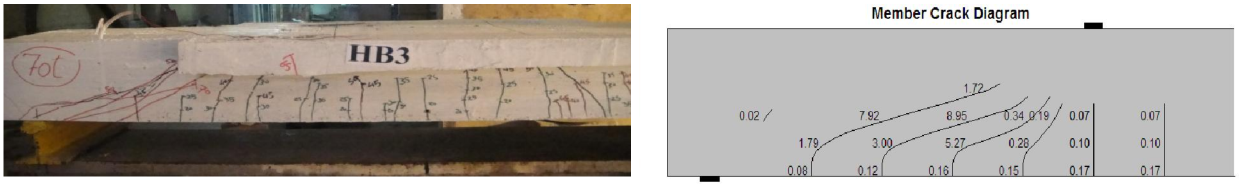


Fig. 8c, Experimental (left) and Analytical (right) crack pattern for Specimen HB3.

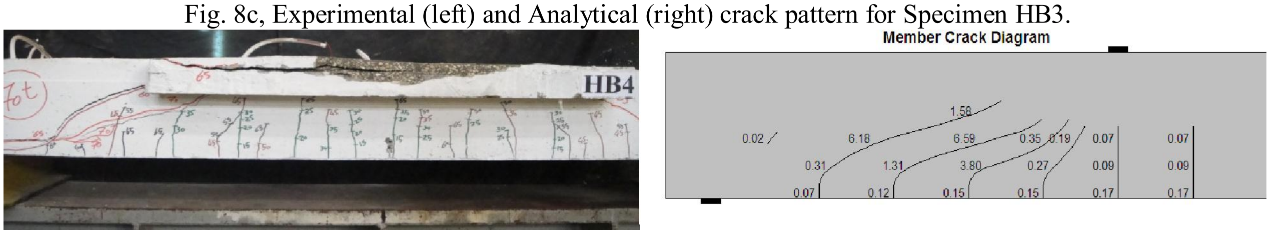


Fig. 8d, Experimental (left) and Analytical (right) crack pattern for Specimen HB4.

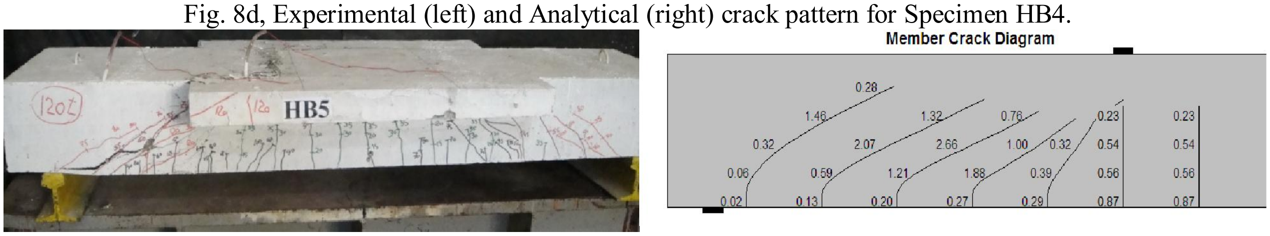


Fig. 8e, Experimental (left) and Analytical (right) crack pattern for Specimen HB5.

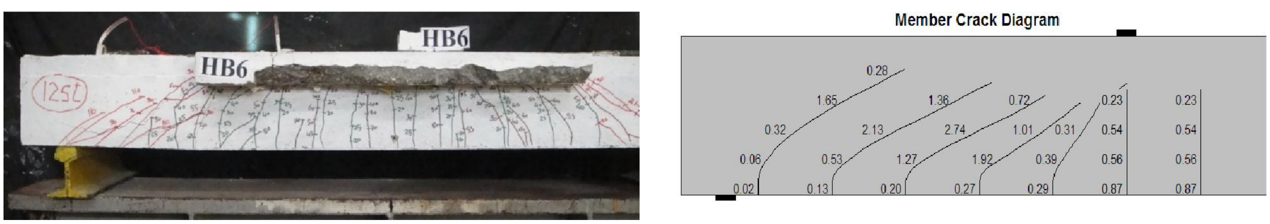


Fig. 8f, Experimental (left) and Analytical (right) crack pattern for Specimen HB6.

Load-Deflection Relationship:

Fig. 9 shows the load versus mid span deflection for the five tested groups. The curves show that the specimens exhibit three stages of behavior which are marked by a significant change in the slope of the load deflection curve.

Stage (1) which is the pre-cracking stage, starts from zero loading till the first cracking load. The behavior in this stage is characterized by the uncracked behavior where the maximum tensile stress is less than concrete flexural tensile strength (concrete modulus of rupture f_r). This is presented through the steep slope of the load deflection line where the deflection almost increased linearly with loading. The pre-cracking stage ends at the initiation of the first crack.

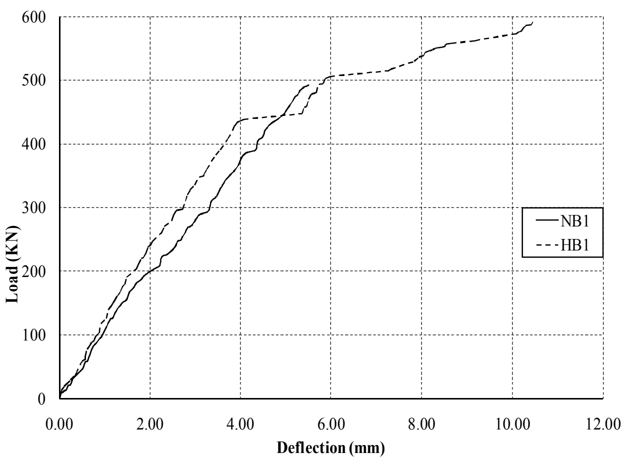
Stage (2) which is the post-cracking stage, begins with the first cracking in the mid span, the specimens behaves with a reduced stiffness compared to the slope of the load deflection line in the first stage where there were slight change in slope of the load deflection curve due to cracking. In this stage, the specimens developed a stable cracking in distribution and width. After cracking, deflections increased linearly with the load again.

Stage (3) which is the post-serviceability stage (steel yields), specimens in this stage behaved with significantly reduced flexural stiffness compared with the previous stages. This is presented through the near horizontal to horizontal load deflection curve in this stage due to substantial loss in stiffness of the specimens section, deeper and wider extensive cracks take placed till failure.

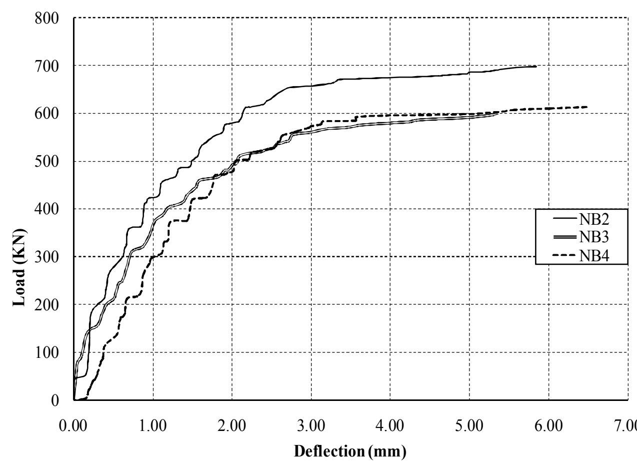
The load-deflection curves of the Normal Strength Concrete specimens show that shear reinforcement had no significant impact on the deflection values at any loading stage where all N.S.C specimens had approximately equal deflection value in different loading stage (maximum deflection value ranged from 5.5 mm to 7.5 mm) except specimen NB6 which developed about 13.60 mm deflection at failure. Similar observation was recorded for H.S.C specimens. They had approximately equal deflection value in different loading stage (maximum deflection value ranged from 4.00 mm to 6.50 mm) except specimens HB1 and HB2 had approximately 10.00 mm deflection at failure. Table 4 summarizes the recorded deflection values at flexural cracking stage and at failure for all specimens. Fig. 10 shows a comparison between analytical and the experimental deflection of the five tested groups.

Table 4 Summary of recorded deflection values

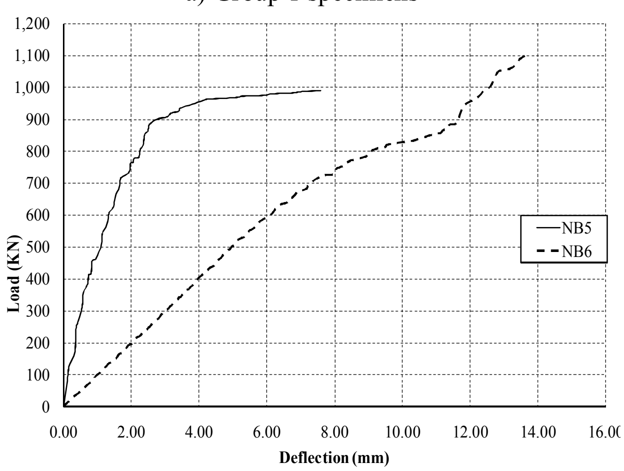
Normal Strength Concrete Specimens			*High Strength Concrete Specimens*		
Specimen	$\Delta_{cracking}$ (mm)	$\Delta_{failure}$ (mm)	Specimen	$\Delta_{cracking}$ (mm)	$\Delta_{failure}$ (mm)
NB1	3.00	5.50	HB1	1.70	10.50
NB2	3.70	5.85	HB2	8.00	9.63
NB3	4.00	5.35	HB3	2.85	4.40
NB4	4.70	6.50	HB4	4.00	5.50
NB5	4.70	7.50	HB5	4.50	6.50
NB6	1.00	13.60	HB6	4.60	6.30



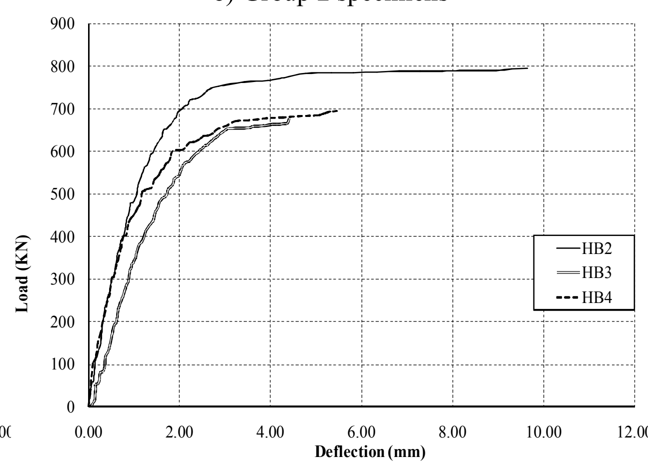
a) Group 1 specimens



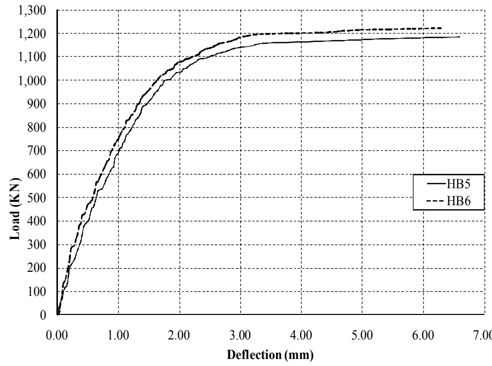
b) Group 2 specimens



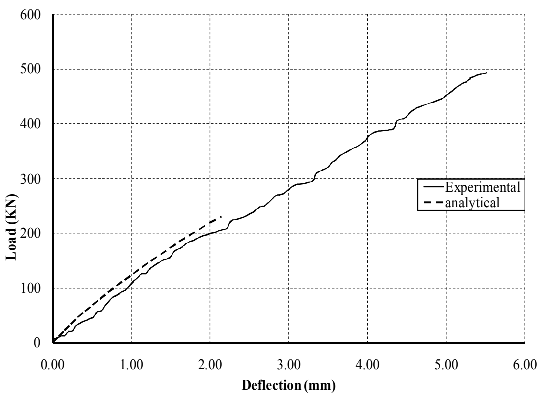
c) Group 3 specimens



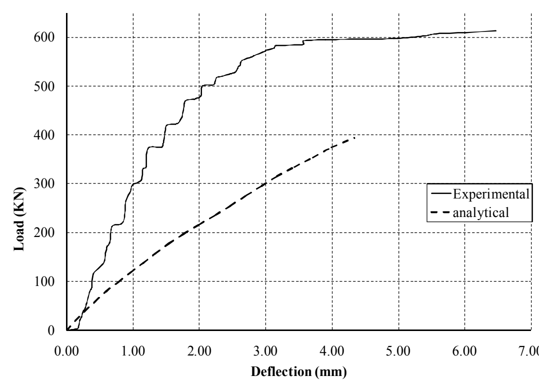
d) Group 4 specimens



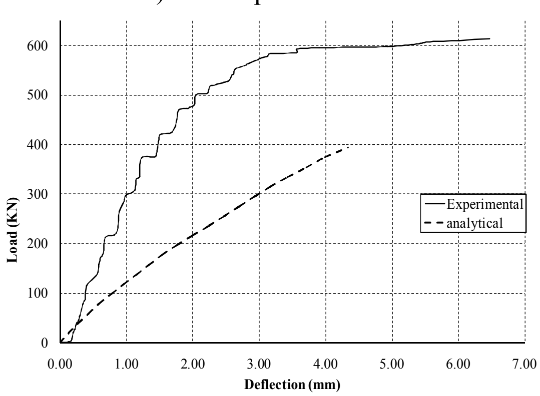
e) Group 5 specimens
Fig. 9, Load Deflection Curves



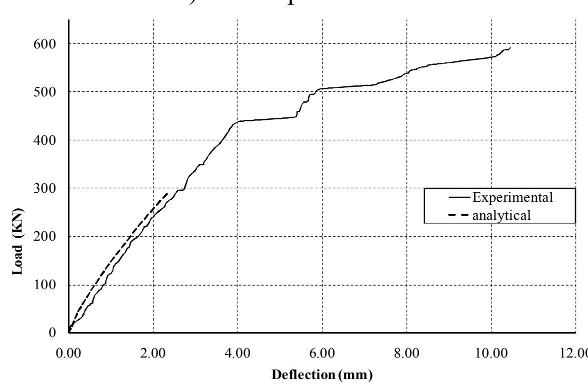
a) N.S.C specimen “NB1”



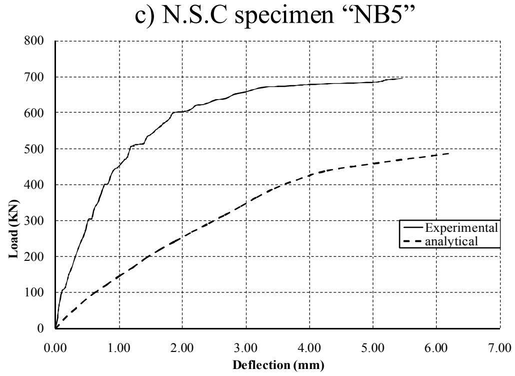
b) N.S.C specimen “NB4”



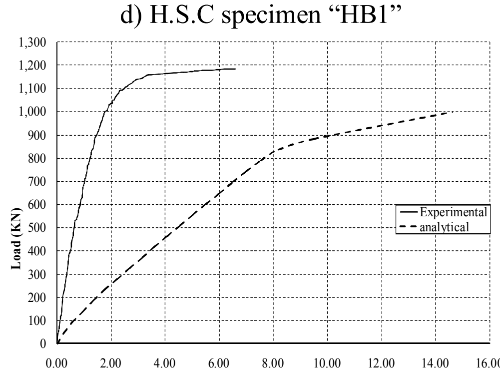
c) N.S.C specimen “NB5”



d) H.S.C specimen “HB1”



e) H.S.C specimen “HB4”



f) H.S.C specimen “HB5”

Fig. 10, Analytical versus the experimental deflection

The ductility can either be represented in terms of the ratio of maximum displacement to the yield displacement; both measured at mid span, or in terms of the stain energy consumed by the specimen during the test measured as the area under the load displacement curve. Since the flexure mode of failure has been secured for all specimens to allow for shear mode of failure, it is found more appropriate to use the second measure of ductility. Figs. 11&12 show ductility measure for all tested specimens. It easily be seen that the increase in web reinforcement generally increases the ductility for both N.S.C. and H.S.C. As the web reinforcement increases beyond the minimum ratio, its effect becomes more pronounced lower concrete strength. However, with no web reinforcement, the ductility increases as concrete strength increases.

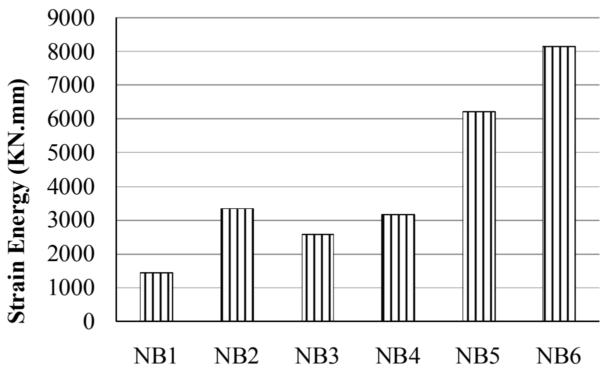


Fig. 11, Ductility measure of NSC specimens

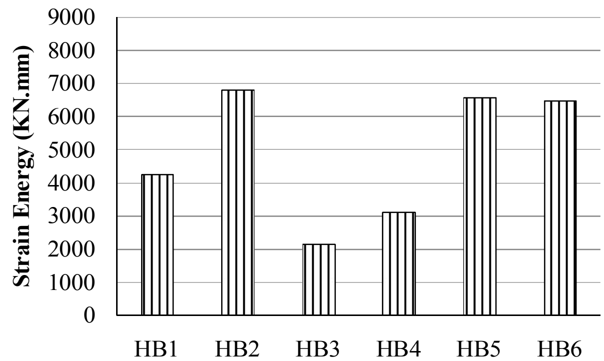


Fig. 12, Ductility measure of HSC specimens

Effect of web reinforcement

The effect in shear capacity due to the presence of the web reinforcement can be concluded as shown in Table 5. It can seen that the increase is evident for both normal and high strength concrete specimens. It is also evident that with minimum web reinforcement, the effect is more pronounced with normal strength concrete. However, as volume of web reinforcement increase, concrete strength has almost no significance.

Table 5. Increase in failure load due to the presence on web reinforcement.

Normal Strength Concrete Specimens			*High Strength Concrete Specimens*		
Specimen	*Experimental Failure Load (KN)*	*% increase Failure load*	*Specimen*	*Experimental Failure Load (KN)*	*% increase Failure load*
NB1	490.0	0	HB1	590.0	0
NB2	700.0	43	HB2	795.0	35
NB3	600.0	22	HB3	680.0	15
NB4	610.0	24	HB4	700.0	19
NB5	990.0	102	HB5	1190.0	102
NB6	1100.0	124	HB6	1220.0	107

Strains in Stirrups

Two electrical strain gages were attached to stirrups vertical branches per specimen, one strain gage was fixed closer to loading point and the other strain gage was fixed closer to support .Only specimens NB3 and HB3 had only one strain gage closer to support. Curves of load–maximum tensile strain in stirrups showed that there are two stages of behavior common between all specimens: Stage (1) is before shear crack load is reached. The strains were compression with small values (less than 100 micro - strain). Compression strains resulted from applying the load on the top surface of the specimens. Stage (2) starts after shear crack load is reached. The stirrups developed tensile strains, thus indicating that the stirrups were successful in resisting the shear stresses in test specimens. The rate of strain increase was small just after formation of first shear crack and increased rapidly when specimens approached failure load. Table 6 summarizes the recorded stirrups strain values as a percentage from stirrups yielding strain for shear tension failure and shear compression failure specimens.

Table 6 Summary of recorded stirrups strain values

Shear- Tension Failure		*Shear- Compression Failure*	
Specimen	$\varepsilon_{failure}$ *	*Specimen*	$\varepsilon_{failure}$ *
NB2	600%ε_{yield}	NB3	15.50%ε_{yield}
NB4	150%ε_{yield}	NB5	52.50%ε_{yield}
HB2	145%ε_{yield}	NB6	40.00%ε_{yield}
HB3	200%ε_{yield}	HB4	87.50%ε_{yield}
HB5	140%ε_{yield}		
HB6	105%ε_{yield}		

* ε_{yield} = 1600 macro strain for mild steel
ε_{yield} = 2000 macro strain for high grade steel

Fig. 13 plots the load strain relationships in the stirrups vertical branches for shear tension failure specimens, the highest recorded strains per specimen are plotted in the figure. No Load-strain curve was plotted for shear compression failure specimens.

Comparison between Test Results and Code Prediction for Shear Strength

Tables (7) and (8) summarize the shear capacity predictions for normal strength concrete and high strength concrete specimens respectively using the above mentioned international codes and also provide the shear capacities obtained from experimental testing. It should be noted that the experimental shear capacity $V_{exp.}$ shown in theses tables is half the failure load of the specimen shown in table 3.

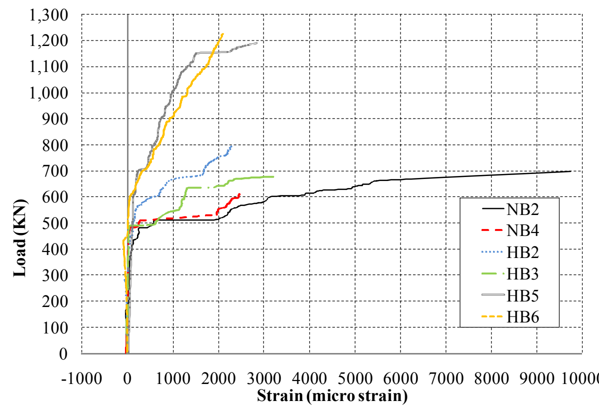


Fig. 13, Load strain in stirrup

Table 7 Summary of codes' prediction of shear capacity for N.S.C specimens.

Group	*Specimen*	*Predicted Shear Capacity "$V_{predicted}$" (KN)* ECP 203-2007	*ACI 318-02*	*EN1992*	*ASHTO LRFD*	*CSA 2004*	*Experimental Shear Capacity "$V_{exp.}$" (KN*
G1	NB1	88.50	128.00	97.50	84.50	115.00	246.50
G2	NB2	88.50	173.20	107.00	180.00	187.50	350.00
	NB3	88.50	173.20	107.00	180.00	183.50	300.00
	NB4	88.50	173.20	107.00	180.00	197.00	306.50
G3	NB5	88.50	278.00	360.00	305.00	395.00	495.00
	NB6	88.50	278.00	360.00	305.00	395.00	550.00

Table 8 Summary of codes' predictions of shear capacity for H.S.C specimens.

Group	*Specimen*	*Predicted Shear Capacity "$V_{predicted}$" (KN)* ECP 203-2007	*ACI 318-02*	*EN1992*	*ASHTO LRFD***	*CSA 2004***	*Experimental Shear Capacity "$V_{exp.}$" (KN)*
G1	HB1	132.50	157.50**	129.30	125.00	148.00	296.00
G4	HB2	132.50	205.00	107.00	230.00	248.00	398.00
	HB3	132.50	205.00	107.00	230.00	244.00	339.00
	HB4	132.50	205.00	107.00	230.00	243.00	350.00
G5	HB5	132.50	310.00	360.00	335.00	500.00	593.00
	HB6	132.50	310.00	360.00	335.00	500.00	612.00

** Value was calculated based on f_c'=70 MPa

Tables (9) and (10) summarize the comparison between the experimental shear capacity and predictions using the current international codes; "$V_{exp.}/V_{predicted}$", for normal strength concrete specimens and high strength concrete specimens respectively.

Table 9 :Experimental shear capacity versus codes' prediction; "$V_{exp.}/V_{predicted}$", for N.S.C specimens

		"$V_{exp.}/V_{predicted}$"				
Group	*Specimen*	*ECP 203-2007*	*ACI 318-02*	*EN1992*	*ASHTO LRFD*	*CSA 2004*
G1	NB1	2.79	1.93	2.53	2.92	2.14
G2	NB2	3.95	2.02	3.27	1.94	1.87
	NB3	3.39	1.73	2.80	1.67	1.63
	NB4	3.46	1.77	2.86	1.70	1.56
G3	NB5	5.59	1.78	1.38	1.62	1.25
	NB6	6.21	1.98	1.53	1.80	1.39

Table 10: Experimental shear capacity versus codes' prediction; "$V_{exp.}/V_{predicted}$", for H.S.C specimens

		"$V_{exp.}/V_{predicted}$"				
Group	*Specimen*	*ECP 203-2007*	*ACI 318-02*	*EN1992*	*ASHTO LRFD***	*CSA 2004***
G1	HB1	2.23	1.88**	2.29	2.37	2.00
G4	HB2	3.00	1.94	3.72	1.73	1.60
	HB3	2.56	1.65	3.17	1.47	1.39
	HB4	2.64	1.71	3.27	1.52	1.44
G5	HB5	4.48	1.91	1.65	1.77	1.19
	HB6	4.62	1.97	1.70	1.83	1.22

** Value was calculated based on $f_c^{'}$=70 MPa

The predictions made by Canadian code CSA 2004 [11] –calculated by *Response 2000* program -correlate much better with the experimental results than various results given by the other codes.

For the six normal strength concrete specimens, the average "$V_{exp}./V_{predicted}$" ratio is 4.25 for ECP 203-2007 [1], 1.85 for ACI 318-02 [5], 2.40 for EN1992 [9] , 1.95 for ASHTO-LRFD [10] and 1.65 for CSA 2004 [11] "*Response 2000* program [13]". For the six high strength concrete specimens, the average "$V_{exp}./V_{predicted}$" ratio is 3.25 for ECP 203-2007 [1], 1.85 for ACI318-02 [5], 2.65 for EN1992 [9] , 1.80 for ASHTO-LRFD [10] and 1.45 for CSA 2004 [11] "*Response 2000* program [13]".

One can see that ECP 203-2007 [1] had the highest average value of "$V_{exp}./V_{predicted}$" ratio in both normal and high strength concrete specimens while CSA 2004 [11] had the lowest average value of "$V_{exp}./V_{predicted}$" ratio in both normal and high strength concrete specimens. This conclusion confirms the fact that the contribution of web reinforcement in enhancing shear capacity of shallow wide beams cannot be ignored especially for normal strength concrete.

It can easily be noticed that (ECP 203-2007) [1], ASHTO-LRFD [10] and CSA 2004 [11] achieved a higher average value of "$V_{exp}./V_{predicted}$" ratio in normal strength concrete specimens compared to the average value in high strength concrete specimens. On the contrary, EN1992 [9] achieved a higher average value of "$V_{exp}./V_{predicted}$" ratio in high strength concrete specimens compared to the value in normal strength concrete specimens, while ACI318-02 [5] had the same average value of "$V_{exp}./V_{predicted}$" ratio in both normal and high strength concrete specimens.

Fig. 14 shows the comparison between "$V_{exp.}/V_{predicted}$" ratios of groups 1 (NB1), 2 and 3 specimens. One can see that the predictions made by (ECP 203-2007) [1] correlate much better with members without stirrups than with members with stirrups. That is due to the fact that (ECP 203-2007) [1] totally discard the shear reinforcement contribution in shallow wide beam shear resistance. On the contrary the predictions made by CSA 2004 [11] "*Response 2000* program [13] " correlate much better with members with stirrups amount more than the minimum, than with members without stirrups due to the fact that CSA 2004 [11] totally acknowledge such contribution. EN1992 [9] achieved the highest value of "$V_{exp}./V_{predicted}$" ratio in group 2 (specimens with minimum stirrups) compared to the values in group 1(NB1) and 3 which can be attributed to the fact that EN1992 [9] discard the concrete contribution in beams shear resistance. Fig. 15 shows that similar conclusion can be drawn for High Strength Concrete specimens.

CONCLUSION

Based on the experimental results and the observed behavior, the following conclusions may be made:

1. Contribution of web reinforcement- in form of vertical stirrups- in shear strength of shallow wide beams cannot be ignored.
2. H.S.C shallow wide beams without web reinforcement presented a more ductile behavior compared to N.S.C beams. On the other hand, H.S.C beams with stirrups, twice as much as the minimum web reinforcement, exhibited a less ductile behavior.
3. For shallow wide beams without web reinforcement, the shear strength generally increases as the concrete compressive strength

increased.

4. The effect of web reinforcement on improving shear strength is more pronounced at lower compressive strength of concrete and low web reinforcement ratio.
5. The influence of stirrups amount on shear strength does not vary according to concrete compressive strength.
6. The spacing between vertical stirrups and branches number of stirrups in cross section have a less effect in improving shear capacity as concrete strength increases.
7. The shear reinforcement significantly enhances the ductility of the shallow wide beams with normal strength concrete. This effect is less pronounced with high strength concrete.
8. The predictions made by Canadian code CSA based on MCFT – calculated by *Response 2000* program - correlate much better with the experimental results than the various results given by the other codes while still provide an average factor of safety around 1.5 for H.S.C. and 1.68 for N.S.C.

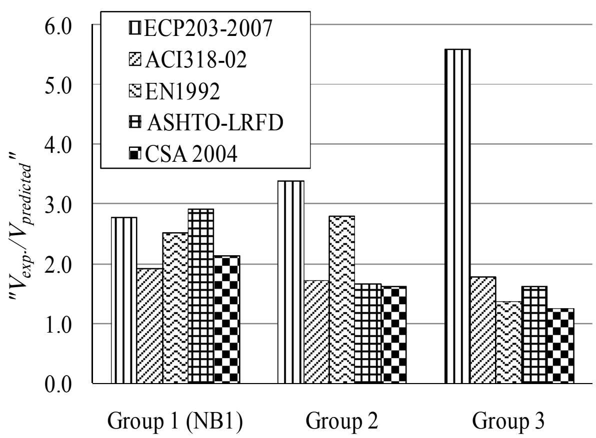


Fig. 14 "$V_{exp.}/V_{predicted}$" Ratio for Normal Strength Concrete

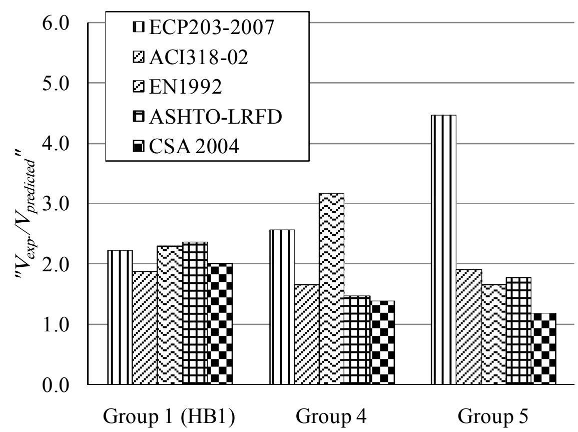


Fig. 15 "$V_{exp.}/V_{predicted}$" Ratio for Normal Strength Concrete

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6/6/2012