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Experimental Investigation on Continuous Reinforced SCC Deep Beams and Comparisons with Code Provisions and Models

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ABSTRACT

The test results on eight two-span deep beams made of self-compacting concrete (SCC) are presented and discussed in this paper. The main parameters investigated were the shear span-to-depth ratio, and the amount and configuration of steel reinforcement. All beams failed due to a major diagonal crack formed between the applied mid-span load and the intermediate support separating the beam into two blocks: the first one rotated around the end support leaving the other block resting on the other two supports. Both concrete compressive strength and web reinforcement had a major effect in controlling the shear capacity of the beams tested. For the shear span-to-depth ratio considered, the vertical web reinforcement had more influence on the shear capacity of the specimens than the horizontal web reinforcement. The shear provisions of the ACI 318M-11 are unconservative for most of the beams tested. Comparisons of test results with the strut-and-tie model (STM) suggested by ACI 318M-11, EC2 and CSA23.4-04 showed that the predictions are reasonable for continuous deep beams made with low and medium compressive strength. Although the equation suggested by ACI 318M-11 is very simple, its prediction is more accurate than the STM suggested by different design codes.

Abbreviations

SCC	Self-compacting concrete
NC	Normal concrete
STM	Strut-and-tie model
l	Beam span
b	Width of beam section
a/d	Shear span-to-depth ratio
d	Effective depth of Beam
h	Overall depth of beam section
f_c'	Cylinder compressive strength of concrete
f_y	Yield strength of steel reinforcement
E_s	Elastic modulus of steel reinforcement
P_t	Failure load
V	Shear capacity
v_n	Normalised shear strength
v	Effectiveness factor of concrete

1. Introduction

Reinforced concrete deep beams are a commonly-used structural member, especially when free space among the columns is required. They are used in different civil engineering applications such as stores, hotels, offshore structures, theatres, tanks, pile caps and others. In practice, continuously-supported deep beams are often used in constructions as an alternative to simply-supported beams. However, all previous investigations have been conducted on simply supported SCC beams [1-6]. In contrast, there are no research investigations on continuous reinforced self-compacting concrete (SCC) deep beams. This area of research is of special interest due to the high depth of deep beams and congested steel reinforcement, making it difficult for normal concrete (NC) to properly be placed and vibrated. SCC provides higher quality, improves productivity and achieves engineering properties similar to those of NC but more durable structures. Moreover, the lower amount and smaller size of coarse aggregate used in SCC lead to different behaviour compared with NC. The lower amount of coarse aggregate in SCC leads to more brittle behaviour as cracks can propagate further through the paste or mortar phase before stopped or diverted by a coarse aggregate particle. Furthermore, inadequate vibration causes high surface permeability, unfilled voids and micro-pores within NC which, in turn, results in negative effects on mechanical properties and durability of NC. SCC requires no vibration as it can easily flow and be placed under its self-weight with excellent surface finishes and homogenous distribution of concrete within the formwork, to the advantage of durability, thanks to concrete lower permeability.

The loads applied at the extrados (top side) in deep beams are transferred to the reaction points through compression struts formed between the loads and the supports. The load carrying capacity of deep beams is controlled by their shear resistance. The failure mode of continuous deep beams is significantly different from that of simply supported deep beams or that of shallow beams. The failure in continuous deep beams generally occurs in regions where high shear simultaneously occurs with high bending moment, whereas in simply supported deep beams the high shear and high moment do not necessarily occur in the same region, depending on the loading arrangement [7, 8].

Although the current design codes introduce limits on the clear span-to-depth ratio to define deep beams, there is disagreement on the limiting value. The shear provisions of the ACI Building Code (ACI 318M-11)[9] defines deep beams as a member whose clear span is equal to or less than four times its overall depth and, in another clause, as a member having a shear span-to-depth ratio less than 2. In contrast, the Euro Code 2 (EC2)[10] considers any beam for which the span is not greater than three times its overall depth as a deep beam. On the other hand, the Canadian Standard (CSA23.3-04)[11] classifies members having a clear span to overall depth ratio less than two as deep beams. Furthermore, most design codes recommend the use of strut-and-tie model (STM) for the design of reinforced concrete deep beams. However, they propose different values for the concrete effectiveness factor calibrated against tests of simply supported NC deep beams.

The load transfer mechanisms in deep beams are somewhat different from those typically found in shallow beams. The main load transfer element in deep beams is a concrete strut formed between the loading point and support [7, 8, 12]. Experimental results showed that there was a 35 to 45% reduction in shear strength of simply supported SCC deep beams and higher mid-span deflection compared with their NC counterparts [6] owing to the lower amount and smaller size of coarse aggregate used in SCC mixtures. A number of studies on simply supported SCC deep beams [5, 13] pointed out that the experimental shear capacity is much higher than that predicted by the shear provisions of the ACI 318-08. However, the prediction of the STM suggested by the ACI 318-08 was reasonably close to the experimental results of simply supported SCC deep beams [5].

Although there are some existing investigations on continuous reinforced concrete deep beams, all of these studies have focused on deep beams made of NC. The load capacity of continuous NC deep beams depends on the web reinforcement which in turn depends on the shear span-to-depth ratio [14]. It was shown [15, 16] that the vertical web reinforcement is more effective for continuous deep beams having shear span-to-depth ratio greater than 1.0, otherwise the horizontal web reinforcement is more effective. The predictions of load carrying capacity of continuous concrete deep beams using the strut-and-tie model of ACI 318-05 were found to be unconservative with the increase of the shear span-to-depth ratio and this unconservatism was more pronounced in continuous deep beams than in simple ones [16].

The present paper reports the experimental results of eight two-span reinforced SCC deep beams. Two shear span-to-depth ratios were considered. Different amounts and configuration of web reinforcement were investigated. The test results were then compared with shear provisions of the ACI 318M-11 as well as with the STM recommended by three different design codes, namely the ACI Building Code (318M-11), the Euro Code 2 (EC2) and the Canadian Standard (CSA23.4-04).

2. Experimental programme

2.1. Test specimens

The test specimens consisted of eight continuous SCC deep beams. The overall geometrical dimensions along with the reinforcement details for all specimens are presented in Table 1, Figure 1 and Figure 2. All beams had the same overall length, L=2750 mm, the same clear span, l=1240 mm, and the same width, b=160 mm. The main parameters investigated were the shear span-to-depth ratio a/d, the amount and configuration of the web reinforcement and main longitudinal reinforcement ratio. The overall depth, h, of the specimens was changed to achieve two different a/d ratios, namely 0.8 (h=600 mm) and 1.7 (h=300 mm). Beams B1 to

B6 had the same overall depth of 600 mm and a/d ratio of 0.8 whereas beams B7 and B8 had a depth of 300 mm and a/d ratio of 1.7.

With regard to the reinforcement, as shown in Table 1 and Figure 2, all test specimens had the same top and bottom main longitudinal reinforcement of 3 bars of 16 mm diameter except for two beams (B6 and B8) in which the amounts of top and bottom reinforcement were increased to 5 bars of 16 mm diameter. All the bottom reinforcing bars were extended to the full length and depth of the beam to ensure sufficient anchorage. The concrete cover to the centre of the main longitudinal bars was 40 mm while the clear cover to the face of the stirrups was 25 mm. The vertical web reinforcement consisting of 8 mm closed stirrups distributed uniformly along the beam length was varied among the test specimens. Specimen B1 had no vertical web reinforcement, specimen B5 had a high amount of vertical web reinforcement (28 vertical stirrups spaced at 100 mm) and all other specimens had an intermediate amount of vertical web reinforcement (14 vertical stirrups spaced at 200 mm). The horizontal web reinforcement of 8 mm diameter was provided on both sides of the beam web at three different levels: none (B2), 2 horizontal bars (B3, B5 and B6) and 4 horizontal bars (B4) on each side.

2.2. Material properties

SCC was produced in the laboratory using readily available raw materials. The concrete ingredients were ordinary Portland cement (PC, class 52.5N), fly ash 450-S (BS EN 450-1 Fineness Category S), 10 mm coarse aggregate, fine aggregate and superplasticizer. All the test specimens were cast in a vertical position using the same wooden mould. The fresh properties, which included the flowability, filling ability, passing ability and segregation resistance, were assessed by the slump flow, the V-funnel tests, the T_{50} test and sieve stability test, respectively. The results of the fresh properties compared to the requirements of the European Guidelines for SCC[17] are shown in Table 2.

The cylinder compressive strength f_c' was measured by testing four 300 mm high by 150 mm diameter cylinders under direct compression for each continuous beam. After demoulding, all the beams and control specimens were stored in the same place in the lab and covered by a polyethylene sheet up to the testing date. The cylinders were tested on the same day as the deep beam test and the results of compressive strength for each test specimen are shown in Table 1.

The mechanical properties of the longitudinal and web reinforcement were provided by the supplier of the steel reinforcing bars. The main longitudinal and web reinforcement have a yield strength, f_y, of 500 MPa and ultimate strength, f_{su}, of 675 MPa. The modulus of elasticity of the reinforcing bars, E_s, is 198 GPa.

2.3. Test set-up

All the specimens were tested under a symmetrical two-point loading system, using a loading frame of a capacity of 2500 kN as shown in Figure 3. After each load increment of 20 kN, the load was kept constant to observe how cracks develop. The middle support was designed to allow rotations only but no horizontal displacements whereas the two end-supports were designed as rollers to allow rotation and horizontal displacements. To avoid concrete bearing failure at the load application points, steel plates were used between the supports and the test specimens. The two end steel plates had a width of 120 mm while the middle and loading steel plates had a width of 200 mm. All the steel plates had a minimum length of 160 mm to cover the full width of the beam and a thickness of 20 mm except for the loading plates which were 40 mm thick. A top steel spreader beam was used to distribute the load from the loading actuator into two point loads.

2.4. Instrumentation

Strain gauges of 5 mm were attached to the main longitudinal and web reinforcement at the most critical locations: six used for beams having either vertical or horizontal web reinforcement and ten for beams having vertical and horizontal reinforcement. The mid-span deflection of each span and the support settlements were measured using linear variable differential transducers (LVDT). One end support reaction was measured using a 1000 kN capacity load cell. The test results from strain gauges, LVDTs and load cell were captured automatically using a data logger. The surface of the test specimens was painted to mark the development of cracking. Three highly-professional cameras were used to capture the flexural cracks in the mid span as well as the diagonal cracks formed between the intermediate support and load plates. The photos captured by these cameras were then employed to estimate the crack widths by applying Image-Pro Plus software version 6.0.

3. Test results and discussions

3.1. Cracking propagation and failure modes

For beams having an a/d ratio of 0.8 (Beams B1 to B6), the flexural cracks at mid-span and above the intermediate support occurred at approximately 12-17% and 60-70% of the failure load, respectively, while for beams B7 and B8 (a/d=1.7) both the first flexural crack over the intermediate support and that in the mid-span occurred at about 13% of the failure load. On the other hand, the first diagonal crack in most of the test specimens started at 30-40% of the failure load as presented in Table 3. The first flexural crack load at mid-span for all specimens was approximately half of that of the first diagonal crack. The diagonal crack occurred suddenly at the mid-depth of the beam between the load point and the intermediate support. After increasing the load, the length and width of the first crack increased and more diagonal and flexural cracks developed. In all beams, the crack patterns were similar in both spans.

All the specimens showed the same failure mode. The main cause of failure was a major diagonal crack started at the mid-depth of beams and extended along the distance between the edge of the load and intermediate support plates as shown in Figure 4: Fig. 4(a) for beam B4 with h=600 mm and Fig. 4(b) for beam B8 with h=300 mm. At failure, concrete crushing occurred at the top of the beams at the contact point between the diagonal crack and the load plate. The significant diagonal crack separated the beam into two concrete blocks: one rotated about the exterior support while the other was fixed over the other two supports similar to the failure mode observed in other investigations for continuous NC deep beams [7, 12, 16].

3.2. Width of diagonal and flexural cracks

The relations between the total applied load and the width of cracks are shown in Figure 5: Fig. 5(a) for the mid-span flexural crack and Fig. 5(b) for the diagonal crack. The limitation for the flexural crack width of 0.4 mm according to EC2 is also plotted in Fig. 5(a). Three high quality digital cameras were used to capture three cracks, namely the main flexural crack at each mid-span and the diagonal crack between the mid-span point load and intermediate support. The images of the cameras were then processed by Image-Pro Plus software version 6.0 to estimate the crack widths. Only one flexural crack is presented in Fig. 5(a) due to the similarity in crack widths between the two spans. It can be observed that the horizontal web reinforcement played an important part in decreasing the width of flexural and diagonal cracks. Beams with horizontal or orthogonal web reinforcement (B1, B3, B4, B5 and B6) had thinner cracks than beam B2 provided with vertical stirrups only, different from the results obtained by Yang et al.[16] who showed that a smaller diagonal crack width was observed in beams having vertical or orthogonal web reinforcement. This may be attributed to the fact that *a/d* ratio considered in the study conducted by Yang et al. was 1.0 compared with 0.8 in the current study. Beams with a smaller depth (B7 and B8) had a higher crack width at a lower applied load. Moreover, by comparing B3 and B6, it can be clearly noticed that increasing the amount of the main longitudinal reinforcement had a clear effect on both flexural and diagonal crack widths. EC2 limits the width of flexural cracks in reinforced concrete members to 0.4 mm. However, ACI 318M-11 does not give any limits for the crack width and relates the acceptable crack width to the type of structure. Older provisions of the ACI Building Code (before 1990) limits the crack width to 0.4 mm, similar to the EC2 limit. Comparing the results in Figure 5(a) and Figure 8(a) shows that for all the specimens, the width of the main flexural crack at the mid-span exceeded the limit of 0.4 mm at the time when the bottom longitudinal reinforcement reached or were close to yielding. However, at the serviceability load of EC2 (0.67 of the failure load), the width of the main flexural crack exceeded the limit of 0.4 mm for four beams (B1, B2, B6 and B8) which had low amount of web reinforcement.

3.3. Support reactions and failure loads

In Figure 6, the total load is plotted against the load transferred to the end supports: Fig. 6(a) for beams having a depth of 600 mm and Fig. 6(b) for beams having a depth of 300 mm. The end-support reaction obtained from a linear elastic finite element (FE) analysis using ABAQUS software is also plotted in Figure 6. The concrete was modelled using 8-node linear brick, reduced integration element (C3D8R) whereas the reinforcing bars were modelled by a 2-node linear 3-D truss element (T3D2). The interaction between concrete and reinforcement was modelled by using the embedded region option available in ABAQUS 6.12 which represents perfect bond between concrete and reinforcement. Up to the first crack, the relationship between the total applied load and the end-support reaction is approximately the same as predicted by the linear FE analysis. However, after the formation of the first diagonal crack, the prediction of the end-support reaction by the FE analysis was slightly lower than the experimental values for all the deeper beams as shown in Figure 6(a). This can be attributed to the fact that after concrete cracking, the applied load is transferred by the stress in the tensile reinforcement leading to a change in the slope of the load-deflection curve. This means that after cracking the redistribution of stresses increases the end-support reaction more than that predicted by the linear elastic FE. However, the difference between the experimental end-support reaction and that predicted by the linear elastic FE was very small even at failure and did not exceed 10% in all beams. This indicates that, although the occurrence of the diagonal crack leads to reduction of the beam stiffness, the redistribution of loads is very limited. For the two shallower beams (B7 and B8), the occurrence of the first diagonal crack did not have much effect on the agreement between the FE prediction and the experimental results. The relationship between the end-support reaction and the failure load obtained in the current study was found to be similar to that observed by previous investigations for continuous NC deep beams [7, 12, 16].

Table 4 presents the total failure load P_t, the maximum shear force V and the normalised shear strength $v_1 = V/bhf_c'$ and $v_2 = V/bh\sqrt{f_c'}$. The main reason for normalising the shear capacity by bh is to eliminate the influence of size effect due to the change of the beam depth from 600 mm to 300 mm. It can be noticed that all the specimens had approximately the same normalised shear strength v_1 of 0.12 when normalised by f_c'. However, the normalised shear capacity v_2 by $\sqrt{f_c'}$ varies between 0.65 and 0.89. Therefore, the size effect seems to have little influence on the shear strength of the specimens due to the use of web reinforcement as also shown in previous research investigations on continuous NC deep beams [7, 12]. Depending on the normalised shear capacity, it can be concluded that the maximum load of the tested beams is influenced by the concrete compressive strength. However, comparing the load capacity of beam B4 (having high amount of horizontal web reinforcement) and that of beam B5 (having high amount of vertical web reinforcement), it can be concluded that the vertical web reinforcement had more influence on the capacity of continuous deep beams than the horizontal web reinforcement. The reason for choosing beams B4 and B5 for this comparison is that both beams have approximately the same concrete compressive strength.

3.4. Mid-span deflections

The mid-span deflections for all specimens versus the total applied load are shown in Figure 7: Fig. 7(a) for beams having h=600 mm and Fig. 7(b) for beams having h=300 mm. The deflections in the two spans were similar and therefore only the mid-span deflections of the failed span are presented. The mid-span deflection measurements were adjusted to take into consideration the interior and exterior support settlements as recorded by the LVDTs at their locations. Up to the development of the first diagonal crack, all specimens having the same depth had almost the same initial stiffness and consequently deflections, indicating that the initial stiffness is independent on the amount and configuration of web reinforcement. For beams having a smaller depth (B7 and B8), the initial stiffness was lower than that of the deeper beams. After the development of the first diagonal crack, the beam stiffness significantly decreased leading to an increase in the mid-span deflection. All the specimens showed very low ductility at failure irrespective of a/d ratio and amount and configuration of web reinforcement, showing different behaviour from continuous NC deep beams tested by Ashour [7] and Rogowsky et al. [8]. They observed that contiuous NC deep beams exhibited different degrees of ductility depending on the a/d ratio and the amount and configuration of web reinforcement.

3.5. Strains in steel reinforcement

The relationship between the strains in steel reinforcement and the total applied load is shown in Figure 8: Fig. 8(a) for strains in bottom longitudinal steel bars, Fig. 8(b) for strains in top longitudinal steel bars, Fig. 8(c) for strains in horizontal web reinforcement and Fig. 8(d) for strains in vertical web reinforcement. The number of strain gauges used in each beam was selected depending on the amount of web reinforcement. For all beams, strain gauges were attached to the web reinforcing bars in the two spans. The strain readings in the two spans were almost the same and therefore only one span strains is presented in Figure 8. The significant redistribution of the strains in the web and longitudinal reinforcement started after the formation of the first diagonal crack.

For all beams, the highest strains were recorded for the web reinforcing bars crossing the main diagonal crack formed between the load plate and the intermediate support. Most of the web reinforcing bars reached the yield strain. Moreover, the strain in the bottom longitudinal reinforcement reached the yield strain for all beams except for two beams (B7 and B8). However, none of the top reinforcing bars yielded as indicated in Fig. 8(b). In some cases, the strain gages might not have been placed in the exact position of the major diagonal crack and therefore, yield could have occurred even though not shown by the strain readings. However, comparing the strain results of beams B4 (having horizontal and vertical web reinforcement), it can be seen that the vertical web reinforcement reached the yield strain before the horizontal one which almost reached the yield strain at failure. Therefore, it can be concluded that the vertical web reinforcement is more effective in carrying loads than the horizontal web reinforcement for the shear span-to-depth ratio tested.

4. Experimental results compared with ACI-318M-11

In this section, comparisons between the experimental results and those predicted by the shear provisions of the ACI Building Code (318M-11) are presented. The provisions of the

ACI Building Code 318M-11 for shear strength of deep beams are applicable for a member with clear span (l_n) to overall depth ratio (h) not greater than 4. All the specimens satisfy the definition provided by the ACI Building Code for the deep beams. The provisions of ACI 318M-11 (Section11.7) assume that the total shear capacity of deep beams V_u can be calculated from equation (1) below:

$$V_u = 0.83\sqrt{f_c'}bd \tag{1}$$

where f_c' is the cylinder compressive strength of concrete in MPa, b is the beam width in mm and d is the beam effective depth in mm.

ACI 318M-11 also stated that to apply Eq. (1) in the prediction of shear strength of deep beams, the area of web reinforcement in both directions (perpendicular and parallel to the longitudinal reinforcement) shall not be less than $0.0025bS$, where S is the spacing between the vertical or horizontal web reinforcement bars. In the current study, four of the beams tested satisfy this condition, namely B3, B4, B5 and B6. Table 5 presents the comparison between the experimental results of the normalised shear capacity v_{nEXP} for the test specimens and that, v_{nACI}, predicted by the ACI 318M-11. The ratio between the shear strength obtained from the experimental results and that predicted by the ACI equation ranges from 0.845 to 1.145, with a mean value of 0.97, a standard deviation of 11% and a coefficient of variation of 11%. The predictions of the ACI 318M-11 are conservative only for three beams (B4, B5, and B6). For the remaining specimens (except beam B3), the unconservative predictions may be attributed to the fact that these beams had web reinforcement in one direction only, not satisfying the condition mentioned above. The discrepancy of the results between the tested beams can be attributed to the fact that the ACI equation determines the total shear capacity of deep beams depending only on the concrete compressive strength. Overall, although equation (1) is very simple, its predictions is very close to the experimental results.

5. Strut-and-tie model based on the current design codes

The current design codes [9-11] suggest that deep beams shall be designed using either nonlinear analysis or strut-and-tie model (STM). In this section, comparison between the experimental results and the STM suggested by different design codes are carried out, namely the ACI Building Code (318M-11)[9], the Euro Code 2 (EC2)[10] and the Canadian Standard for the Design of Concrete Structure (CSA23.3-04)[11]. The total applied load is estimated by using a set of equations based on a simple STM [16, 18] shown in Figure 9. For two spans continuous deep beams, the total load P_t due to the failure of concrete struts can be determined from equations (2) to (5) below:

$$P_t = 2vf_c'b\,[w_{ES} + w_{IS}]\,Sin\,(\theta) \tag{2}$$

$$w_{ES} = w_t\,Cos\,(\theta) + \frac{[l_{EP} + 0.5l_{LP}]}{2}\,Sin\,(\theta) \tag{3}$$

$$w_{IS} = w_t\,Cos\,(\theta) + \frac{[\,l_{LP} + l_{IP}]}{4}Sin\,(\theta) \tag{4}$$

$$\theta = tan^{-1}\frac{(h - c - c')}{a} \tag{5}$$

where v is the effectiveness factor of concrete, f_c' is the cylinder compressive strength of concrete, b is the beam width, w_{ES} is the average effective width of the exterior concrete strut, w_{IS} is the average effective width of the interior concrete strut, θ is the angle between the concrete strut and the longitudinal axis of the beam, l_{EP} is the width of the exterior bearing plate, l_{IP} is the width of the interior bearing plate, l_{LP} is the width of the load bearing plate, h is the total height of the beam, c and c' are the concrete covers of the bottom and top longitudinal reinforcement, respectively, a is the shear span and w_t is the effective tie width which equals twice the concrete cover ($w_t = 2c$).

In the above equations, the effectiveness factor of concrete (v) is the only difference among the three design codes considered in the current comparison. Each design code suggests a different value for the effectiveness factor as presented in Table 6.

Table 7 shows the comparison between the test results and those predicted by the STM provided by the current design codes considered. Moreover, the test results were compared with previous studies conducted by Yang et al.[12, 16] for continuous deep beams made with normal concrete as shown in Figure 10: Fig. 10(a) for SSC, Fig. 10(b) for NC having $f'_c < 60\ MPa$ and Fig. 10(c) for NC having $f'_c > 60\ MPa$. The ACI STM prediction was the closest to the current test results with an average of 1.15, a standard deviation of 4% and a coefficient of variation of 4%. The predictions of all considered codes were conservative for all SCC specimens. However, in case of NC beams, the predictions of ACI and EC2 are conservative for most of the beams. Moreover, the predictions of the Canadian Code underestimate the results of SCC beams and overestimate those of NC beams. It should be noted that most of the results overestimated by the three codes refer to deep beams made of high-strength concrete (f'_c ≥60 MPa). As a result, it can be suggested that a modified lower value for the concrete effectiveness factor is needed for f'_c> 60 MPa to adjust the prediction of STM for continuous deep beams.

6. Conclusions

Test results of eight continuous SCC deep beams have been presented. The parameters investigated were the shear span-to-depth ratio, and the amount and configuration of web reinforcement. Based on the work presented in this paper, the following conclusions are drawn:

- All the specimens failed due to a major diagonal crack formed between the load and inner support plates. At failure, the controlling diagonal crack caused the tested beams to separate into two concrete blocks: the first free to rotate about the outer support, and the second restrained by the two remaining supports, similar to that observed for continuous NC deep beams tested in other investigations.
- All the tested specimens failed in a brittle manner irrespective of a/d ratio and amount and configuration of web reinforcement.
- The load-end support reaction relation and the load-deflection response observed in the current study are similar to those observed by previous research investigations for continuous NC deep beams.
- The shear strength of the specimens is significantly controlled by the concrete compressive strength, and to a lesser degree by the amount and configuration of web reinforcement. For the shear span-to-depth ratio studied, the vertical web reinforcement had more effect on shear capacity than the horizontal web reinforcement.
- The highest strain was recorded in web reinforcement crossing the major diagonal crack between the load and inner support plates. Most of the web reinforcing bars crossing the diagonal crack at the failure region reached the yield strain.
- The simplified provisions of the ACI Building Code for the shear strength of continuous deep beams tend to slightly overestimate most of the test results.
- The strut-and-tie model recommended by different design codes showed conservative results for all specimens. The ACI Building Code (318M-11) predictions were more accurate than the EC2 and the Canadian Code (CSA23.3-04). The comparison between

the three design codes for two-span NC deep beams showed that the strut-and-tie model resulted in unsafe predictions for high strength concrete (more than 60 MPa).

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