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An experimental and numerical investigation on the effect of longitudinal reinforcements in torsional resistance of RC beams

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Abstract. It is evident that torsional resistance of a reinforced concrete (RC) member is attributed to both concrete and steel reinforcement. However, recent structural design codes neglect the contribution of concrete because of cracking. This paper reports on the results of an experimental and numerical investigation into the torsional capacity of concrete beams reinforced only by longitudinal rebars without transverse reinforcement. The experimental investigation involves six specimens tested under pure torsion. Each specimen was made using a cast-in-place concrete with different amounts of longitudinal reinforcements. To create the torsional moment, an eccentric load was applied at the end of the beam whereas the other end was fixed against twist, vertical, and transverse displacement. The experimental results were also compared with the results obtained from the nonlinear finite element analysis performed in ANSYS. The outcomes showed a good agreement between experimental and numerical investigation, indicating the capability of numerical analysis in predicting the torsional capacity of RC beams. Both experimental and numerical results showed a considerable torsional post-cracking resistance in high twist angle in test specimen. This post-cracking resistance is neglected in torsional design of RC members. This strength could be considered in the design of RC members subjected to torsion forces, leading to a more economical and precise design.

Keywords: cracking torque; post-cracking torsional resistance; RC beams; longitudinal rebars; FE analysis

1. Introduction

Compared to other types of forces, investigating the behavior of structural members subjected to torsion can be a more difficult task both analytically and experimentally. On the other hand the application of pure torsion, or shear and torsion simultaneously, involves practical difficulties in an experimental set-up which could compromise the accuracy of the results. However, due to the brittle behavior of RC members in torsion, studying the important factors on the torsional behavior is of high priority.

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Early attempts to study torsion as an elasticity problem dates back to the middle of previous century (Ameli 2005). Many experimental, analytical and numerical studies (Arockiasamy 1964 Bishara and Peir 1968 Cevik *et al*. 2012 Chakraborty 1979 Chiu *et al*. 2007 Elfren *et al*. 1974 Ernst 1957 Klus 1968 Martin 1973 Rahal 2000 Rahal and Collins 1995 Rahal and Collins 2003 Victor and Muthukrishnan 1973 Yoon *et al*. 2012) have been conducted to investigate the torsional behavior of RC members. Researchers have also investigated the torsional strength of concrete beams reinforced only with longitudinal reinforcement. Ersoy and Ferguson (1967) tested 25 small-scale reinforced concrete L beams without stirrups to evaluate the effects of different parameters including longitudinal reinforcement ratio, flange width and load eccentricity in the torsional resistance. They concluded that longitudinal steel ratio has a considerable effect on the torsional strength. This effect decreases with increasing eccentricity. Also, based on their results the load eccentricity has a significant influence in the torsional behavior of the test specimens. In another experimental study, Aryal (2005) found that increasing the longitudinal tensile reinforcement would not change the torque values. However, with the increment of longitudinal compressive reinforcements, an increase in torque values at different points was observed. In addition, his results described the effect of longitudinal tensile and compressive reinforcements in improving the torsional rigidity at cracking and ultimate points depending on the amount of the torque to bending moment ratio. His tests also indicated a moderate increase in the torsional ductility factor with increasing the longitudinal compressive reinforcements. Victor and Ferguson (1968) suggested an interaction relationship of torsion and moment in RC beams without stirrups in the absence of flexural shear. Mizra and McCutcheon (1968) also investigated the bending-torsion interaction behavior of concrete beam reinforced only with longitudinal reinforcement. Their experimental tests under pure torsion indicated that increasing the longitudinal reinforcements would improve the initial cracking and ultimate strength in interaction curve by approximately 15% and the twist angle was almost doubled. They also reported a linear brittle failure for unreinforced beams up to failure point in which the ultimate normal stress of concrete corresponded to the concrete rupture modulus. Based on their test results, the behavior of reinforced beam before initial cracking was identical to the unreinforced beam. After cracking, however, the reinforced specimens showed a somewhat ductile behavior with a reduction in torsional rigidity. Using past experimental tests, Hsu (1968) presented a non-dimensional interaction surface for combined shear, torsion, and bending in beams without stirrups. They have also suggested a simple and conventional design criterion for beams reinforced with longitudinal reinforcement only under combined actions.

In an experimental and analytical study, Ramakrishnan and Vijayarangan (1963) developed a relation between torque and twist of rectangular RC beams with and without transverse reinforcements under pure torsion. The ultimate strength of RC beams containing only longitudinal reinforcement under combined bending and torsion was also evaluated by Gesund and Boston (1964) using a series of experimental testing. They also developed a theoretical model to verify their test results. Based on their experimental outcomes, they reported the predominant effect of dowel action of longitudinal reinforcement in resisting the torsion in RC beams without transverse reinforcement.

Nonlinear finite element (FE) analysis has also being implemented extensively to scrutinize the behavior of reinforced concrete beams under torsion. May and Al-Shaarbaf (1989) proposed a 20-node iso-parametric brick element as the most suitable element for three dimensional (3D) nonlinear finite element analysis of a beam subjected to pure and warping torsion. In another numerical investigation, Mahmood (2007) has performed a nonlinear FE study to investigate the effects of different parameters including the beam's length and torsional reinforcements in torsional strength, and behavior of beam before and after cracking. Recently, Mostofinejad and Talaeitaba (2011) developed a numerical model using the concept of smeared cracking model in ANSYS (2005) to investigate the behavior of RC beams under torsion. Comparison of their FE results with the experimental outcomes, endorsed the accuracy of numerical analysis in predicting the cracking pattern and fracture torque. The smeared cracking model has also been used by Lisantono (2013) for nonlinear FE analysis of RC hybrid deep T beam with opening. The obtained torque-twist angle curve from the nonlinear FE analysis could accurately estimate experimental linear behavior of deep T beam before cracking. After cracking, however, the nonlinear curve obtained from the FE analysis was stiffer than the experimental results.

Despite a vast amount of research studies on the torsional behavior of reinforced concrete members, a few have focused on the effect of longitudinal reinforcements in post-cracking torsional strength of concrete RC beams without transverse reinforcements and under pure torsion. Although some earlier structural codes (e.g., ACI 318-89 1989) have considered the torsional resistance provided by concrete in torsion design of RC member, the recent structural concrete codes neglect it. According to ACI 318-89, the post-cracking strength of a RC member subjected to torsion, T_c , can be defined as

$$T_c = 0.067\sqrt{f_c'}\,x^2 y \tag{1}$$

where f_c' is the concrete compressive strength. In addition, the parameters x and y are the small and large dimension of a rectangular section, respectively. However, based on the provisions of ACI 318-99 (1999) and subsequent versions, the torsional strength of RC members, without adequate stirrups, shall be neglected in the conventional design calculation after cracking. This attests to the fact that the contributions of some phenomena, such as the aggregate interlocking in concrete and dowel action of longitudinal reinforcements, are currently disregarded in torsion calculations. However, compared to the plain concrete section, RC members only strengthened with longitudinal reinforcements can provide a considerable strength after cracking under pure torsion up to relatively high twist angles. Called inherent concrete torsional resistance, T_c, this strength could assist in the economical design of RC members leading to a more intact behavior under pure or heavy torsion forces.

In an experimental and numerical attempt, the current study was aimed at investigating the effect of longitudinal reinforcement into the post cracking torsional resistance of RC members. To pursue this objective, the results of an experimental and numerical investigation into the RC beams reinforced by longitudinal rebars, without transverse stirrups, were compared in terms of torsional cracking, twist angle, and the resistance after cracking. In the following, at first a brief overview of the theoretical models suggested for predicting the cracking torque in the past is presented. Then, the outcomes of these models are compared with that extracted from the experimental tests.

2. Theoretical models for cracking torsional resistance

Cracking torsional capacity of a concrete member is a critical field in structural design. In order to compare the cracking torque obtained from experimental tests with those predicted in theoretical models, some existing models suggested for the cracking torque are shortly described in the following.

2.1 Elastic theory

Developed initially by Bach (1912), and then followed by other researchers (Andersen 1937, Young 1923), this theory predicts the torsional failure occurs when maximum principle tensile stress reaches the concrete tensile strength, f_t. As a result, the elastic cracking torsional moment can be considered as

$$T_{cr,e} = \alpha x^2 y f_t \tag{2}$$

where α is the Saint–Venant factor based on the dimension ratio.

2.2 Plastic theory

The development of this theory was based on the fact that the elastic theory underestimates the cracking torsional capacity of concrete members. According to Nyland (1945), this additional capacity is related to the plastic characteristics of concrete. In this theory, failure criterion is similar to the elastic theory and states as

$$T_{cr,p} = \alpha_p x^2 y f_t \tag{3}$$

In the above equation, the concrete plastic characteristics are introduced using the parameter α_p which is given by

$$\alpha_p = 0.5 - x/(6y) \tag{4}$$

It should be mentioned that on the basis of the experimental results, this theory could not accurately predict the cracking torque of specimens with small dimensions (Hsu 1968).

2.3 Skew bending theory

According to this theory, the torsional moment at surface failure consists of bending and torsional moment components. The bending component might result in the flexural rupture in the failure surface. Hence, a tensile and compressive stress might be sustained by an element decreasing the concrete tensile strength by 15%. In this theory, the cracking torsional strength of a concrete member could be determined using Eq. (5) which is similar to the relation proposed by Hsu (1968)

$$T_{cr,sp} = 0.5\sqrt{f_c'}\,\frac{x^2 y}{3} \tag{5}$$

3. Experimental investigation

In order to evaluate the effect of longitudinal reinforcements on the torsional behavior of RC beams, six small-scale specimens were designed, built, and tested. As the control specimen, the first beam was built without any reinforcement, while different amounts of longitudinal steel rebars were used to reinforce the others. Following the main purpose of the study, all specimens were reinforced only with longitudinal rebars without transverse reinforcement. The rebars were placed in their positions using two wooden plates which also acted as a mould at both ends. The specimens were allowed to cure for 28 days before testing.

3.1 Dimensions of the specimens

The dimensions of all specimens were identical and a 20 mm concrete cover was considered for rebars in all specimens. Fig. 1 shows the schematic illustration along with the geometry details of the test specimens. Due to the stress concentration, the end parts of each specimen were considered wider than the rest. The length of the middle part was taken equal to the section perimeter. This particular design would lead to the torsional failure of the middle part where the applied torque results in a uniform shear stress. The reinforcement details of all specimens are given in Table 1. All the test specimens except the last one were designed based on increasing the longitudinal reinforcement ratio as the main test parameter. The sixth specimen was reinforced with different steel rebars at the top and bottom of the beam to investigate the effect of different longitudinal reinforcement ratios.

3.2 Material properties

Concrete used in the experimental procedure was normal weight concrete with a water/cement ratio of 0.5, type I Portland cement content of 400 kg/m^3, coarse aggregate, and 0-5 mm river aggregates either at weight mix ratio of 900 kg/m^3. The compressive strengths of concrete obtained

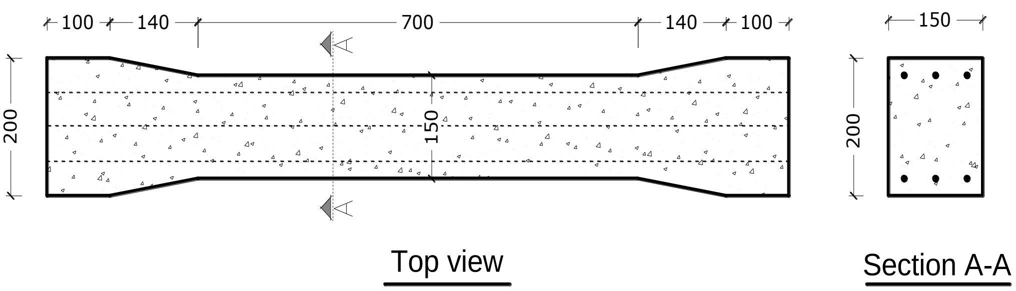

Fig. 1 Geometry and reinforcement detail of the test specimens (all dimensions are in mm)

Table 1 Reinforcement details and concrete properties of all specimens

Specimen no.	Concrete compressive strength (MPa)	Top reinforcement		Bottom reinforcement	
		Rebars	Ratio (%)	Rebars	Ratio (%)
1	22.6	0	0	0	0
2	21.7	2ϕ8	0.4	2ϕ8	0.4
3	19.8	2ϕ10	0.6	2ϕ10	0.6
4	22	2ϕ12+1ϕ8	1	2ϕ12+1ϕ8	1
5	20	3ϕ12	1.3	3ϕ12	1.3
6	23.1	3ϕ12	1.3	2ϕ12	0.6

Fig. 2 Test set-up

from standard cylinder tests after 28 days are tabulated in Table 1. The steel rebars used in the test specimens were deformed shape, categorized as Grade 60 (f_y = 414 MPa) according to the ASTM A615 (2009).

3.3 Test set-up

As observed in Fig. 2, experimental tests were performed using a torsion testing machine, especially designed and made at the laboratory. In this set-up, the test specimen was fixed in one end while an eccentric load was applied at the other end to simulate torsional moment. The test set-up was designed to apply a pure constant torsion to the specimens. The twist along the specimen was measured from two dial gauges installed at both ends and at mid-point.

4. Experimental results and discussion

The torque-twist curves obtained from the testing of all specimens are plotted in Fig. 3. The behavior of all specimens is linear up to the cracking point. Due to the lack of steel reinforcements in the control specimen, a brittle failure without preserving any torsional capacity was observed at the onset of cracking in the specimen. This type of failure for unreinforced specimen was also confirmed by other researchers (Mizra and McCutcheon 1968). For the reinforced specimens, however a ductile behavior was observed after cracking in which they sustained large twist values without a significant lost of strength. This is particularly validated for the specimens with higher reinforcement ratio (e.g., specimens 4-6). Following a linear behavior, the torque-twist curves of the reinforced specimens were similar to the control specimen before cracking. The values of the cracking torque were approximately identical for all specimens. In general, the performance of the reinforced specimens can be estimated by a bilinear curve with a change in the trend after cracking. The experimental outcomes confirmed the negligible effect of longitudinal reinforcement in torsional rigidity of the specimens before cracking. However, the amount of the post-cracking resistance and torsional rigidity rises consistently with increasing the amount of longitudinal steel reinforcements, as observed in Fig. 3. Compared to the specimens 4 and 5, the torque-twist curve of the specimen 6 was experienced a higher rate of strength loss. This might be due to the lower ratio of the longitudinal bottom reinforcements in the specimen 6 and in turn can point to the

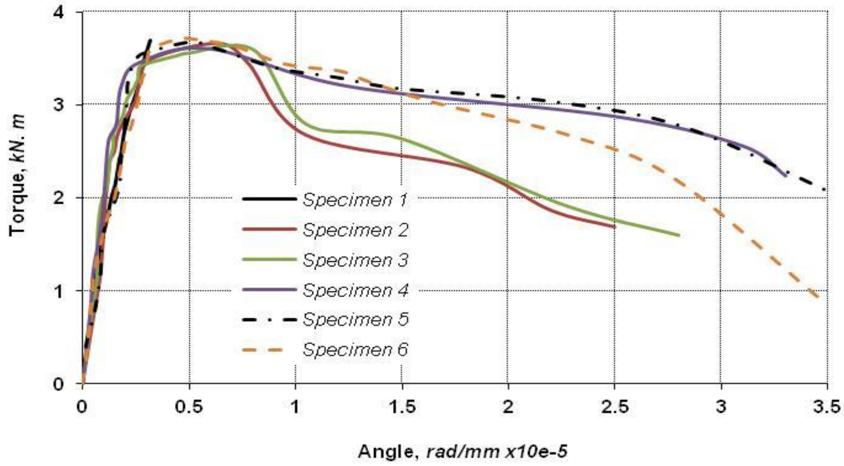

Fig. 3 Torque vs. twist curves of all test specimens

Fig. 4 Cracking pattern of the forth specimen

significance of bottom reinforcements in preserving the post-cracking resistance of the beams without stirrups.

The failure mode of the control specimen was like a brittle material. It failed suddenly after cracking. However, a ductile behavior was observed for the reinforced specimens after cracking. For the sake of illustration, the cracking pattern of the specimen four after testing is indicated in Fig. 4. As outlined earlier, many theories were developed to predict the cracking torsional resistance of concrete members. In this study, three theoretical models were selected and their estimated values of the cracking torque of the test specimens were compared with those measured in the experimental tests in Table 2. In the current study, the cracking strength of the test specimens was measured at the onset of cracking observed during the experimental test. Compared

Table 2 Experimental and theoretical values of cracking torsional resistance

Specimen no.	1	2	3	4	5	6
Compressive strength, MPa	22.6	21.7	19.8	22	20	23.1
Experimental cracking strength, N.m	3690	3500	3455	3490	3560	3660
Elastic theory, N.m	2845	2788	2663	2807	2676	2876
Plastic theory, N.m	4813	4716	4505	4749	4528	4866
Skew bending cracking strength, N.m	3565	3494	3337	3518	3354	3605
Experimental to skew bending strength ratio	1.03	1.00	1.03	0.99	1.06	1.01

Table 3 Twist ductility factor and post-cracking strength of all specimens

Specimen no.	Experimental cracking strength (N.m)	Post cracking strength (N.m)	Experimental cracking rotation $(\mathrm{rad/mm})\times10^{-7}$	Experimental ultimate rotation $(\mathrm{rad/mm})\times10^{-7}$	Ductility factor
2	3500	2057	26	95	3.65
3	3455	2098	26	120	4.62
4	3490	3009	25	245	9.8
5	3560	3084	26.7	278	10.41
6	3660	2507	27	149	5.52

to the other models, the skew bending theory estimated the cracking torsional capacity of the test specimens with reliable accuracy. The average ratio of the values predicted by skew bending theory to that determined in the experimental tests was calculated to be about 1.02 which refers to 2% difference. While the elastic theory underestimated the cracking torsional strength of the test specimen, the values predicted by plastic theory were higher than those measured in experimental tests. The incapacity of the elastic and plastic theories in predicting the small scale test specimens were also reported by other researchers (Hsu 1968).

To elaborate the effect longitudinal reinforcement percentage on rotational capacity of the beams without stirrups after cracking, the twist capacities of the test specimens were compared employing the twist ductility factor (herein referred to as ductility factor). This parameter was defined as the ratio of the ultimate twist to the crack twist angle. Following the concept of ductility in the flexural members, the ultimate twist was assumed at the attainment of 20% drop in the maximum torsional resistance (Park 1989). The crack twist corresponds to the cracking torque of the specimens measured in the experimental tests (See Table 2). The calculated values for all specimens are summarized in Table 3. Due to the brittle failure of the control specimen, this specimen was eliminated in these calculations. It is evident that the ductility factor of the control specimen is one based on the above definition. Fig. 5 also plotted the distribution of the ductility factor against the longitudinal reinforcement ratio. As observed in Table 3 and also plotted in Fig. 5, the twist capacity of the specimens was improved with increasing the amount of longitudinal reinforcements. However, a significant drop was observed in the ductility factor and ultimate rotation capacity of the specimen six compared to the specimens four and five. This can be attributed to the lower amount of bottom reinforcements in this specimen compared to the specimens four and five.

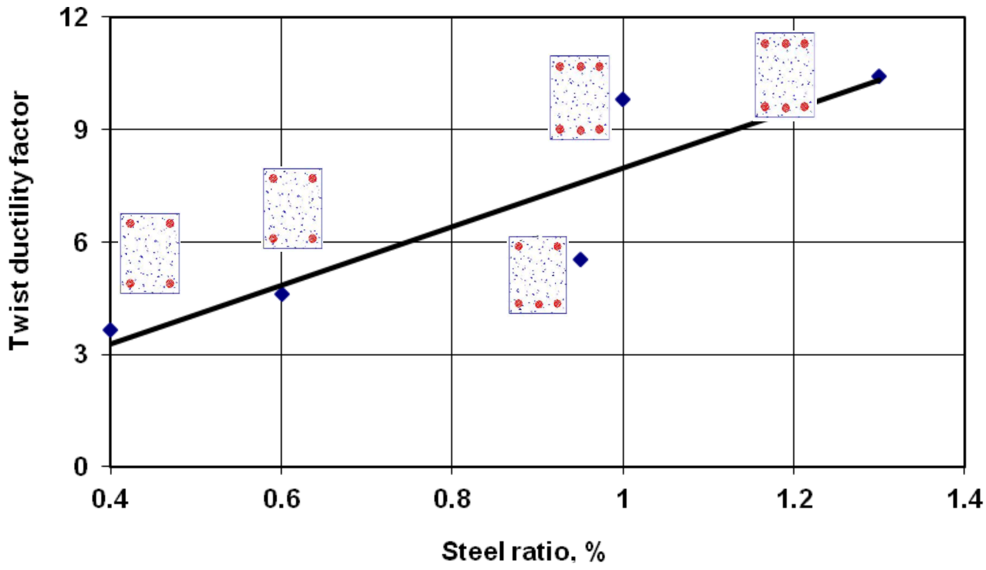

Fig. 5 Variation of the twist ductility factor with the steel ratio

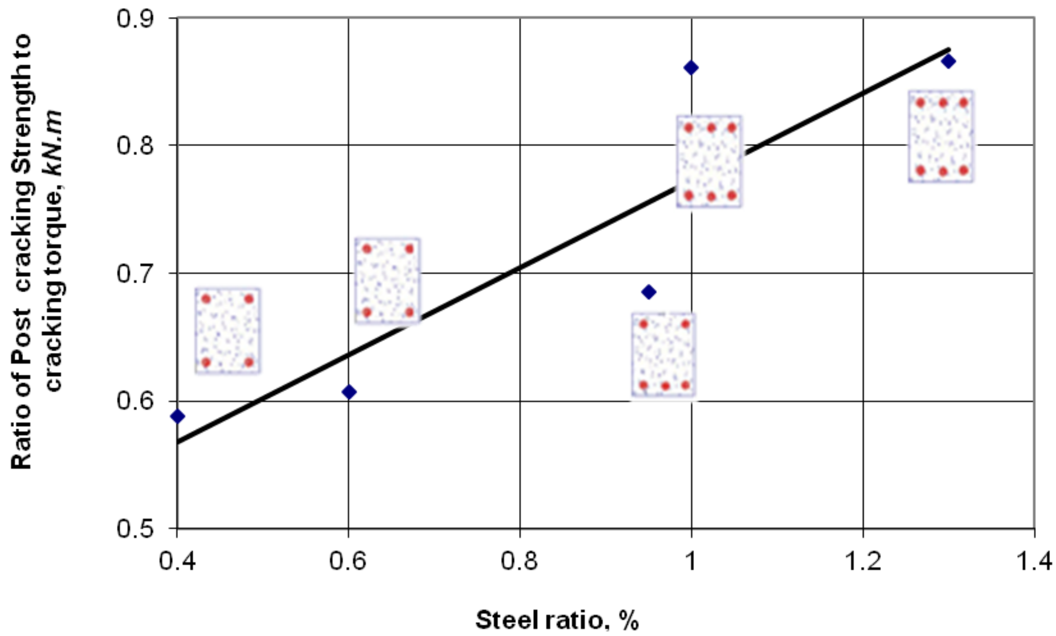

Fig. 6 Ratio of the post cracking to cracking tensional capacities vs. steel ratio

The experimental results showed a considerable torsional resistance after cracking in reinforced specimens. In order to quantify the post-cracking torsional strength, their values could be compared at a same twist angle after cracking. Herein, the torque values measured at a twist angle equal to eight times of the cracking twist was considered as the post-cracking torsional resistance and their values were given in Table 3. Although negligible, however, to eliminate the effects of concrete compressive strength, the obtained strength at the target twist angle was normalized by the cracking torsional strength and considered as a non-dimensional parameter. As shown in Fig. 6, a significant resistance could be observed after cracking in all specimens. In addition, this post cracking resistance follows almost an upward trend with increasing the steel reinforcements in all reinforced specimen. For the specimen six, however, this increment was remarkably smaller than the specimens four and five because of lower bottom reinforcement ratio. From the experimental outcome, it can be concluded that longitudinal reinforcements have a considerable influence in post-cracking torsional behavior of RC beams without stirrups.

5. Nonlinear FE modeling of the specimens

In the second part of this study, in order to assess the capability of numerical analysis in estimating the cracking and post-cracking torsional resistance of RC members, the nonlinear FE analyses of all specimens were performed using ANSYS (2005). The boundary conditions and applied loads in the numerical modeling were aimed to simulate the experimental conditions. The concrete material was defined based on the concept of the smeared cracking model, as suggested and employed by other researchers (Ameli *et al*. 2007, Dalalbashi *et al*. 2013, Dalalbashi *et al*. 2012, Mostofinejad and Talaeitaba 2011). Also, a perfect bond was considered between concrete and steel during the FE analysis.

The compressive strength of concrete was taken as measured for each specimen (see Table 1). In addition, the yield steel strength of steel reinforcement was taken as 414MPa. In this study, the stress- strain behavior of concrete was simulated using the Hognestad model (Hognestad *et al*. 1955). In this model, the strain under uni-axial stress conditions corresponding to the concrete compressive strength was taken as 0.002, as recommended for normal concrete by Park and Paulay (1975). The ultimate concrete strain was assumed to be 0.0038. The simplified bilinear model with strain hardening was also used to simulate the behavior of longitudinal steels.

All steel rebars were modeled using the LINK8 truss element, while the SOLID65 element was employed to model concrete. With the capability of modeling both cracking in tension, and crushing in compression, the latter has been especially designed for modeling concrete in ANSYS. To provide a more even stress distribution at support and load point, two steel plates were modeled using an eight-node 3D solid element called SOLID45. A typical FE model of the specimens is illustrated in Fig. 7.

In ANSYS, the five-parameter model developed by William-Wranke (1975) has been suggested as a concrete failure criterion (ANSYS Manual 2005). Both cracking and crushing are considered in the adopted model. The 3D failure surface of concrete as calculated based on this model, is illustrated in Fig. 8. The parameters σ_{xp}, σ_{yp}, and σ_{zp} refer to the principle stresses in x, y, and z direction, respectively. The concrete failure mode is a function of the sign of σ_{zp}. For example, if σ_{xp} and σ_{yp} are both negative and σ_{zp} is slightly positive, cracking would be predicted in a direction perpendicular to the σ_{zp} direction. However, if σ_{zp} is zero or slightly negative, the concrete is assumed to crush. In addition, this model uses the concept of a smeared crack model first introduced by Rashid (1968). Because, in reality concrete cracking is comprised of a system of parallel cracks continuously distributed over the concrete mass, this model represents the cracks by parallel micro-cracks distributed (smeared) over the finite elements. In this model, the cracks are considered to be an indication of changing in the material property of the element, over which

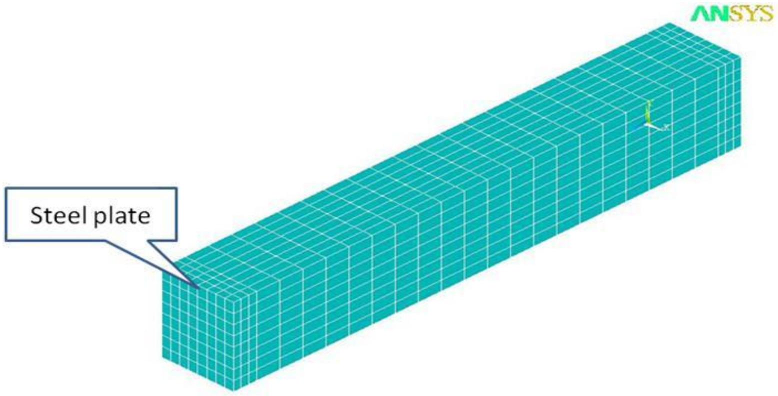

Fig. 7 A typical FE model of the test specimens

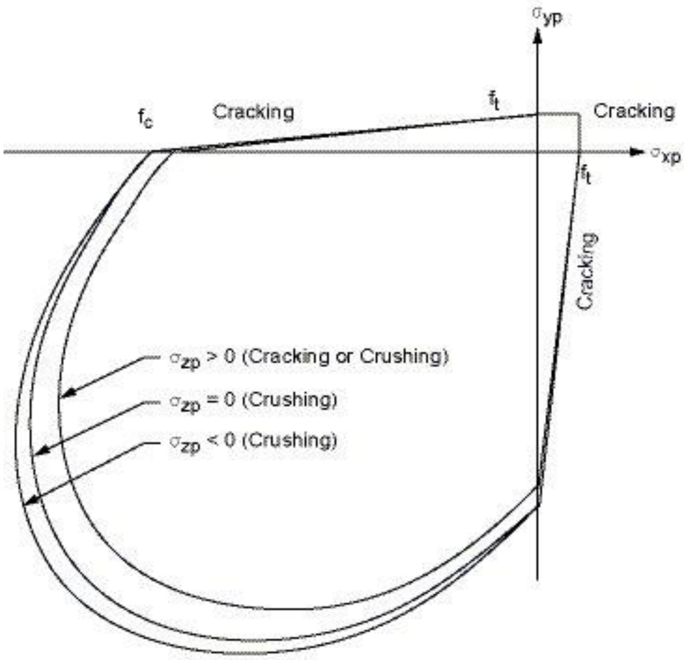

Fig. 8 Three dimensional failure surface calculated based on the William-Wranke model (Willam and Warnke 1975)

the cracks are assumed to be smeared. Being computationally very convenient, and because the crack can develop in any direction, this model suits the FE analysis of concrete elements (Karayannis 2000).

Amongst the important parameters which control the failure of concrete in ANSYS, are elastic modulus (E_c), concrete compressive strength $\left(f_c'\right)$, concrete tensile strength or modulus of rupture (f_t), poisson's ratio (v). In addition, two shear transfer coefficients, β_t and β_c, were introduced for open cracks and closed cracks, respectively. Both coefficients possess values between 0 and 1, and could significantly affect the results of a nonlinear analysis of RC members. The value used for β_t in the past studies, however, varied between 0.05 and 0.3 (Kachlakev *et al*. 2001, Mostofinejad and Talaeitaba 2011). In this study, based on the results of other numerical investigations (Dalalbashi *et al*. 2013, Dalalbashi *et al*. 2012, Eslami *et al*. 2013, Mostofinejad and Talaeitaba 2011), a value of 0.3 and 0.99 were selected for open and closed crack coefficients, respectively.

Numerical analyses of the tested specimens were carried out according to the aforementioned assumptions which have also been adopted by other researchers (Ameli *et al*. 2007, Mostofinejad and Talaeitaba 2011) in nonlinear FE modeling of RC members under torsion. During the analysis, the load was applied step-by-step using the modified Newton-Raphson method to arrive at the solution. Figs. 9-14 compare the torque-twist curves extracted from the experiment tests with those obtained in nonlinear analysis. Before cracking the torque-twist curves predicted by nonlinear FE analyses were almost linear and adequately close to the experimental results. After cracking, the FE behavior was stiffer than the obtained experimental curves. This can be due to the differences between the smeared cracking assumption adopted in nonlinear simulation and the actual behavior of concrete after cracking in which some large cracks are developed after linear behavior. Despite some discrepancy between experimental and numerical result, a good agreement was observed between both curves in each specimen, particularly in estimating the cracking strength. The predicted post-cracking behavior using nonlinear FE analyses can also approximate the experimental response of the test specimens up to relatively high twist angles with a reliable accuracy.

Fig. 15 plots the variation of the normalized ratio of the post-cracking to cracking torsional resistance against the longitudinal reinforcement ratio. As mentioned before, the post-cracking resistance of the specimens was considered as the torsional resistance corresponded to the twist angle equal to eight times of cracking twist angle. Similar to the experimental results, the nonlinear results also indicated a considerable post-cracking torsional strength in all reinforced specimen. Considering the inherent difficulties related to the experimental investigation of the structural members under torsion forces, numerical analysis can also be utilized as a reliable and convenient option. Moreover, due to the restriction in the number of experimental specimens, the preliminary nonlinear analysis is usually necessary to design optimal specimens, before performing the experimental program.

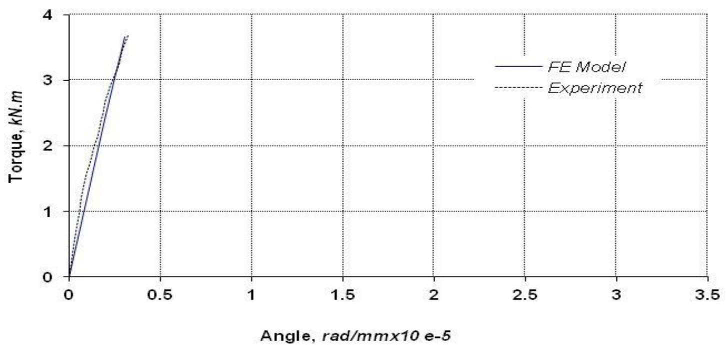

Fig. 9 Comparison of torque vs. twist curve obtained from experimental tests and nonlinear FE analysis for specimens no. 1

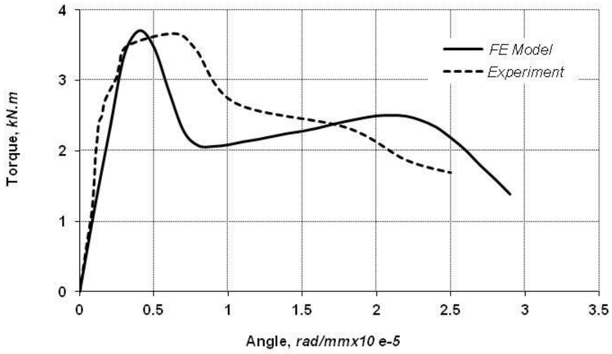

Fig. 10 Comparison of torque vs. twist curve obtained from experimental tests and nonlinear FE analysis for specimens no. 2

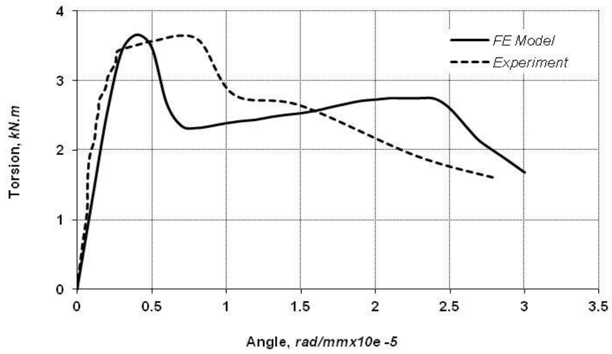

Fig. 11 Comparison of torque vs. twist curve obtained from experimental tests and nonlinear FE analysis for specimens no. 3

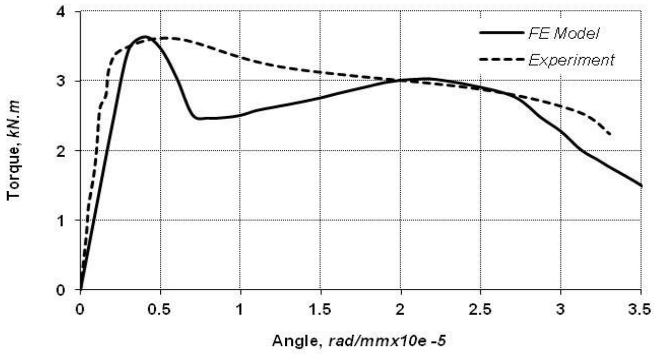

Fig. 12 Comparison of torque vs. twist curve obtained from experimental tests and nonlinear FE analysis for specimens no. 4

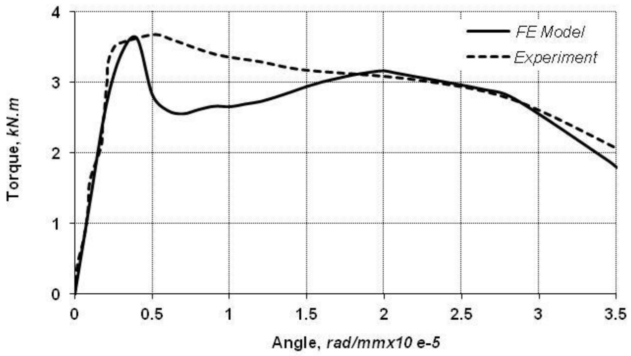

Fig. 13 Comparison of torque vs. twist curve obtained from experimental tests and nonlinear FE analysis for specimens no. 5

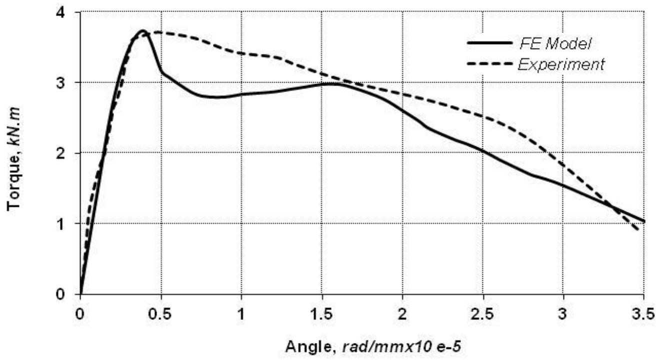

Fig. 14 Comparison of torque vs. twist curve obtained from experimental tests and nonlinear FE analysis for specimens no. 6

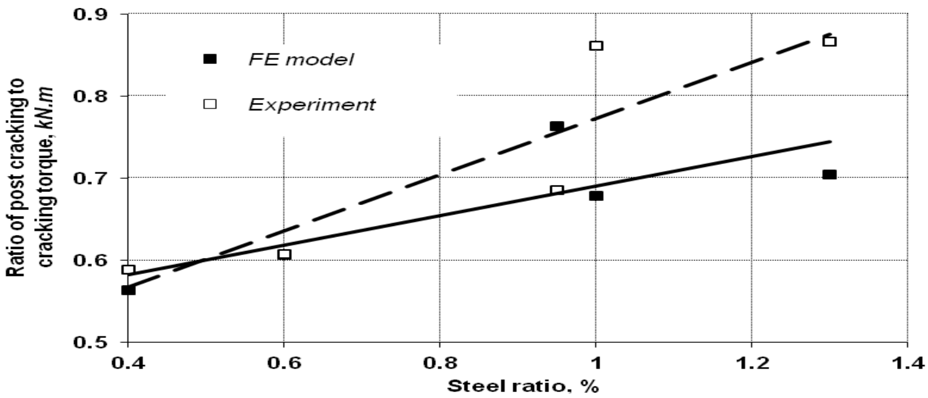

Fig. 15 Ratio of the post cracking to cracking tensional capacities vs. steel ratio obtained from both experimental and numerical analyses

Table 4 Comparison of the experimental results with ACI318-89 predictions

Specimen no.	Post cracking strength (N.m)	Strength ACI318-89 (N.m)
2	2057	1404
3	2098	1341
4	3009	1414
5	3084	1348
6	2507	1449

6. Comparison with ACI provisions

As outlined earlier, ACI 318-89 (1989) had considered concrete inherent strength, T_c, in design of RC members under torsion. This strength which does not account for the longitudinal and shear reinforcements was given in Eq. (1). In order to evaluate the results of the experimental tests, a comparison between post-cracking torsional strength of the test specimens with those predicted in ACI 318-89 are provided in Table 4. It can be observed that the post-cracking torsional strength of the specimens, defined as the torque value corresponding to the twist angle equal to eight times of the cracking twist, is much higher than the values suggested according to ACI provisions. This difference can be attributed to the fact that the values predicted by ACI 318-89 were based on the 40% of the values suggested in skew bending theory and neglect the effect of longitudinal reinforcements. It should be mentioned that as indicated in Table 2, the cracking torque of the experimental tests were almost similar to the values determined according to the skew bending theory. In addition, Eq. (1) was defined to be conservative considering the probable bending moment in members under torsion. However, the current specimens were tested under pure torsion.

7. Conclusions

This paper deals with an experimental and numerical investigation into the effect of longitudinal reinforcements on torsional behaviour of RC members. Of particular interest was the post-cracking behaviour of the test specimens. Based on the experimental and numerical result, the following conclusions have been drawn:

• While the recent structural design codes neglect the torsional capacity of concrete after cracking, RC members reinforced only with longitudinal reinforcements can preserve a significant torsional resistance at large twist values. This is the most important outcome of the current study, which could be considered in order to create a more economical and elaborate design of the RC structures under torsion forces.

• Comparison of the cracking torque of the specimens obtained from experiments, with those measured from the theoretical models, showed that the skew bending theory provides a more accurate response compared to the other hypotheses.

• The nonlinear FE analyses of the test specimens reconfirmed not only the capability, but also the accuracy of this type of analysis in the evaluation of torsion problems. Due to the inherent difficulties of the experimental investigation of RC members under torsion, many researchers prefer to conduct numerical studies.

The aforementioned conclusions have been validated within the range and dimension of the test specimens considered in this study. More experimental study with a greater number of specimens should be conducted, to investigate the effect of different parameters and to develop a practical design approach, which takes into consideration the post cracking torsional capacity of RC members.

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