

How Construction Errors Affecting the Bearing Capacity of the Concrete Beams? Inelastic Deflection of Concrete I-Beams

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Abstract— Reinforced concrete I-beams are widely used in bridge construction. The span length to beam height ratio in these decks is sometimes between 15 and 30. Due to the widespread utilization of these bridges and their heavy traffic loads, special attention must be directed at recognizing the behaviors that lead to construction errors associated with these structures. Because of various construction and environmental factors, construction faults might result in concrete quality or during concrete placement which can consequently lead to inhomogeneities in beam sections. This means that concrete density and compressive strength in the beam cross section might deteriorate along the section height.

Extensive research has been conducted on the non-linear and inhomogeneous behavior of concrete beams. However, few researchers have specifically addressed density and compressive strength variations along beam cross sections. The present research is aimed at estimating from elastic material parameters, the concrete I-beam inelastic deflection resulting from inhomogeneous behavior along the beam cross sectional height. The behavior of the beam under study was checked by comparing its relevant parameters with the results obtained from the OpenSees Software and the method proposed in the Concrete Code ACI318. Excellent agreement was observed in both cases. Moreover, a value of unity was proposed for parameter “n” in the relation set forth by Branson in the Concrete Code ACI318 for estimating section cracking moment in long span I-beams.

Index Terms—Inelastic Deflection, Concrete Beam, Inhomogeneity, Cracked section, Construction errors.

I. INTRODUCTION

The RC flexural elements such as one-way beams and slabs bend under applied loads. Such deformation is called “deflection,” which depends on loading and stiffness of a flexural element. Therefore, under various load levels, the measure of such deformation is different. In RC structures, the deformations in flexural elements are deemed significant, therefore, necessitating the study of this phenomenon. Since a deformation in flexural elements does not bring about structure failure, many do not feel the necessity observe safety confidence factor and also load factors. Nevertheless, logically, studying the deflection of a structure under service loads and controlling deflections in critical cases are essential.

The simple equations for the strength of material can be used to calculate the instantaneous deflection in a steel flexural

element. However, calculating the deformation of one beam of RC encounters the problem of determining stiffness of its section. In fact, in a beam with a steel section, the elasticity module is stable during loading and reaching the yielding limit. Furthermore, under bending, the steel section will present its ultimate strength both in compression and tension, and therefore, its moment of inertia and consequently its bending stiffness will be fixed. However, in a beam with a concrete section both the elasticity module and the moment of inertia will vary during loading due to cracking. In such a case, it is important that the change in the moment of inertia be proportional to the change in the amount of load along the beam. There are some factors which cause the mechanical factors of concrete in one dimension are not uniform and isotropic in the other one. These factors effect on the concrete elasticity modulus, Poisson coefficient and regular relations of deflection of concrete beams.

The effective moment of inertia expression proposed by Branson (1965) was developed empirically based on the test results of simply-supported rectangular reinforced concrete beams with reinforcement ratios between 1% and 2%. Branson's expression accurately estimates the moments of inertia of concrete beams with medium to high reinforcement ratios ($\rho > 1\%$). Nonetheless, different studies [Scanlon et al. (2001), Gilbert (1999), Gilbert (2006)] indicated that the expression constantly overestimates the moments of inertia of reinforced concrete beams with low reinforcement ratios ($\rho < 1\%$), which causes underestimation of the deflections.

The purpose of this research is to investigate further the effects of non-uniformities in concrete density and compressive strength on cracking and cross sectional EI of reinforced concrete I-beams in bridge decks.

II. LITERATURE REVIEW

Several research studied have been devoted to propose the variations in concrete member stiffness caused by cracking. In 1960 Yu and Winter has suggested modification factors for the flexural rigidity while in 1963 Branson identified an effective moment of inertia for cracked concrete. Other procedures for concrete non-linear effect are due to Beeby in 1968, making adjustments to the curvature along the span and at critical sections and due to Ghali, et al. Kripanarayanan and Branson (1976) proposed an extension of the Nilson and Walters's equivalent frame procedure to include cracking using the so-called effective moment of inertia. The Vecchio and Collins (1986) have applied the finite-element analyses incorporating material non-linearity to study the lateral response of reinforced concrete structures. In 1989, making use of a section-curvature incremental evaluation.

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In 1991 Stafford, Smith and Coull treatise has an extensive treatment of the cracked concrete stiffnesses in tall buildings. Joseph E. Wickline (2002) studied on effective moment of inertia models for full-scale reinforced concrete T-beams subjected to a tandem-axle load configuration, [1]. Chang1, et. al. (2004) studied on a simplified method for nonlinear cyclic analysis of reinforced concrete structures based on direct and energy based formulations, [2]. A trilinear bending moment-curvature model is used by Ahmed and Perry (2004) to develop a procedure for determining the effective flexural stiffness of concrete walls in linear seismic analysis. Tikka and Mirza in 2005 have developed EI equation including the non-linear concrete stress-strain curve for slender RCC Columns. Bischoff (2005) found out that the misestimation of the moments of inertia and deflections of lightly-reinforced concrete beams by the Branson's approach is caused by the overestimation of the tension stiffening of concrete. According to the analytical study carried out by Bischoff, the tension-stiffening component in Branson's method depends on the applied load level (M_a/M_{cr}) and on the ratio of the gross moment of inertia to the cracked moment of inertia (I_g/I_{cr}) of the beam, which varies inversely with the reinforcement ratio (ρ). Branson's expression provides accurate estimates for reinforced concrete beams with reinforcement ratios greater than 1%, which corresponds to an I_g/I_{cr} ratio of 3. For lower reinforcement ratios ($I_g/I_{cr} > 3$), the member response estimated by Branson's approach is stiffer than the actual response, resulting in the underprediction of the deflections.

Abd El Aziz and Mohamad Fathy (2006) investigated on non-linear analysis of concrete beams prestressed and post-tensioned with carbon fiber reinforced polymer (CFRP) bars, [3]. Gilbert (2006) and Bischoff and Scanlon (2007) compared the experimental results of the beams and slabs with different reinforcement ratios to the analytical deflection estimates obtained from the two approaches [Branson (1965) and Bischoff (2005)]. The advances in non-linear concrete stiffness studies to optimize the structural design of tall buildings are due to Chun and Wang (2006). Cengiz Dundar and İlker Fatih Kara (2007) researches were on three dimensional analysis of reinforced concrete frames with cracked beam and column elements, [4]. Bischoff, P. H. and Scanlon, A. (2007) studied on effective moment of inertia for calculating deflections of concrete members containing steel reinforcement and fiber-reinforced polymer reinforcement, [5]. Studies on static and dynamic modulus of elasticity of concrete were proposed on the ACI-CRC Final Report (2008), [6]. Also M. Ahmed et. al. (2008) [7], İlker Kalkan (2010) [8], Bischoff, P. H., and Gross, S. P. (2011) [9], Kulkarni S.K. et. al. (2012) [10], S. Roohollah Mousavi and M. Reza Esfahani (2012) [11], Jing Bo An (2012) [12], Amadio, Claudio et. al. (2012) [13], Lin, X. and Zhang, Y. (2013) [14], Shahida Manzoor, Shuaib Ahmad (2013) [15], Naser Kabashi et al., (2013) [16], Alameer Ali (2013) [17], S. B. Kandekar et. al. (2013) [18], Md Shahnewaz (2013) [19], Patel, Vipulkumar Ishvarbhai (2013) [20], Amini Najafian H et. al. (2013) [21], Bukhari IA et al. (2013) [22] and Stochino, Flavio (2013) [23] have been sum research on concrete beams nonlinearities.

III. METHOD

A. Material Properties, Nonlinearity, Inhomogeneity

It is obvious that due to the non-homogeneous quality of

concrete, its tension resistance is not absolutely consistent in any given length. As long as the stresses in concrete are inconsiderable, $f_t < f'_t$, and the concrete will not experience any cracks. However, under service loads or when shrinkage in concrete increases considerably, the first cracking will occur as soon as f_t surpasses f'_t or f_r . From this point, the distribution of tension stress in the concrete is uneven, and the tension surrounding the crack will rise from 0 to the maximum amount of f_t , fig. 1.

There is wide variability in the values of the modulus of rupture reported in literature. The value reported by various investigators for the modulus of rupture range from 0.33 to 1.0 f_c MPa and the actual deformation values can vary between 25 to 40 percent using different expressions of modulus of rupture.

The moment-curvature slope represents the flexural stiffness (EI). Using this curvature, one can calculate the flexural stiffness (EI) of a beam per different amounts of the bending moment. As previously discussed, calculating the instantaneous deflection faces two major problems: stiffness change in the section of the beam at the time of loading because of tension stiffening, and stiffness change in the length and other various areas of the beam. Not to mention that the elasticity modulus of concrete varies depending on the amount of load. To simplify the calculation of deformations in RC beams, the concept of effective flexural stiffness (EI_{eff}) is used. If the elasticity modulus of concrete is considered constant-its amount equaling the slope which connects the origin point to the analogous tension point $0.4f_c'$ in the stress-strain-, and if the moment-deformation curve of the beam is assumed as a two-line one, the effective moment of inertia can be considered uniform along the beam. This effective moment of inertia will be a function of the moment of inertia of the uncracked section (I_{uc}) and the moment of inertia of the cracked section (I_{cr}). According to his broad experimental studies, Branson has proposed the following equation to determine the effective moment of inertia of a beam when calculating deflections, "(1)", [24].

$$I_e = \left(\frac{M_{cr}}{M_a} \right)^a I_{uc} + \left[1 - \left(\frac{M_{cr}}{M_a} \right)^a \right] I_{cr} \quad (1)$$

Where I_{uc} and I_{cr} are the moment of inertia of the uncracked section and the moment of inertia of the cracked section, respectively; M_{cr} and M_a represent the cracking moment of a section and the maximum flexural moment of the section. Also, a is a constant coefficient which is $a=4$ for sections under consistent moment and is used to include tension stiffening of concrete in the equation; furthermore, $a=3$ is proposed by Branson for simply supported beams and is used to include tension stiffening of concrete and also the variations in the stiffness along beams.



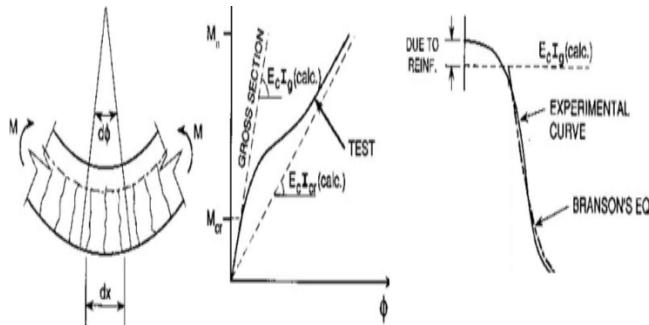


Fig. 1. variation of effective stiffness of the cracked and un-cracked section

According to ACI 318, if the calculation of displacements is desired, the displacements after applying loads can be calculated through the common methods and equations and including the effects of cracking and presents of bars on the stiffness of an element. If the stiffness of beams is not calculated by using a comprehensive analysis, to calculate the instantaneous deflection of beams, the concrete code defines the elasticity modulus of concrete as:

$$E_0 = 0.1347 W_c^{1.5} \sqrt{f_c} \left(\frac{kg}{cm^2} - ACI\ 318 \right) \quad (2)$$

In such a case, according to Branson's equation, the effective moment of inertia of the cracked section is defined as "(1)". For the purpose of this research, I have done the following steps:

1, Defining the Problem and the relevant parameters. 2, Choosing of a finite element nonlinear analyzing software, OpenSees. 3, Introduction of the problem to the software. 4, Programming in MATLAB and Introduction of the problem to the software. 5, Extracting of the m-θ curves from the opensees and use it in the matlab. 6, Obtaining of preliminary results from the softwares. 7, Analyze and compare the results with the application. 8, Analysis using ACI318 Code. 9, Comparison of three methods and discuss the resulting graphs.

B. The Problem Definition

Assume a reinforced concrete I-beam with a clear length of 20 meter, cross sectional area of A1, and longitudinal cross sectional area of A2 as shown in figure 2. The compressive strength of concrete and its density are 300 (kg/cm²) and 2400 (kg/cm³) respectively. The yield stress for longitudinal reinforcing steel bars is 4000 (kg/cm²). The two parameters "μ" and "ε" represent variations associated with non-uniformities in concrete density and strength and are assigned the values 0.9, 0.5, 0.1 and 0 in this article. The cross sectional area of the beam is 1600 cm², and tension reinforcement levels of one, two, and three percent are introduced in the cross section. The percentage of compression reinforcement used is one third that of the tension reinforcement. Meanwhile, with due regard to the beam height, 0.5% of the beam cross section was allocated to cheek (torsion) reinforcement of the beam. The modeling of the beam under study was done in the OpenSees Software environment, and the results (obtained upon careful analysis in each case) were compared with those calculated from other methods. Subsequently, a discussion of the results was conducted.

Modules of the method are developed for implementation in the Open System for Earthquake Engineering Simulation

framework (OpenSees). "OpenSees" software was obtained by the scientists of Berkeley University, and is a finite element program with unique facilities and outstanding flexibility. One can get this software in www.opensees.berkeley.edu. The beam is introduced like nonlinear element of beam-column in complete 2D, to the OpenSees software.

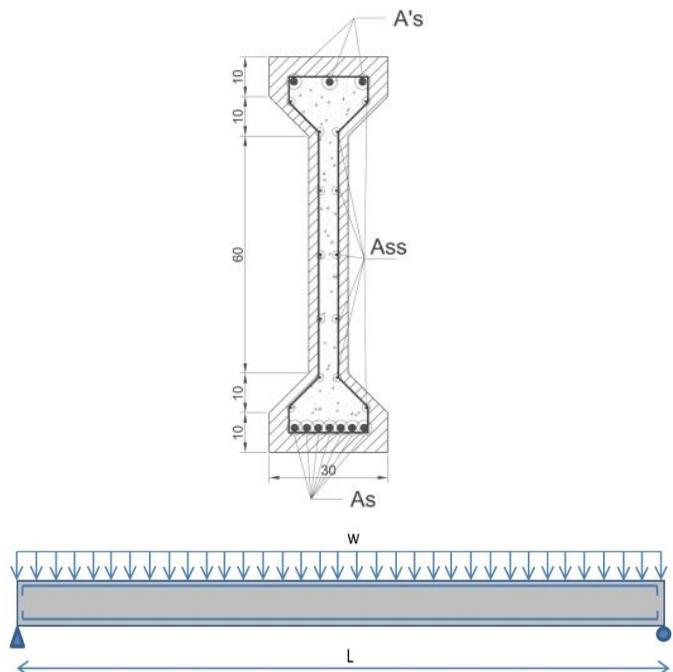


Fig. 2. Longitudinal and cross section of the desired beam

In finite element modeling of the beam, the Concrete-02 element was used to model the concrete in the beam core (in the confined area) as well as the beam lining (in the unconfined area), and the Hysteretic element was used to model the section reinforcements. The beam cross section was introduced to the software through the fiber elements and the Zero-length-section. For this section, the moment-curvature analysis was conducted and continued until the section's rupture limit. In each case, the M-θ diagram for the beam cross section was obtained for demonstrating the results. The M-θ curves were subsequently used to determine the beam cross section instantaneous stiffness.

As a subsequent step in the OpenSees finite element analysis, another analysis was performed to determine exactly the inelastic beam deflection. The Displacement elements were used to introduce the beam to the software. The beam was exposed to uniformly-distributed linear loading and analyzed through the overload failure method until total failure occurred. The results of this analysis were also compared with those obtained from other methods.

It must be pointed out here that the inhomogeneous density and inhomogeneous compressive strength (along the beam cross section) of the concrete were considered and introduced to the software as well. Therefore, a change in the values of "μ" and "ε" would bring about a change the mechanical properties and consequently the flexural capacity of the beam cross section.



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The nonlinear geometry impacts and P-Delta are considered in the related options. For considering the nonlinear effects of girder in this study, the girder is modeled by the nonlinear element of the column related to the displacement and distributed plasticity, which is one of the advantages of this study in comparison with the others. It should be considered that in most of the research activities, the girder of bridge is supposed liner and modeled with this supposition. The flexural stiffness reduction of beams and columns due to concrete cracking plays an important role in the nonlinear load-deformation response of reinforced concrete structures under service loads.

IV. SOFTWARE OUTPUTS AND RESULTS

Figures 3-a, 3-b and 3-c show the $M-\theta$ diagrams obtained for $\rho= \%$ 1, %2, %3 respectively. Each $M-\theta$ diagram was plotted for different values of “ μ ” and “ ε ”, and subsequently compared with other diagrams. The yield point moment was shown for the 1% reinforcement ratio and $\varepsilon=\mu=0$ or M_{y1} . From Fig. 3-a, the yield point moment M_{y1} is obtained as 74 t.m. The yield point moment M_{y2} corresponding to a reinforcement ratio of 2% was obtained as 127 t.m from Fig. 3-b, and that corresponding to a reinforcement ratio of 3% was obtained as 178 t.m from Fig. 3-c. It was observed that when the tension reinforcement ratio was doubled (from 1% to 2%) and tripled (from 1% to 3%), the corresponding yield point moment ratios were increased by 70% and 140% respectively.

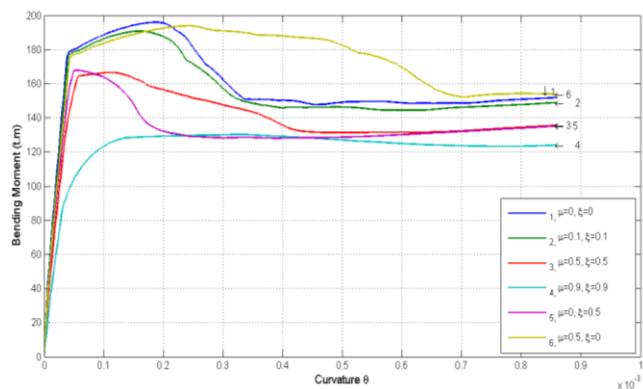
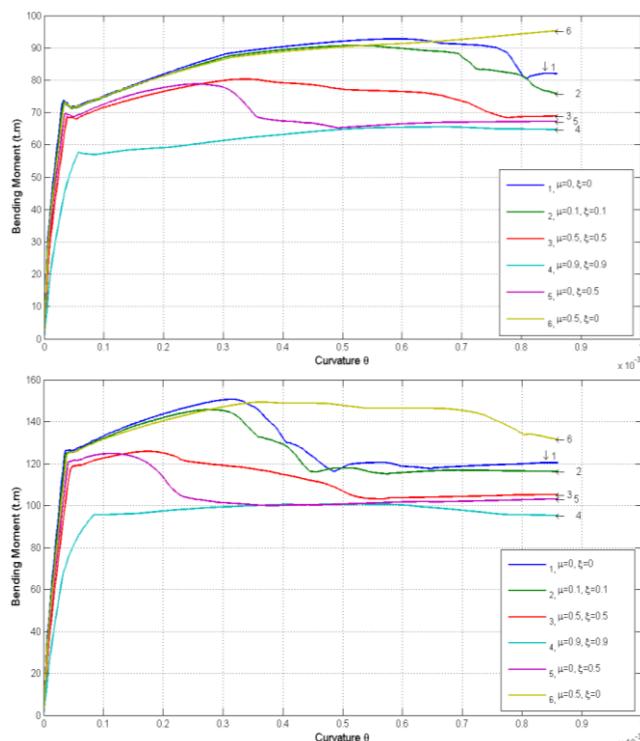


Fig. 3, Moment-Curvature plots, a, $\rho=%1$ of A_g , b, $\rho=%2$ of A_g , c, $\rho=%3$ of A_g

The $EI-M$ curves for $\rho= \%$ 1, %2, %3 are plotted in Figures 4-a, 4-b, and 4-c respectively. These figures show the effective EI variations at different levels of section moment. The slope in the $M-\theta$ diagrams (figures 4-a, 4-b, and 4-c) lead to the conclusion that for a reinforcement ratio of 1% and a bending moment of up to 20 t.m, the EI remained constant at a value equal to EI before the occurrence of cracking.

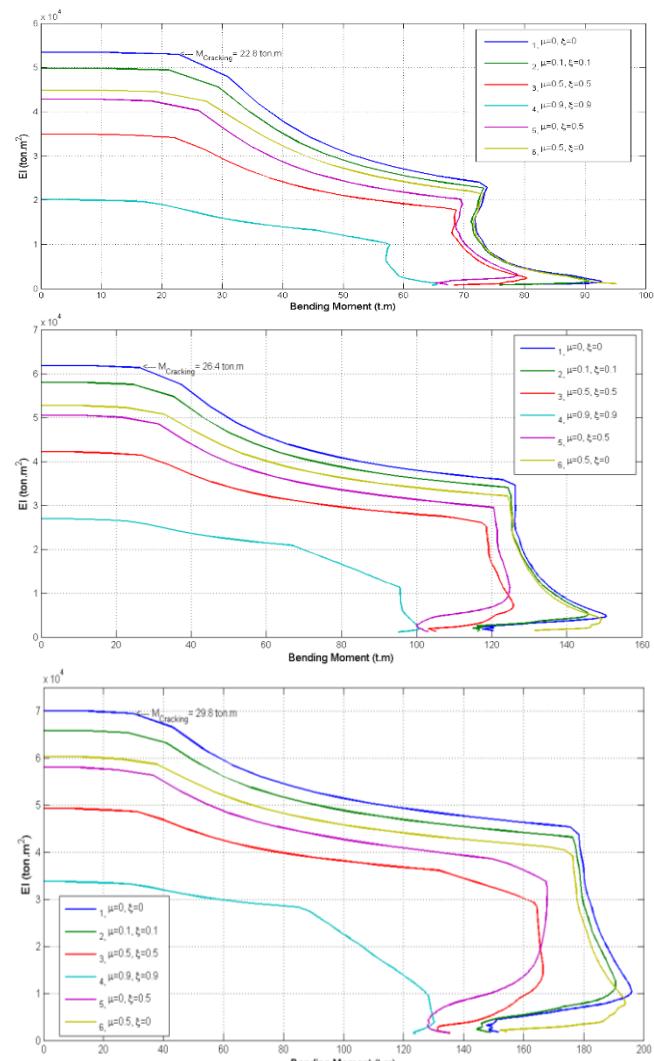


Fig. 4, Effective EI-Moment plots, $\rho=%1$, %2 and %3

In this case, EI is represented by the symbol EI_{cr1} . Thus, EI_{cr1} , EI_{cr2} , and EI_{cr3} are equal to $5.4e^4 \text{ t.m}^2$, $6.2e^4 \text{ t.m}^2$, and $7e^4 \text{ t.m}^2$ respectively. As expected, increasing the section reinforcement ratio did not have a significant effect on the EI values of the cracked surface. Moreover, as can be seen in Figures 3 and 4, the curves are not necessarily parallel and might intersect after the yield point. This behavior is mostly due to the fact that concrete and steel have nonlinear properties and that concrete module of elasticity does not remain constant at different compressive stress levels. Through reverse engineering, the final factored load that Section 1 can tolerate at different reinforcement ratios of 1, 2, and 3 percent can be calculated from the ACI318 Code. For example, if $\rho=1\%$, then the maximum factored moment that the section can tolerate was obtained as 54 t.m. By setting M_{yl} equal to 74 t.m from Fig. 3-a, M_{ul}/M_{yl} was obtained as 0.73. In Figures 5-a to 5-c, the bending moment diagrams for a non-dimensional length $L=1$ of the beam subjected to various factored loads are shown. The M_{u2}/M_{y2} ratio is obtained from figures 3-b and 5-b as 0.73 which is equal to the value obtained for M_{u2}/M_{y2} from figures 3-c and 5-c. The corresponding EI for each point in the moment diagram can be extracted from the related diagrams in Fig.4. Thus, the EI variation along the beam was obtained. As expected, the section developed cracks at a small distance from the support, leading to a decrease in EI .

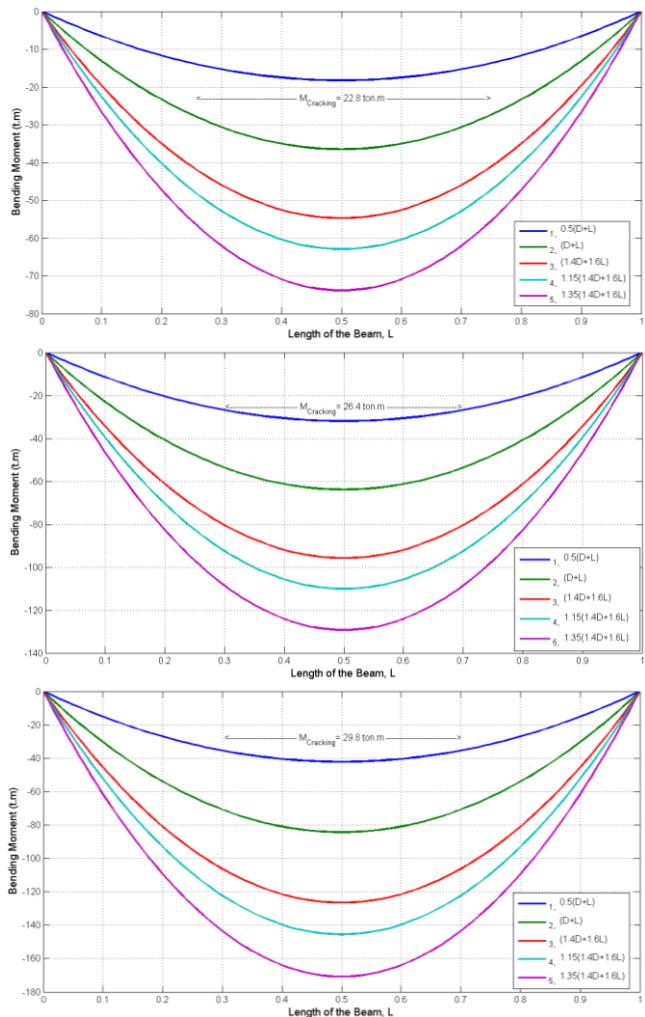


Fig. 5. Bending moment diagrams for a non-dimensional length, $\rho=%1, \%2$ and $\%3$

The diagram in Figure 6-a shows the effective EI to the uncracked EI ratio at different reinforcement ratios.

uncracked EI ratio (for 1% reinforcement ratio) variation at different factored load levels. In this figure, “ μ ” and “ ε ” were assumed to be zero and the section was assumed to be isotropic as well as homogeneous. At $\rho=1\%$, increasing the load to 35% above the allowable limit caused 20% of the midspan length to enter the nonlinear zone and reduced EI to approximately 17% of that for the uncracked section. Moreover, at $\rho=1\%$, the EI at midspan was 55% of that obtained for the uncracked section. At $\rho=2\%$ and $\rho=3\%$, the EI was obtained as 63% and 70% respectively (figs. 6-b and 6-c). In addition, as the reinforcement ratio increased, the true EI section form along the beam tended to be more open. Figure 7 shows variation of EI/EI_{cr} along the beam for different “ μ ” and “ ε ” values. In Figure 7-a ($\rho=1\%$), a comparison is made between the $\mu=\varepsilon=0$ and the $\mu=\varepsilon=0.5$ curves, which reveals a 35% and a 30% change in EI/EI_{cr} at the support (uncracked section) and at the mid-point of the support (maximum bending) respectively. As can be seen, the shapes of EI/EI_{cr} variation curves are different in the three cases $\rho=1\%, 2\%, 3\%$, reaching as reinforcement ratio increases values of 0.38, 0.47, and 0.52 respectively. No two curves in Figures 7 (a, b, and c) are parallel.

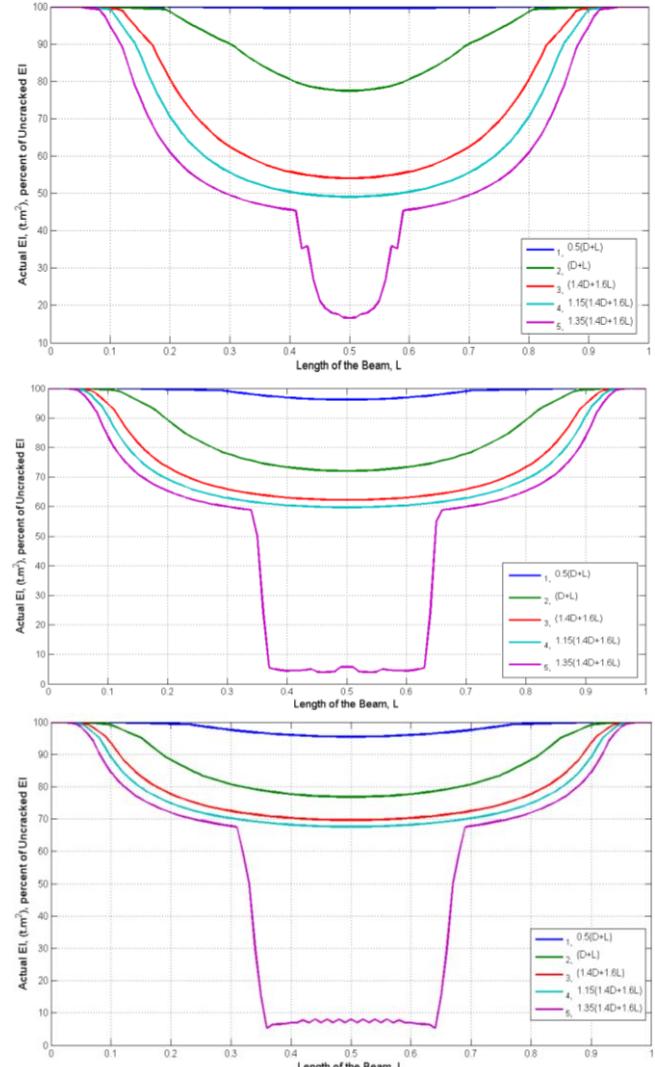


Fig. 6, Effective EI to the uncracked EI ratio, $\rho=%1, \%2$ and $\%3$



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The reason for this is the nonlinear variation of concrete modulus of elasticity due to the applied compressive stresses. In other words, the nonlinearity of the $M-E$ curve leads to non-parallel curves in Figure 7. To explain further, we can say that if the parameters ε (concrete compressive strength change) and μ (concrete density change) are equal to 0.1 ($\mu=\varepsilon=0.1$), then at $\rho=1\%$ reinforcement, only 92% of the section flexural strength would be unavailable; while if $\mu=\varepsilon=0.5$, then this availability is reduced to 65%. The corresponding values for $\rho=2\%$ are 94% and 75%, and for $\rho=3\%$ are 95% and 76% respectively. As the reinforcement ratio decreased, the effect of non-uniform parameters (strength and density) became more pronounced.

Figure 8 depicts the variation of the final factored load applied to the beam against its mid-span displacement. For example, in Figure 8-a, the beam designed according to the ACI-318 Code for tolerating a uniformly distributed load of 1100 kg/m with a 7 cm allowable instantaneous deflection was designed for a reinforcement ratio of $\rho=1\%$. If the same load were applied at $\mu=\varepsilon=0.5$, then the same beam would demonstrate an instantaneous deflection of 10.5 cm.

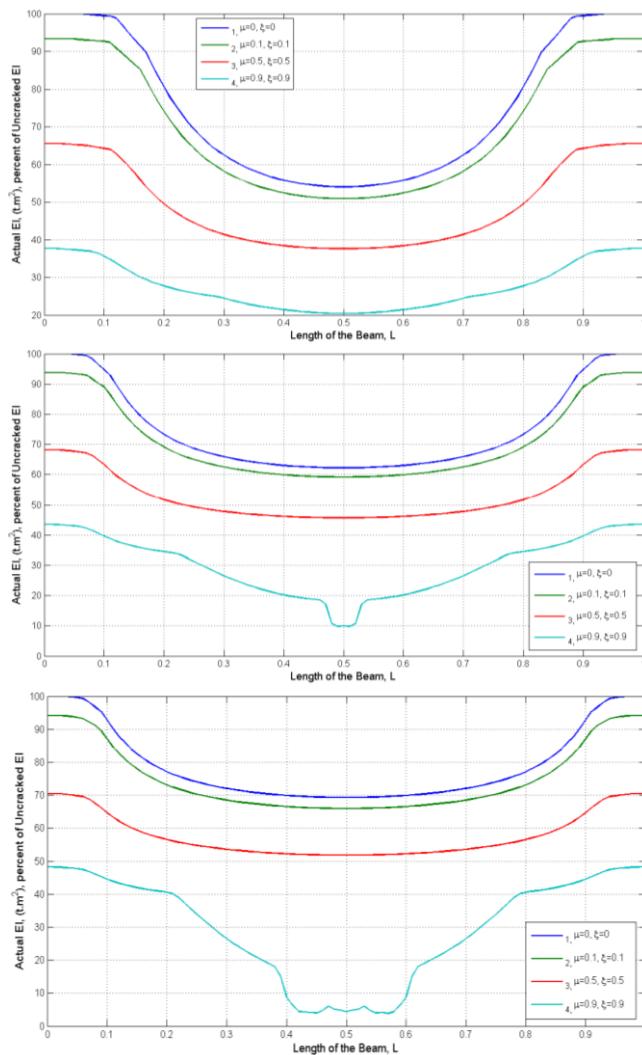


Fig. 7. Effective EI to the uncracked EI ratio, $\rho=1\%$, $\rho=2\%$ and $\rho=3\%$

To avoid the increased deflection and maintain it at less than 7 cm, we would have to reduce the maximum allowable load to 850 kg/m. Now, if $\mu=\varepsilon=0.9$, then we would require 18.5 cm of instantaneous deflection at the allowable design load, and the maximum allowable load that can be applied would

be equal to 589 kg/m.

Figure 9 shows variations of the maximum final factored moment applied to the beam against the maximum mid-span deflection of the beam. Like figure 8, the limited variations of allowable applied loads as well as the instantaneous deflection resulting from these loads in each case are shown. Once a beam has been designed according to concrete codes provisions and the non-uniformity of concrete parameters (due to construction and executive faults) in this beam is significant but not very great (i.e., between 0.1 to 0.5) then the beam would face problems regarding its serviceability if its design is based on maximum instantaneous deflection. However, if the beam design is not based on maximum instantaneous deflection, there would be no particular problems regarding utilization of the beam.

It must be noted that at high reinforcement ratios ρ , using beams with $\mu=\varepsilon>0.5$ is not at all advisable and must be avoided, even if results obtained from elastic bending equations governing the section recommend such an application. As can be seen in figures 9-b and 9-c, deflection increases so dramatically at $\mu=\varepsilon>0.5$, that the beam section might undergo complete rupture.

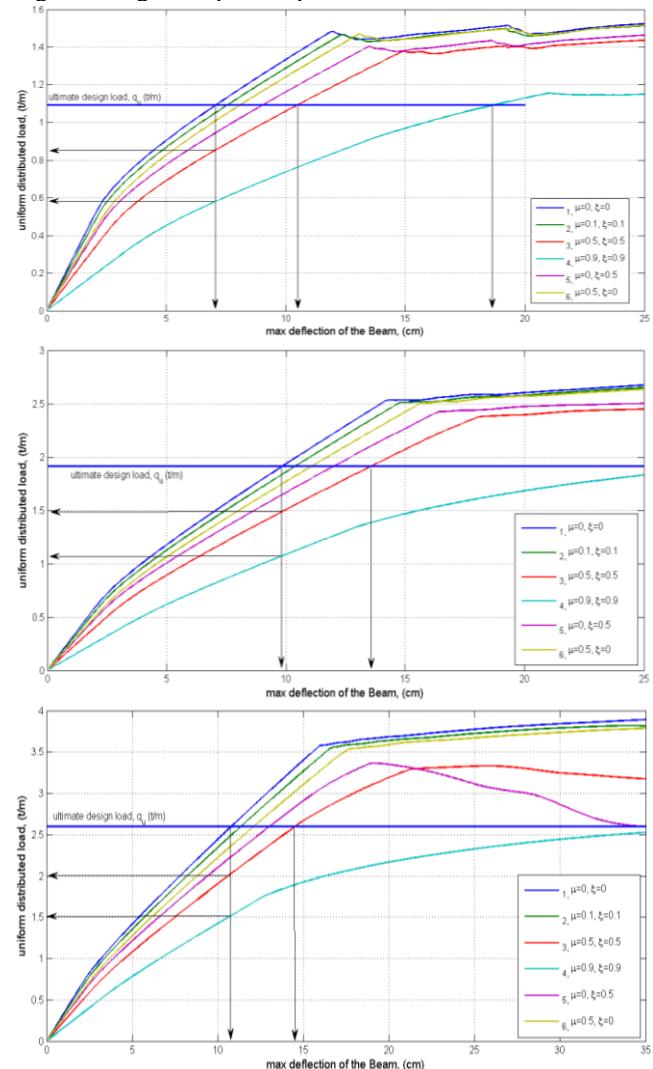


Fig. 8. Variation of the final factored load applied to the beam against its mid-span displacement, $\rho=1\%$, $\rho=2\%$ and $\rho=3\%$

In table 1, the maximum deflections obtained for the beam from the following three methods are compared: the OpenSees finite element software, the Cracked Sections Analysis conducted through MATLAB software, and the ACI318 Code method. Tables 1-a, 1-b, and 1-c which present the results obtained for reinforcement ratios of 1%, 2%, and 3% respectively, show excellent agreement between the OpenSees and MATLAB results, with a difference between corresponding results of less than 0.1 percent. Therefore, the Cracked Sections Analysis offers significant efficiency for calculating the exact inelastic deflection in reinforced concrete beams.

The values for deflection obtained from the formula proposed in the ACI318 for various amounts of n ($n=0.5, 1, 2, 3$) are shown in tables 1-a to 1-c. As can be observed, in all cases, the ACI318 values for deflection are overestimations of the real deflection values. This means that the effective stiffness used in the ACI318 is less than that employed in the exact analysis. Moreover, the obtained results point out to the fact that for serviceability states loading levels, using $n=0.5$ in Branson equation produces more acceptable results, while for ultimate (final) limit states loading levels, setting $n=1$ creates more realistic and more exact results which are closer to those obtained from analytical methods. For this reason, values more than these are not recommended for the reinforced concrete I shaped beam under study.

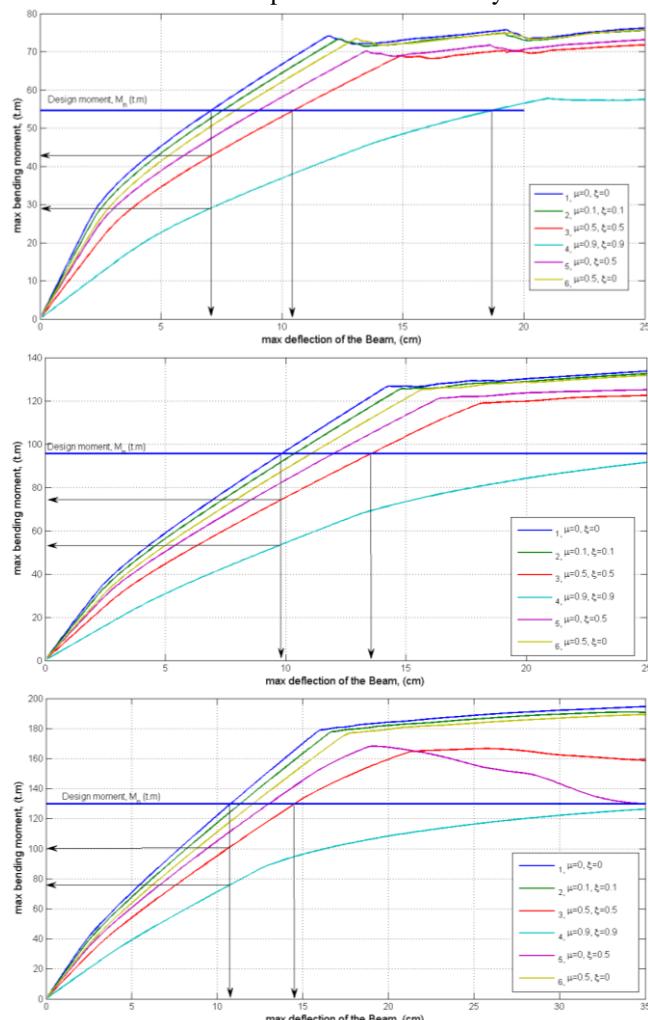


Fig. 9. Variations of the maximum final factored moment applied to the beam against the maximum mid-span deflection of the beam, $\rho=1\%$, 2% and 3%

V. CONCLUSION

The objective of this paper was to compare exact finite element, ACI318 Code and analytical deflections of concrete I shaped beams. The deflection in a reinforced concrete beam is a function of two variables, namely, E and I . Many researchers have endeavored to determine the amount of deflection in reinforced concrete beams. In doing so, they have used various methods including mathematical methods and finite element analyses, as well as linear and non-linear approaches. Few researchers have studied the effects of inhomogeneous behavior of concrete on inelastic deflection in beams. In this article, the deflection in such beams was studied in three different ways and their results compared and further discussed. Results from the simplified method agree well with finite element and fiber layer models.

For serviceability states loading levels, using $n=0.5$ in Branson equation produces more acceptable results, while for ultimate (final) limit states loading levels, setting $n=1$ creates more realistic and more exact results which are closer to those obtained from analytical methods. For this reason, values more than these are not recommended for the reinforced concrete I shaped beam under study.

In principle, if a beam is designed in accordance with provisions of construction codes, it is capable, due to the various safety factors considered at the design stage, of tolerating approximately 35% more load than the ultimate limit state without undergoing failure. As the beam stays in the linear elastic state, only the non-allowable instantaneous deflection demonstrates itself elastically. However, due to the non-uniform properties of concrete, namely, μ and ϵ , this overstrength factor against overloading is dramatically reduced, so much so that practically there remains no overstrength in the beam against overloading for those values of ϵ higher than 0.6 (i.e., for $\mu=\epsilon>0.6$).

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Tab. 1, Maximum midspan deflections obtained from the analysis

As=%1 of Ag								
Load combination	Max. midspan deflection (cm)				ACI-318			
	qu (t/m)	matlab	opensees	error%	n=0.5	n=1	n=2	n=3
0.5(D+L)	0.3648	1.4278	1.4275	0.021%	1.5288	1.6223	1.8119	2.0016
(D+L)	0.7296	3.3750	3.3708	0.125%	3.6814	4.4841	5.8022	6.5871
1.4D+1.6L	1.0944	7.1078	7.1062	0.023%	6.071	7.7076	9.7936	10.5663
1.15(1.4D+1.6L)	1.2586	9.0779	9.0735	0.048%	7.1964	9.2146	11.5447	12.2740

As=%2 of Ag								
Load combination	Max. midspan deflection (cm)				ACI-318			
	qu (t/m)	matlab	opensees	error%	n=0.5	n=1	n=2	n=3
0.5(D+L)	0.6498	2.2405	2.2403	0.009%	2.8531	3.0927	3.4169	3.5880
(D+L)	1.2996	5.7738	5.7762	-0.042%	6.134	6.7752	7.3139	7.4487
1.4D+1.6L	1.9494	10.1072	10.1070	0.002%	9.517	10.4965	11.1155	11.2159
1.15(1.4D+1.6L)	2.2418	12.1518	12.1504	0.012%	11.0587	12.1752	12.8158	12.9054

As=%3 of Ag								
Load combination	Max. midspan deflection (cm)				ACI-318			
	qu (t/m)	matlab	opensees	error%	n=0.5	n=1	n=2	n=3
0.5(D+L)	0.8664	2.6651	2.6629	0.083%	3.5278	3.5989	3.6673	3.6913
(D+L)	1.7328	6.4993	6.5016	-0.035%	7.1551	7.3012	7.3892	7.4045
1.4D+1.6L	2.5992	10.8026	10.8030	-0.004%	10.8002	11.0045	11.0992	11.1100
1.15(1.4D+1.6L)	2.9891	12.8354	12.8216	0.108%	12.4438	12.6711	12.7675	12.7771

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NOTATION

The following symbols are used in this paper:



A_s	cross sectional area of longitudinal bars;	I_{ucr} uncracked moment of inertia;
A_g	Area of concrete cross-section	M_a applied moment;
E_c	elastic modulus of concrete;	M_y yielding moment.
E_s	elastic modulus of Steel bars;	M_u ultimate moment;
f'_c	concrete compressive strength;	M_{cr} cracking moment;
f_r	Modulus of rupture of concrete	w distributed load;
f_t	Concrete tensile stress due to applied loads	δ mid-span deflection;
f_y	Specified yield strength of non-prestressed reinforcement	ρ reinforcement ratio;
h	Overall thickness or height of member	ω_c concrete density;
I_e	effective moment of inertia;	ε concrete compressive strength change;
I_g	moment of inertia of the gross section;	μ concrete density change;
I_{cr}	cracked moment of inertia;	