



## Influence of Minimum Tension Steel Reinforcement on the Behavior of Singly Reinforced Concrete Beams in Flexure

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### KEY WORDS

Cracking load, Flexural strength, High strength concrete, Minimum flexural reinforcement, Reinforced concrete

### ABSTRACT

*The requirements of minimum flexural reinforcement in the last decades have been a reason for controversy. The structural behavior of beams in bending is the best way of investigating and evaluating the minimum reinforcement in flexure. For this purpose, twelve singly reinforced concrete beams with a rectangular cross-section of (125 mm) width by (250 mm) height and (1800 mm) length were cast and tested under two-point loads up to failure. These beams were divided into three groups with different compressive strengths (25, 50, and 80 MPa). Each group consists of four beams with different amounts of tension steel reinforcement approximately equal to (0%  $A_{smin}$ , 50%  $A_{smin}$ , 100%  $A_{smin}$  and 150%  $A_{smin}$ ), two bar diameters ( $\varnothing 6$  mm and  $\varnothing 8$  mm) were used as the longitudinal tension reinforcement with different yield and ultimate strengths, the minimum amount of reinforcement required is calculated based on ACI 318M-2014 code. The results show that for the reinforced concrete beams, the flexural reinforcement in NSC beams increases the first cracking load and the increment increased with an increasing amount of reinforcement, while for HSC beams the increasing in first cracking load are very little when the quantity of reinforcement less than the minimum flexural reinforcement and increased with the increasing amount above the minimum flexural reinforcement. The equation of ACI 318M-14 code gives adequate minimum flexural reinforcement for NSC and overestimate value for HSC up to (83 MPa), A new formula is proposed for HSC rectangular beams up to (90 MPa) concrete compressive strength by reducing the equation of ACI 318M-14 code for minimum flexural reinforcement by a factor depending on concrete compressive strength.*

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## 1. Introduction

It is an important factor in the design of a reinforced concrete member to guarantee that the member not experience brittle failure and has large deformation at maximum load. The reinforcement amount is a very important factor to influence the nature of the failure that should happen when the concrete element reaches its design strength capacity in order to prevent unwanted failure that may happen without any warning [1].

One type of failure is expected to take place without any warning when a reinforced concrete beam with a very light quantity of reinforcement is needed to satisfy the design, or in case of the cross-section dimensions of the concrete beam are greater than required by strength consideration in respect to architectural design requirements, under overloading the lightly reinforced concrete beam may present sudden major flexural crack and the stored energy in the concrete in tension is transferred immediately into the longitudinal reinforcement, large strain in steel may exceed the steel ultimate capacity of strain resulting a fracture in it causing sudden failure [2].

The ACI 318M-14 [3] and earlier codes editions have many limitations on the spacing and amount of steel reinforcement. These codes are designed to enhance the characteristics of reinforced concrete members and reducing the probability of undesirable failure modes [4]. Historically, the minimum reinforcement requirements have been purposed to achieve one of the following two points:

- To prevent the flexural member having a sudden failure at first cracking
- To allow a failure only at a resistance that is higher than the factored moment, which results from the practical strength load combinations.

Many approaches have been used to estimate the quantity of minimum flexural reinforcement, one of the most common approach used in ACI codes is equating the moment of cracking for an unreinforced section to the flexural strength of the beam after cracking using the full steel yield strength [2]. Cracking moment defined as the moment that produces maximum flexural tensile stress in the outer fiber exceeding the modulus of the rupture of the concrete.

The ACI 318M code in 2014 [3] gives, a minimum amount of steel reinforcement

$$A_{s_{min}} = \sqrt{\frac{f'_c}{4f_y}} b_w d \geq \frac{1.4}{f_y} b_w d \quad (1)$$

where:  $A_{s_{min}}$ : minimum area of non-prestressed longitudinal tension reinforcement ( $mm^2$ )  $f'_c$ : concrete compressive strength (MPa),  $f_y$ : tensile yield strength of non-prestressed reinforcement (MPa),  $b_w$ : web width (mm),  $d$ : distance from extreme compression fiber to centroid of tension reinforcement (mm).

Existing researches indicate that several factors influence the minimum steel requirement in flexure. These include, but are not limited to:

- 1) The influence of compressive strength of concrete where in some cases it is indicated that existing design equations are adequate and in other cases, this may not be the case as for high strength concrete.
- 2) The influence of the cracking load is very significant, which may affect the minimum steel requirement.

This work intends to clarify some of the factors that affect the minimum steel requirements in order to precisely evaluate the available minimum reinforcement equations.

## 2. Experimental work

### 1. Specimen description

To investigate the flexural behavior, twelve singly reinforced concrete beams with a rectangular cross-section of 125 mm width by 250 mm height and 1600 mm clear span were cast, the beam length is selected to satisfy the requirements of shear span greater than twice the beam height and avoid the deep beam. These beams were divided into three groups with different concrete compressive strength (25, 50, and 80) MPa. Each group consists of four beams with different amounts of tension steel reinforcement equal to (0%  $A_{s_{min}}$ , 50%  $A_{s_{min}}$ , 100%  $A_{s_{min}}$  and 150%  $A_{s_{min}}$ ). The minimum amount of reinforcement was calculated based on ACI 318M-14 [3]. Two bar diameters ( $\varnothing 6$  mm and  $\varnothing 8$  mm) with different yield and ultimate strengths were used as the longitudinal tension reinforcement.

When the two bar diameters were used in the same beam as tension reinforcement, the required minimum flexural reinforcement calculation is adopted on the weighted average of the yield stresses of the two bars.

For all beams, 2Ø4mm top bars were used in the shear span only in the compression face to hold the stirrups. Since very low amounts of tension reinforcement were used, the flexural capacity was lower than the diagonal cracking capacity and no shear cracks were expected. However, the beams were over-reinforced for shear failure with Ø6 mm closed stirrups spaced at (100 mm) and (50 mm) on center for groups of beams with compressive strength (25 MPa, 50 MPa) and (80 MPa) respectively. The stirrups and the top bars holding these stirrups were terminated at the boundaries of the constant moment region. The provided concrete clear cover was 20 mm. The variation of supplying the amount of reinforcement from the required amount was unavoidable because of the constant area of the bars used in the work, therefore the bars were used to obtain the supplied area that is closer to the required quantity. The control specimens for each batch consisting of three cylinders for compressive strength test and similarity of splitting tensile strength test, three prisms for modulus of rupture. After casting, all the molds filled with concrete were left in the laboratory for approximately 24 hours for NSC and after 48 hours for HSC, then molds were dismantled. The beams and control specimens were laid in a tank filled with tap water for (28) days. Table 1 shows the tension reinforcement for each beam. The beam reinforced with the required minimum flexural reinforcement according to ACI 318M-14 [3] is used as a control beam for comparison of the performance with the other reinforced beams. The general behavior, experimental cracking load, experimental nominal load, vertical deflection, and failure mode of beam specimens were recorded and compared among each other to explore the importance of the experimental variables.

### II. Proportions of concrete mixes

One of the goals of this study is to investigate the effect of the parametric study related to the different concrete compressive strength of the behavior of the beams. In order to choose the target concrete mixture, several trial mixes for each targeted concrete compressive strength were done in order to determine a distinctive cylinder strength of (25, 50, and 80) MPa at 28 days. Ordinary Portland cement, sand with a maximum size of 4.75mm, fine sand with 600  $\mu$  maximum particle size, coarse aggregate with a maximum size of 12 mm.

### III. Testing program

The beams were tested under the effect of two-point loads up for failure by using a microcomputer controlled electronic testing machine. This machine has a capacity of (200 kN), power-operated, and applying the load continuously rather than intermittently, and without shock. As the beam rested on two rollers support, the load was applied symmetrically to the beam at a distance of 550 mm from the nearest support by using a steel girder (HP150×150× 14) mm with a total length of (700 mm) which has been used to transfer the load from the electronic testing machine to the beam specimen through steel plates of (50 × 125 × 20) mm placed on the beam to avoid localized crushing of concrete as shown in Figure 1. The deflection was recorded by the electronic testing machine at the load points during the test time. All these data were collected in the computer. The crack formation was monitored and recorded by a digital camera placed in front of the specimens.

**Table 1: Specimens reinforcement**

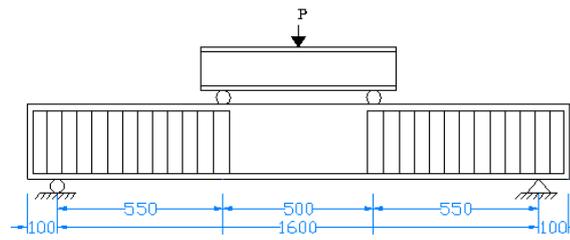
Specimens	$f'_c$ (MPa)	$f_y$ (MPa)	Tension reinforcement	$A_{s_{prov}}$ mm <sup>2</sup>	$A_{s_{min}}$ mm <sup>2</sup>	$\frac{A_{s_{prov}}}{A_{s_{min}}}$ (%)
B25-S0	25	-	-	-	-	-
B25-S 0.5	25	470.0	2Ø6	51.90	82.28	63%
B25-S 1.0	25	382.5	2Ø8	97.53	101.1	96%
B25-S 1.5	25	382.5	3Ø8	146.30	101.1	144%
B50-S0.0	50	-	-	-	-	-
B50-S 0.5	50	470.0	2Ø6	51.90	103.9	50%
B50-S 1.0	50	470.0	2Ø6+2Ø6	103.80	95.00	109%

B50-S 1.5	50	412.8 9	2Ø6+2Ø6	149.40	108.2 0	138%
B80-S0.0	80	-	-	-	-	-
B80-S 0.5	80	382.5 0	2Ø8	97.53	161.7 0	60%
B80-S 1.0	80	382.5 0	3Ø8	146.30	161.7 0	90%
B80-S 1.5	80	382.5 0	2Ø8+2Ø8	195.00	147.8 0	132%

Sika-viscocrete® -5930 as superplasticizer and silica fume were used with the quantities shown in Table 2.

**Table 2: Concrete mixture components**

Material	Units	Concrete compressive strength (MPa)		
		25	50	80
Cement (C)	kg/m <sup>3</sup>	350	520	900
Sand (S)	kg/m <sup>3</sup>	680	620	990
Gravel (G)	kg/m <sup>3</sup>	996	1025	-
Silica Fume (SF)	kg/m <sup>3</sup>	-	-	225
Super-plasticizer (SP)	kg/m <sup>3</sup>	-	1.5	67.5
Water (W)	kg/m <sup>3</sup>	175	146	180
W/Cm	-----	0.5	0.28	0.16



**Figure 1: Test setup of beam specimens**

### 3. Test Results and Analysis

The experimental cracking load ( $P_{cr}$ ) has been identified as the load corresponding to the deviation of the load-deflection curve, and the experimental yielding load ( $P_y$ ) corresponding to the flat plateau that is observed in the load-deflection curve. In the case it is not possible to determine the flat plateau from the curve, the method that has been summarized by Park [5] is used to estimate yielding load and deflection of an equivalent elasto-plastic system with an elastic stiffness as the secant at 75% of maximum load and the yielding strength equal to the peak. Other researchers such as Pam et al [6], Elrakib [7], and Jang et al [8] also used this method. The experimental nominal load ( $P_n$ ) represents the peak load during testing. Two modes of failure were encountered during the beam tests, namely, crushing of concrete at the edge in compression after yielding of tension steel and rupture of the rebar at the edge in tension. The reserved flexural strength (defined as the ratio of yielding load to cracking load) is used to evaluate the test results of beams.

#### *1. Load –deflection behavior of tested beams*

The load - deflection curves for the beams (B25-S0.0), (B25-S0.5), (B25-S1.0) and (B25-S1.5), also the beams (B50-S0.0), (B50-S0.5), (B50-S1.0) and (B50-S1.5) are shown in figures 2-3, respectively. It can be seen that the first stage of the curves is linear up to cracking load for all beams. For the beams (B25-S0) and (B50-S0) no increase in the load occurred after cracking, whereas in the other remaining beams for the same compressive strength the loads increased after cracking accompanied by more cracks, thus the load-deflection curve starts with deviation until the yielding of flexural reinforcement, the formation, and propagation of cracks is related to a reduction in stiffness of the beam. The stage after yielding is most inclined and depends on the strain hardening of reinforcement

on the tension side. Also, it can be noted that after cracking load was reached for beams (B25-S0.5) and (B50-S0.5), the curve becomes inclined and the yielding load occurred speedily at short deflection compared with control beam (B-S1.0) for each group, then the load increased until failure. The beams (B25-S0.5) and (B50-S0.5) exhibited failure with relatively small deflection. This is due to the amount and type of tension steel. The reserved flexural strength was obtained less compared with the control beam. For beams (B25-S1.5) and (B50-S1.5), after cracking linear relationship exists between load and deflection up to the point of steel yielding which occurs at higher load and further deflection compared with control beam (B-S1.0). Next, the load increased slightly with an increasing deflection after yielding.

In general, it was noted that the reinforcement of beams (B25-S0.5) and (B50-S0.5) is not enough to give the appropriate ductility. Ductility can be defined as the ability of the element to submit to plastic deformation without loss in its strength. For beams (B25-S1.5) and (B50-S1.5) there was an improvement in the deflection after yielding with an increasing amount of flexural reinforcement, and the increase in the tension steel raised the load-carrying capacity of beams.

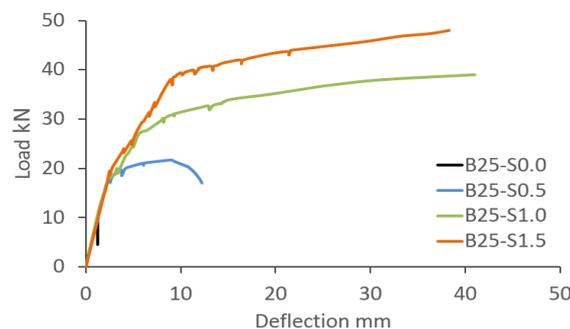


Figure 2: Load-deflection relationship for beams with  $f'_c = 25.3$  MPa

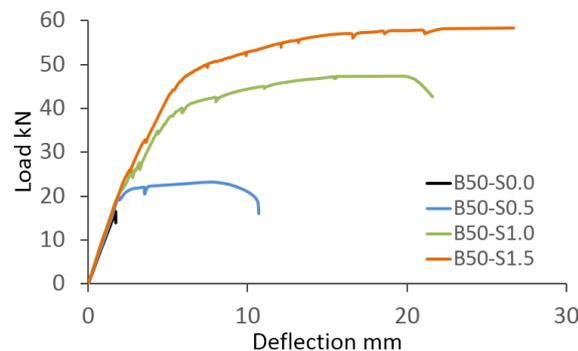


Figure 3: Load-deflection relationship for beams with  $f'_c = 48.45$  MPa

Figure 4 illustrates the load-deflection curves for beams (B80-S0.0), (B80-S0.5), (B80-S1.0), and (B80-S1.5). It can be observed that although reducing the amount of tensile reinforcement in the beam (B80-S0.5) the deviation takes place in the curve after cracking to the yielding of steel with a linear relationship between load and deflection. Thereafter high inclination happened after steel yielding exhibited adequate deflection with increasing load-carrying capacity of the beam. For the beam (B80-S1.5) with larger flexural reinforcement than the control beam (B80-S1.0), the load-deflection curves for these beams show that there was an increase in cracking load compared with the beam (B80-S1.0), and it can be observed that after cracking the curve deviation continued until the steel yielding at higher load and deflection compared with control beam (B80-S1.0). Then the load increased softness with increasing deflection. For beams with compressive concrete strength (83 MPa) it is possible to use flexural reinforcement (60%) of minimum reinforcement required by ACI 318M-14 [3] code with adequate ductility after yielding as illustrated in Figure 4.

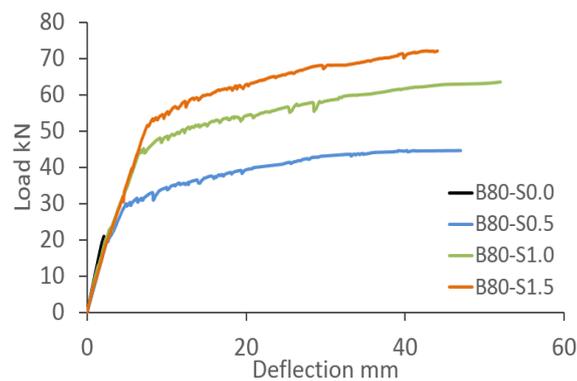


Figure 4: Load –deflection relationship for beams with  $f'_c=83$  MPa

## II. Failure mode of tested beams

For the beams (B25-S0.0), (B50-S0.0), and (B80-S0.0) which contain no flexural reinforcement within the pure bending zone the failure occurs at first cracking load at (10.0, 16.4, and 20.8 kN) respectively as shown in Table 3. All beams collapsed by cutting into two separate segments.



Figure 5: Failure of plain concrete beams

For beams (B25-S0.5), (B50-S0.5), and (B80-S0.5) with flexural steel approaching (50%) of ACI 318M-14[3] required minimum reinforcement, it is found in beams (B25-S0.5) and (B50-S0.5) as the load was increased another few flexural cracks initiated. The cracks extended with increasing load and increased in width. The yielding of reinforcement occurred rapidly, no more cracks were observed. The observed first crack propagated upward with increasing in width, resulting in beams failing due to rupture of the tension steel reinforcement. The rupture of the rebar occurred suddenly and instantaneously. The low amount of reinforcement made the beam reach the yield strength rapidly with extending cracks. This means that the stresses in reinforcement were close to yielding stresses when the first crack started. While for beam (B80-S0.5) the number of cracks in the constant moment region and deflection increased as the applied load increased until yielding of reinforcement, then the crack width and depth increased rapidly and sub-cracks were observed with increasing load accompanied with increasing deflection. This beam behaved with ductility due to the large deformation before failure. The cracks started with the deviation when reaching the compression zone. The flexural cracks also appeared outside the constant moment zone. The crack pattern of these beams is shown in Figure 6.



**Figure 6: Cracks pattern of beams (25-S0.5), (50-S0.5), and (80-S0.5)**

For beams (B25-S1.0), (B50-S1.0), and (B80-S1.0). The number of cracks in the constant moment region and deflection increased as the applied load increased until yielding of flexural reinforcement, then the crack width and depth developed with increasing load accompanied with increasing deflection. Flexural cracks appeared outside the constant moment region. For beam (B25-S1.0) the failure occurred at a load of (39 kN) when the compressive stresses exceeded the compressive strength in concrete and the concrete crushed in the compression zone. For beam (B50-S1.0) the maximum applied load occurred at (47.35 kN) with a greater number of cracks, then the load decreased with increasing deflection and the failure happened due to rupture of the tension steel reinforcement.

**Table 3: Experimental loads for beams**

Specimen	$f_c'_{exp}$ (MPa)	$\frac{As_{prov}}{As_{min}}$ %	Cracking load (kN)	Pcr relative to the control beam %	Yielding load (kN)	Py relative to the control beam %	Nominal load (kN)	Pn relative to the control beam %	Reserve strength Py/ Pcr
B25-S 0.0	25.3	0	10.00	-	-	-	-	-	-
B25-S 0.5	25.3	63	16.60	- 7.78	18.4	- 33.10	21.58	- 44.67	1.10
B25-S 1.0	25.3	96	18.00	-	27.5	-	39.00	-	1.52
B25-S 1.5	25.3	144	19.50	8.33	39.0	41.80	48.00	23.00	2.00
B50-S 0.0	48.4	0	16.40	-	-	-	-	-	-
B50-S 0.5	48.4	50	20.00	-11.70	20.2	-49.50	23.50	-50.30	1.01
B50-S 1.0	48.4	111	22.64	-	40.0	-	47.35	-	1.76
B50-S 1.5	48.4	140	25.80	13.90	50.0	25.00	58.30	23.10	1.94
B80-S 0.0	83	0	20.80	-	-	-	-	-	-
B80-S 0.5	83	59	21.20	-10.10	29.6	-32.80	44.60	-29.70	1.39
B80-S 1.0	83	89	23.60	-	44.1	-	63.50	-	1.87
B80-S1.5	83	129	32.00	35.60	57.0	29.20	72.00	13.38	1.78

For beam (B80-S1.0) the hair cracks grew rapidly in-depth and slightly in width and the sub-cracks appeared with increasing load and linked with other cracks accompanied by increasing in deflection and some parts of the concrete cover spalled. Next the cracks started with the deviation reaching the

compression zone and the beam failed at a load of (63.5 kN) with a loud thump due to the explosion of concrete in the compression zone. Unlike the previous group of beams, these beams exhibited ductile behavior by increasing the number of cracks with smaller width and uniformity in crack distribution due to the amount of reinforcement provided that resulted in a better behavior for beams. The crack pattern for these beams is shown in Figure 7.

For beams (B25-S1.5), (B50-S1.5), and (B80-S1.5) with steel in tension equal to (144%), (140%), and (129%) of required minimum flexural reinforcement calculated based on ACI 318M-14 [3] code. The number of cracks increased until reaching the load causing the yielding of flexural reinforcement, then with increasing load the crack propagates upward and increased in width slightly. Additional flexural cracks appeared outside the constant moment zone. For beam (B80-S1.5) the hairline cracks grew rapidly in-depth and slightly in width, and sub-cracks appeared with increasing load and linked with other cracks accompanied by increasing deflection and some parts of the concrete cover spalled. The diagonal shear cracks were observed near the end of the test. The beams (B25-S1.5) and (B50-S1.5) failed due to the yielding of flexural reinforcement followed by the crushing of concrete. The crack pattern at the end of the test is shown in Figure 8.



Figure 7: Cracks pattern of beams (25-S1.0), (50-S1.0), and (80-S1.0)



Figure 8: Cracks pattern of beams (25-S1.5), (50-S1.5), and (80-S1.5)

### III. Effects of reinforcement on first cracking and nominal load

Table 3 shows the first cracking load and nominal load for beams with compressive strength (25.3, 48.45, and 83) MPa. It can be seen that the influence of existing flexural reinforcement of the cracking load has an important effect on beams made from NSC and HSC compared with the beams without reinforcement. The justification for this is that the concrete tends to crack when the tensile stresses in concrete reach the modulus of rupture, and because of existing of flexural reinforcement adjacent to the tension outer fiber of concrete the initiated crack required enough deformation in longitudinal steel. This means a higher load to initiate cracking. The cracking load increased with the increasing amount of reinforcement around the minimum reinforcement, and the difference in cracking load for beams made from low strength concrete depends on the amount and type of reinforcement. Decreasing the reinforcement for NSC beams (B25-S0.5) and (B50-S0.5) is considered insufficient since the yielding load is close to cracking load resulting in reserve strength equal to (1.1) and (1.01), respectively, causing the fracture loading to decrease with extended cracking, and failure occurs without warning. For other beams, the reserve strength increased due to an increase in the amount and type of reinforcement resulting in increased yield strength.

When the flexural reinforcement increased to (144%) and (140%) as for beams (B25-S1.5) and (B50-S1.5), the increase in cracking load was about (8.33%) and (13.9%), respectively, with respect to cracking load of the control beam for each group. The flexural capacity rose by about (23%) and (23.1%) with respect to the corresponding values of the control beam. The influence of existing low reinforcement of the cracking load has a little effect for beam (B80-S0.5) made from high strength concrete compared with the beam (B80-S0) without reinforcement since the load required to make the steel deform close to load causing cracking of concrete in the beam without reinforcement. The difference in cracking load depends on the amount of reinforcement, and the cracking load increased with an increasing amount of flexural reinforcement. That means the cracking load is influenced by the amount of tension steel, with low tension reinforcement there is little influence of tension steel, especially in HSC. The reserved strength for beam (B80-S0.5) is higher than the value of ACI 318M-14 [3] that recommended  $(P_y/P_{cr})$  at least (1.33). The experimental nominal load capacity for NSC and HSC beams increased with the amount of flexural reinforcement since after yielding of flexural steel the crack developed upward resulting in an increase of stress of the compressive stress block causing in a reduction in the depth of neutral axis and increase in the lever arm, therefore the strength of the beam increased and depends on the amount and properties of reinforcement. Figure 9 illustrates the increase in the cracking load for beams with varying amounts of minimum flexural reinforcement compared with the beam without reinforcement for different concrete compressive strength values. It can be seen that for beams with NSC the reinforcement is more effective in increasing the cracking load. In contrast, the increase in cracking load is much less significant for the HSC with light tension steel and increases with increasing flexural steel above the minimum value. Figure 10 indicates that the increase in flexural reinforcement leads to increasing load-carrying capacity of the beams for the same concrete compressive strength because of decreasing in the depth of stress block and increasing in the arm of resistance moment, and with increasing compressive strength of concrete the quantity of reinforcement increased and the load-carrying capacity of the beams increased too.

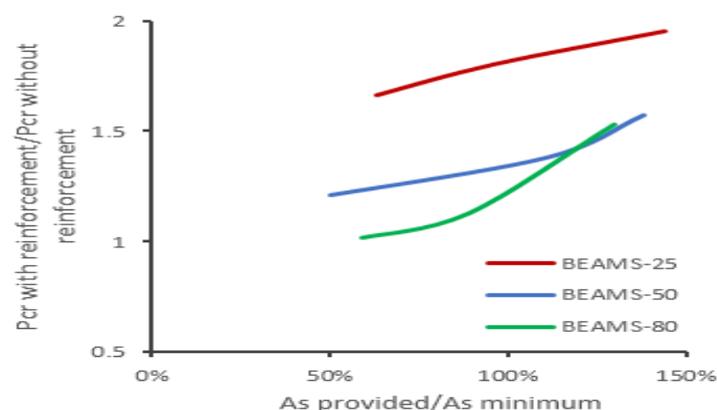


Figure 9: Relationship between percentage of reinforcement

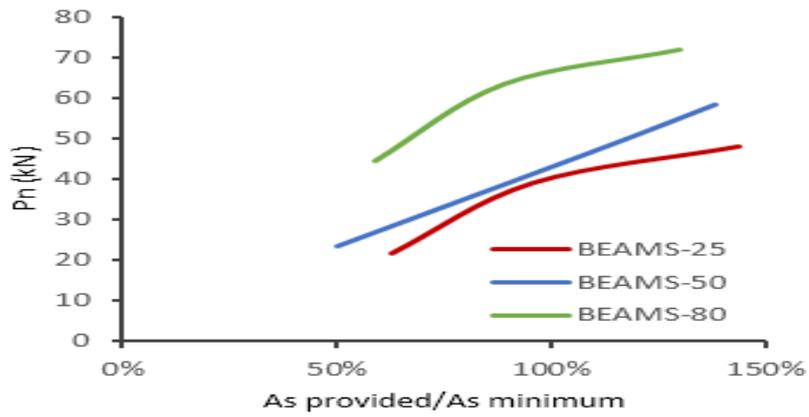


Figure 10: Relationship between the percentage of reinforcement and nominal load for testing beams

Figure 11 shows that the increase in flexural reinforcement leads to increasing reserve strength for the same concrete compressive strength because of the increased yield load caused by increasing flexural steel. For HSC with tension-steel (129%) the reserve strength decreases due to increasing cracking load with an increasing amount of reinforcement. On the other hand, it can be seen from the curves of beams (25, 50, and 80) MPa that the increase in compressive strength of concrete causes an increase in reserve strength, this is attributed to an increase in providing flexural reinforcement with increasing the strength of concrete.

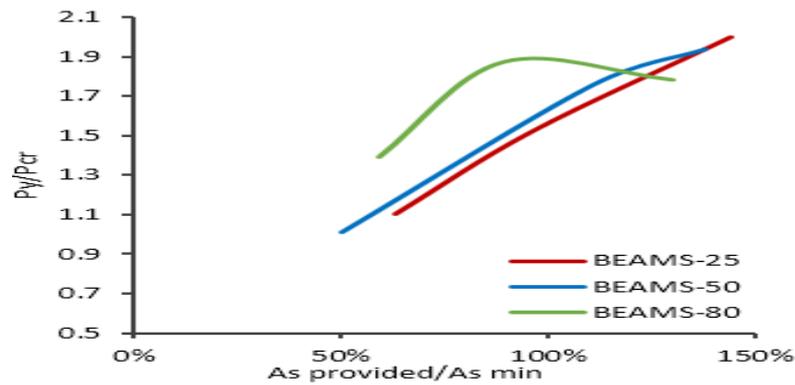


Figure 11: Relationship between the percentage of reinforcement and the reserve flexural strength

IV. Effect of compressive strength of concrete on first cracking and nominal load

It can be noted that for beams made of plain concrete such as (B25-S0.0), (B50-S0.0), and (B80-S0.0) these beams failed at initiating the first crack at different loads. The reason for the difference is due to the increase in flexural strength of concrete with an increase in concrete strength causing increased cracking load, and the load causing the first crack very sensitive to the root of the compressive strength of concrete. The load-deflection curves for these beams are shown in Figure 12.

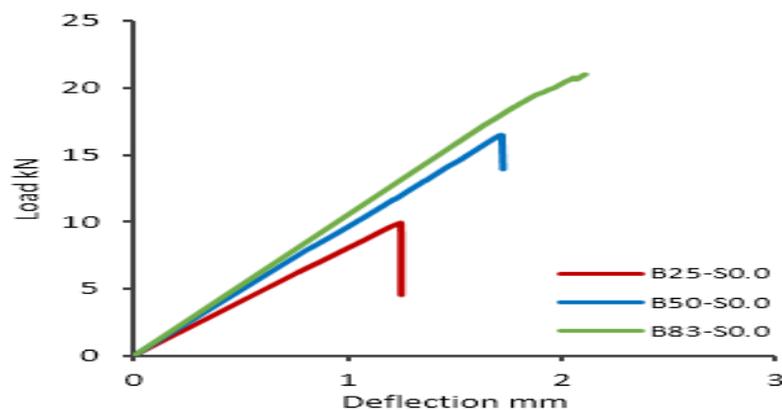


Figure 12: Load-deflection curves for beams (B25-S0.0), (B50-S0.0), and (B80-S0.0)

Figure 13 shows the relationship between the compressive strength of concrete with cracking load for beams reinforced approximately close to the same percentage of minimum flexural reinforcement. It can be noted that as the compressive strength of concrete increases the cracking load increased and the increment increases with an increasing amount of tension reinforcement.

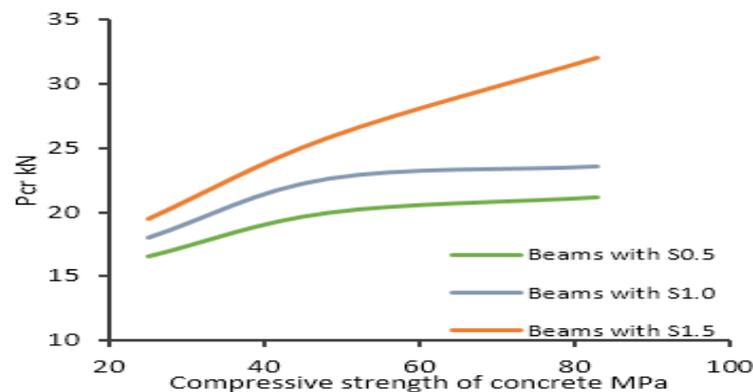


Figure 13: The relationship between the compressive strength of the concrete and cracking load

The relationship between the compressive strength of concrete and the nominal load of beams is shown in Figure 14. The nominal load increased with concrete compressive strength. This is attributed to increasing of minimum flexural reinforcement. The nominal load increased by a high percentage compared with the increasing cracking load.

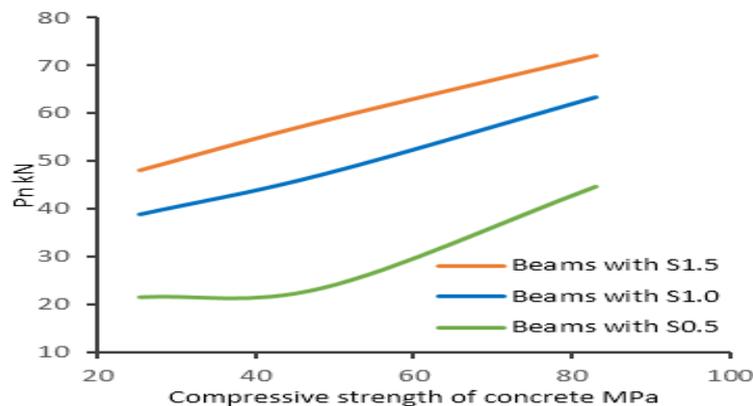


Figure 14: The relationship between the compressive strength of concrete and nominal load

#### 4. The Proposed Formula for Rectangular HSC Beams

According to ACI 318M-14 [3] code, it can provide at least third more flexural reinforcement than that required by the analysis (which means  $P_y/P_{cr} \geq 1.33$ ) at every section where the equation of minimum reinforcement needs not be satisfied. Freyermuth and Aalami [9] urged the use of reserve flexural strength in their research and revealed that the reserve flexural strength ( $P_y/P_{cr}$ ) can be used in any section as an alternative for the equation of minimum reinforcement. Also, Elrakib [7] in his research used the reserve strength in evaluating minimum tension reinforcement and suggested reducing the limit specified by the ACI 363R [10] of  $\rho_{min}$  by (25%) without any harmful effect on the behavior of beams in flexure depending on test results of his work.

The results of the tested beams with different concrete compressive strength indicated that the reserve flexural strength ( $P_y/P_{cr}$ ) required by the ACI 318M-14 [3] code satisfied for NSC and HSC beams reinforced with tension-steel equal to minimum flexural reinforcement. The factor (0.6) of minimum steel reinforcement of ACI code is adequate for the HSC beam (B80-S0.5) as illustrated in Table 3. Depending on the above, the value of reserve flexural strength ( $P_y/P_{cr}$ ) can be used to suggest a new formula for calculating minimum flexural steel reinforcement for HSC rectangular beams.

Based on (30) test results which have been taken from the literature; Besco et al [2], Elrakib [7], and Wafa and Ashour [11] included the new test results with a range of the compressive strength of concrete for the existing database variables between (59-90) MPa and the difference amount of steel

close to minimum flexural reinforcement and variable yield stress. The proposed formula suggests reducing the amount of reinforcement provided by the equation of ACI 318M-14 [3] code by the factor (0.8) as indicated in equation (2).

$$A_{s_{min, prop.}} = (0.8) \frac{\sqrt{f'_c}}{4f_y} b_w d \quad (2)$$

Seguirant et al [12] suggested reserve strength at nominal ( $P_n/P_{cr}$ ) at least (1.46) for evaluating minimum flexural reinforcement for rectangular beams, therefore, a comparison conducted for the beams satisfied the proposed formula and the results shows that all the reserve strength values at a nominal capacity of these beams are greater than the recommended value.

The proposed formula strength of concrete, satisfying the reserve strength at yielding of steel reinforcement ( $P_y/P_{cr}$ ) and the reserve strength at nominal capacity ( $P_n/P_{cr}$ ).

## 5. Conclusions

1. For the beams, without reinforcement, the compressive strength of concrete has a significant influence on the first cracking load, and with increasing compressive strength the first cracking load increased.
2. Tension steel reinforcement has a significant effect on increasing the first cracking load for beams with NSC. For HSC with low tension steel reinforcement, the increase in cracking load is less significant.
3. The increase in yielding strength of tension steel reinforcement increased the cracking load.
4. The increase in flexural reinforcement has more effect on the nominal load and less effect on the cracking load for the same concrete compressive strength, and increased the reserve strength and supplies better control on crack propagation.
5. The number of cracks increased as the tension reinforcement increased. With increased concrete strength, the cracks grew rapidly after their formation for HSC beams.
6. The increase in the compressive strength of concrete increases the cracking load and the nominal load, but the nominal load increment decreases.
7. The reserve flexural strength increases with increasing flexural reinforcement for the same type of concrete strength.
8. The reserve flexural strength increases with the increase in concrete compressive strength.
9. The increase in the nominal load of the beam depends on the concrete compressive strength and the amount of tension steel reinforcement.
10. The results show that the equation of ACI 318M-14 [3] code gives adequate minimum flexural reinforcement for NSC and conservative for HSC up to (83 MPa).
11. A new formula is proposed for HSC rectangular beams up to (90 MPa) concrete compressive strength by reducing the equation of ACI 318M-14 [3] code for minimum flexural reinforcement by the factor (0.8).

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