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Preface

Dear Participants,

A year preparation for The 1st International Conference on Sustainable Civil Engineering Structures and Construction Materials has already been elapsed. Announcement for abstracts and full papers submission has also been delivered through electronic mail to many related domestic and foreign universities as well as public / private institutions in Europe and Asian Countries since the beginning of this year. Up to the closing date of the abstract submission, more than 63 abstracts have been submitted from the USA, the UK, the Germany, the Switzerland, the Japan, the Taiwan, the Singapore, the Thailand, the Pakistan and the Iran.

But unfortunately, for many reasons not all participants submitting their abstracts proceeded with their full papers on the appointed date. For this reason the organizing committee decided to extend the period of the submission. As the number of the submitted full papers was less than we expected, not all reviewers were involved in the review process. The organizing comittee realizes that providing a good quality of papers is not just a matter of writing itself, but a comprehensive knowledge. Therefore, the organizing committee does understand to prospective participants whom are not able to finish writing on the scheduled date. The organizing committee still think that a number of 44 full papers will be reasonably enough to host this International Conference for the first time. The organizing committee do believes that in the next International Conference on SCESCM, the adequate time for preparing full papers can be accommodated by the organizing committee for involvement of more participants.

As the chairperson of scientific committee, anyway, would like to thank for the generousity and help of the reviewers who have reviewed and delivered their comments. I also would like to say sorry to all reviewers for any inconvenience that may happen during this review process.

Prof. Priyosulistyo

Chairperson of Scientific Committee The 1st International Conference on SCESCM

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Table of Contents

Material Engineering

Construction Management

Nonlinear Analysis of Reinforced Concrete Beams using FEM with Smeared Crack Approach, Mohr's Failure Criteria, and The Tomaszewicz Model

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Abstract: Cracking has a predominant effect on the failure mode of reinforced concrete beams. Their presence results a nonlinear stiffness behavior. Previous research work explained the concrete cracking process due to the exceeding of the major tensile principal stress to the concrete tensile strength only. The effect of the minor principal stress is neglected. This incorrectness results a less precise prediction of crack propagation and beam failure. Hence, an advanced analysis predicting the occurrence of cracks and taking into account the presence of biaxial principal stresses, is developed. The nonlinear finite element analysis is used to calculate stress and strain. To model steel reinforcement and the concrete cracking, the smeared element and smeared crack approach are chosen. The material behavior is consider nonlinear, while the bond slip effect and dowel action is neglected. The existence of cracks as a result of the combination of normal and shear stress is predicted based on the Mohr's failure criteria. The uniaxial concrete stiffness-modulus, not constant under incremental loading, is generated by the Tomaszewicz model. The load–displacement curves of analyzed beams are validated with identical laboratory tested specimens.

Keywords: Reinforced concrete beam, finite element analysis, smeared crack, smeared element, Mohr's failure criteria.

1 INTRODUCTION

The design of reinforced concrete structures is currently based on a limit state design concept in which structures are designed to undergo large deformation before their ultimate failure. Allowing structures to experience large deformation before their ultimate failure means the design is based on the performance of structures. Nowadays, it is more reasonable than strength based design due to the expense, especially for huge structures. Hence, it is notable to discover the behavior of reinforced concrete structures at their ultimate state and how the structures fail.

Generally, nonlinear behavior and failure in reinforced concrete beams is initiated from cracks in the concrete. After the first crack, the stiffness of reinforced concrete beams decreases gradually, causing inaccuracy in predicting the failure of the beams. Previous research work (*Subranto, 2007*) explained the cracks in concrete existed due to the exceeding of the major tensile principal stress to the concrete tensile strength. The role of the minor principal stress in the occurrence of cracks was neglected. In order to consider the effect of minor principal stress or the presence of biaxial principal stresses, an advanced analysis is developed.

The nonlinear finite element analysis is used to calculate stress and strain. In this analysis, quadrilateral elements with four gaussian points are utilized to model the beams. The steel reinforcement is modeled using smeared element approach. Cracks are assumed to be distributed evenly in a quadrilateral element thus this approach is called smeared crack. The Tomaszewicz model establishes the used concrete uniaxial stiffness modulus which is not constant under gradual loading. Mohr's failure criteria are used to predict the existence of cracks as a result of combination of normal and shear stress. During the analysis, self weight of concrete, bond slip between concrete and reinforcement, and dowel action of reinforcement are omitted.

2 SMEARED CRACK AND SMEARED ELEMENT APPROACH

2.1 Smeared Crack Approach

The smeared crack approach assumes cracks spread evenly on concrete elements that have failed while the direction of cracks is perpendicular to the direction of tensile principal stress. The advantages of this approach are cracks can occur freely in all elements when crack requirement is achieved, and the model using this approach should not be remeshed to be new elements to represent the cracked concrete. The concrete element which has been cracked is still assumed as an element that has considerable small stiffness. Analysis that uses smeared crack approach can easily include complicated concrete properties such as cracking and crushing (*Jendele, 2001*).

2.2 Smeared Element Approach

While smeared crack approach is functioned to model the cracks, smeared element is intended to take into account the steel reinforcement which also affects structure stiffness. The role of steel reinforcement is applied to a concrete element by adding steel reinforcement stiffness to concrete element stiffness (*Subranto, 2007*). Thus, concrete element which contains steel reinforcement will have new stiffness which consists of steel reinforcement stiffness and concrete stiffness. It is not necessary to add special elements to model the reinforcement, in other words, it will be convenient to model steel reinforcement behavior during nonlinear analysis.

3 NONLINEAR BEHAVIOR

3.1 Concrete

At the beginning of loading, concrete still acts as elastic material. It means concrete, in this condition, can be assumed as isotropic material which properties are identical in all directions. This assumption is appropriate unless stress and strain increase so much and cracks occur on concrete. The stiffness matrix of isotropic concrete is expressed as:

$$\mathbf{D_{ci}} = \frac{E_c}{1-v^2}\begin{bmatrix} 1 & v & 0 \\ v & 1 & 0 \\ 0 & 0 & \left(\frac{1-v}{2}\right) \end{bmatrix} \tag{1}$$

where $\mathbf{D_{ci}}$ is stiffness matrix of isotropic concrete, E_c is elastic modulus of concrete and v is Poisson's ratio of concrete.

At transition stage between elastic stage and cracking stage, stiffness modulus of concrete is different than when it is in elastic stage. While the formation of crack has not occurred in concrete, biaxial stresses are supposed to work in direction parallel to global axis x and global axis y, respectively. In this transition stage, the stiffness modulus of each global axis decreases as strain in the same axis increases. The decreasing stiffness modulus will be established using a model of uniaxial stress-strain relation in normal and high strength concrete named Tomaszewicz model. If the increase of strain in x-axis and increase of strain in y-axis is not equal, concrete should be considered as orthotropic material. The stiffness matrix of orthotropic concrete before cracks occur is shown below (*Kwak and Filippou, 1990*):

$$\mathbf{D_{co}} = \frac{1}{1-v^2}\begin{bmatrix} E_x & v\sqrt{E_x E_y} & 0 \\ v\sqrt{E_x E_y} & E_y & 0 \\ 0 & 0 & (1-v^2)G \end{bmatrix} \tag{2}$$

$$(1-v^2)G = \frac{1}{4}\left(E_x + E_y - 2v\sqrt{E_x E_y}\right) \tag{3}$$

where $\mathbf{D_{co}}$ is stiffness matrix of orthotropic concrete before cracks exist, v is Poisson's ratio of concrete, E_x is stiffness modulus in direction of global axis x, E_y is stiffness modulus in direction of global axis y, and G is shear modulus that is expressed in Equation (3).

Cracked concrete is considered absolutely as orthotropic material. Crack direction is perpendicular to major tensile principal stress which works when crack occurs. If local axis 1 is parallel to the major tensile principal stress and local axis 2 is perpendicular to local axis 1 and parallel to the minor principal stress then Equation (2) is rewritten as below to express stiffness matrix in direction of local axis:

$$\mathbf{D_{col}} = \frac{1}{1-v^2}\begin{bmatrix} E_1 & v\sqrt{E_1 E_2} & 0 \\ v\sqrt{E_1 E_2} & E_2 & 0 \\ 0 & 0 & (1-v^2)G \end{bmatrix} \tag{4}$$

$$(1-v^2)G = \frac{1}{4}\left(E_1 + E_2 - 2v\sqrt{E_1 E_2}\right) \tag{5}$$

where $\mathbf{D_{col}}$ is stiffness matrix of orthotropic concrete in direction of local axis, v is Poisson's ratio of concrete, E_1 is stiffness modulus in direction of local axis 1, E_2 is stiffness modulus in direction of local axis 2, and G is shear modulus that is expressed particularly in Equation (5).

In a cracked concrete element, stiffness modulus at a direction in which working stress is tension is almost zero. At the same time, Poisson's ratio of that element is reduced to zero due to the lack of Poisson's effect. However, stiffness modulus at direction of compression stress in the same element is not zero unless the cracked concrete element fails under compression at the direction of compression stress. Hence based on Equation (4), the stiffness matrix of a cracked concrete element is written:

$$\mathbf{D_{coc}} = \begin{bmatrix} 0 & 0 & 0 \\ 0 & E_2 & 0 \\ 0 & 0 & \lambda G \end{bmatrix} \tag{6}$$

where $\mathbf{D_{coc}}$ is stiffness matrix of cracked concrete element and λ is cracked shear constant which is vary from 0 to 1.0 and aimed to take into account the influence of shear of cracked concrete element (*Kwak and Filippou, 1990*).

Compressive strength of a cracked concrete element in direction of compression stress is less than uniaxial compressive strength of concrete due to the occurrence of compressive softening effect. This effect is coming up while cracks are widening. To accommodate this effect, β reduction factor of uniaxial compressive strength f_c' is used (*Vecchio & Collins, 1993*).

$$\beta = \frac{1}{0.85 - 0.27\left(\frac{\varepsilon_1}{\varepsilon_2}\right)} \tag{7}$$

if f_c' is greater than 50 MPa then

$$\beta = \frac{1}{0.8 + 0.6(1000\,\varepsilon_1 + 0.2)^{0.39}} \tag{8}$$

where β is reduction factor of uniaxial compressive strength f_c', ε_1 is strain in direction of local axis 1, and ε_2 is strain in direction of local axis 2.

3.2 Steel Reinforcement

The stiffness matrix of steel reinforcement can be written (*Subranto, 2007*):

$$\mathbf{D_s} = \begin{bmatrix} \rho_x E_{sx} & 0 & 0 \\ 0 & \rho_y E_{sy} & 0 \\ 0 & 0 & 0 \end{bmatrix} \tag{9}$$

where $\mathbf{D_s}$ is stiffness matrix of steel reinforcement, ρ_x is steel ratio in x-axis direction, ρ_y is steel ratio in y-axis direction, E_{sx} is elastic modulus of steel in x-axis direction, and E_{sy} is elastic modulus of steel in y-axis direction.

The stress-strain relation of steel reinforcement can be assumed linear unless the working stress surpasses yield strength of steel. Stiffness modulus of steel approaches zero after the stress exceeds the yield strength due to the increase of stress is tremendously small while the increase of strain is very large. Besides, the strain hardening effect of steel is neglected. Thus E_{sx} and E_{sy} in Equation (9) is set to zero if the yield strength has been achieved.

The role of steel reinforcement is applied to a concrete element by combining its stiffness with stiffness of the concrete element. This stiffness of new element which consists of the stiffness of steel reinforcement and stiffness of concrete can be written:

$$\mathbf{D_{cs}} = \mathbf{D_c} + \mathbf{D_s} \tag{10}$$

where $\mathbf{D_{cs}}$ is stiffness matrix of a new element in which stiffness of steel and concrete is combined, and $\mathbf{D_c}$ is stiffness matrix of concrete that is explained in Equation (1), (2), (4), and (6).

4 THE APPROACH OF TOMASZEWICZ MODEL

The Tomaszewicz model which can express stress-strain relation of normal and high strength concrete is used to establish the stiffness modulus of concrete that is not constant under incremental loading. The model is expressed below (*Dahl, 1992*):

$$\sigma = f_c \frac{\varepsilon}{\varepsilon_p} \times \frac{n}{n - 1 + \left(\frac{\varepsilon}{\varepsilon_p}\right)^{kn}} \tag{11}$$

where

$$\varepsilon_p = \frac{0.7}{1000} f_c^{\,0.31}$$

$$E_p = \frac{f_c}{\varepsilon_p}$$

$$E_{co} = 10{,}000 f_c^{\,0.3}$$

$$n = \frac{E_{co}}{E_{co} - E_p}$$

$$k = \frac{f_c}{20} \quad \text{for} \quad \varepsilon > \varepsilon_p$$

$$k = 1.0 \quad \text{for} \quad \varepsilon \le \varepsilon_p$$

σ is stress, ε is strain, f_c is compressive strength of concrete, ε_p is strain at stress equal to f_c, E_{co} is elastic modulus of concrete, and E_p is secant modulus at stress equal to f_c.

The simplified Tomaszewicz model is generated by modifying the original curve to be four linear lines that have tangent modulus E_A, E_B, E_C, and E_D, respectively. As an example for using these tangent moduli as stiffness moduli, if the current strain is between $0.5\varepsilon_p$ and $0.7\varepsilon_p$ then the used stiffness modulus in the same direction of the strain is E_C. Stiffness modulus E_E, which is used if the strain larger than ε_p, is set to approach zero (1% of E_D) in order to prevent numerical error.

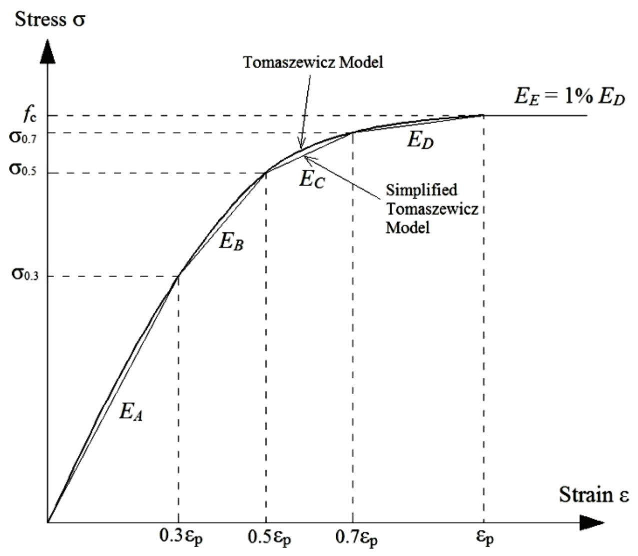

Figure 1. The simplified Tomaszewicz model.

Stiffness modulus E_A is also used if the strain is tensile strain and the tensile stress still not exceeds tensile strength of concrete. The tensile strength is written (*Aoyama, 2001*):

$$f_r = 0.3\sqrt{f_c} \tag{12}$$

where f_r is tensile strength of concrete.

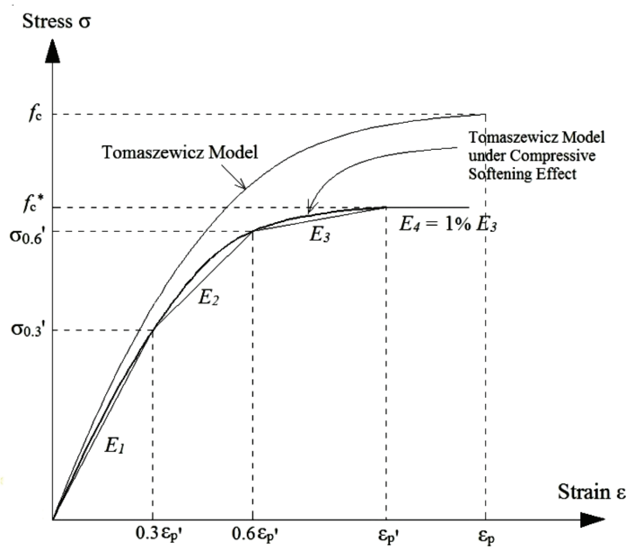

Figure 2. The adjustment of the simplified Tomaszewicz model to consider compressive softening effect.

When cracks exist in a concrete element, stiffness modulus in direction of major tensile principal stress is set to zero whereas stiffness modulus in direction of minor principal stress, when this is compressive stress, is determined using the simplified Tomaszewicz model with an adjustment in order to consider the compressive softening effect. As a result of this effect, compressive strength in direction of minor compressive principal stress is reduced to f_c* which equals βf_c. In the same time, ε_p is also diminished to ε_p' that is equal to $\beta\varepsilon_p$. As shown in Figure 2, to simplify the new curve, tangent modulus E_1, E_2, and E_3 are used as stiffness modulus at their respective strain ranges. The establishment of E_4 that is equal to 1% of E_3 has the same purpose as the establishment of E_E.

5 THE APPROACH OF MOHR'S FAILURE

The Mohr's failure criteria are applied as failure envelope in the Mohr's diagram. These criteria are represented by the envelope or curves in the Mohr's diagram that describes the critical state of stresses over a range of differential stresses *(Fossen, 2010)*. Two exact critical states of stresses in concrete are acknowledged, these are uniaxial tensile and uniaxial compression failure. Based on these two critical cases, the Mohr's failure envelope can be drawn.

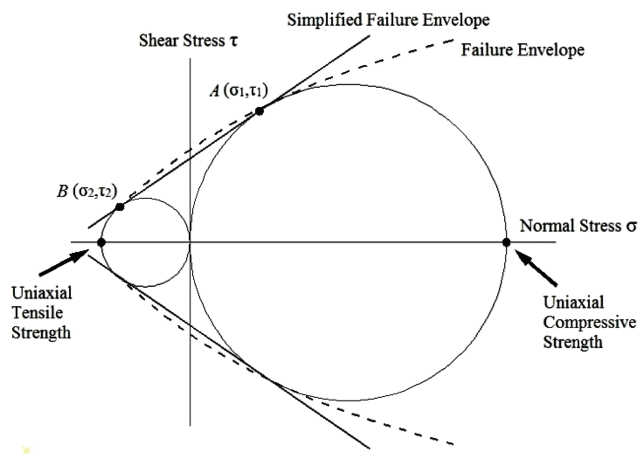

Figure 3. Mohr's failure envelope

Although research has proven that the envelope is not exactly linear, in the analysis it is simplified to a linear line (Figure 3). Each state indicating an uniaxial tensile or compression failure, is represented by an individual Mohr's circle. If a tangent point at the Mohr's circle in uniaxial compressive failure is *A* (σ_1,τ_1) and *B* (σ_2,τ_2) for uniaxial tension failure, the equation for the simplified envelope is written as:

$$\tau = \left(\frac{\tau_2 - \tau_1}{\sigma_2 - \sigma_1}\right)\sigma + \left(\tau_1 - \sigma_1\frac{\tau_2 - \tau_1}{\sigma_2 - \sigma_1}\right) \tag{13}$$

where τ is shear stress and σ is normal stress.

6 ANALYSIS AND DISCUSSION

The numerical model is verified by analyzing four beams which have been tested in physical experiments. The beams are beam 1-NWC (*Lim et al.,* *2011*), beam 4B2-0.50, 4B3-0.05, and 4B3-0.10 (*Jang et al., 2008*).

6.1 Beam 1-NWC

This beam has overall length 3100 mm, span length 2800 mm, width 150 mm, and effective height 257 mm. Its material properties are concrete compressive strength f_c 37.9 MPa, yield strength of steel D10 550 MPa, and yield strength of steel D16 512 MPa. The beam has double reinforcement that is tensile reinforcement 2D16 and compressive reinforcement 2D10. Closed stirrups D10-130 are used as shear reinforcement along its span. The beam, that is simply supported, is loaded at middle span with two point load system in which the distance between two point loads is 800 mm.

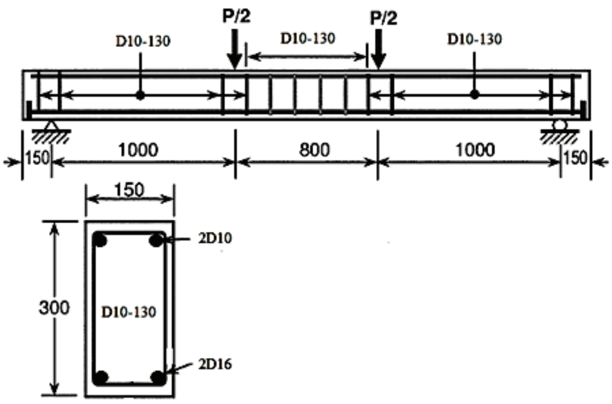

Figure 4. Reinforcement details of beam 1-NWC.

In the analysis, the applied load increment is 2000 N for each point load. The used cracked shear constant λ are 0.10 and 0.05. The applied elastic modulus E_A and Poisson's ratio are 28,694.77 MPa and 0.20, respectively. Elastic modulus of steel D16 and D10 are 183,000 MPa and 185,000 MPa, respectively.

According to the analysis, initial cracks come up when load level equals 12 kN and yield of tensile reinforcement occurs at 104 kN load level. Whereas, based on physical experiment, initial cracks occur when load level is equal to 15.1 kN and yield of tensile reinforcement appears at 100.3 kN load level. These show that the numerical model can predict initial cracks of beam 1-NWC and yield strength of its tensile reinforcement quite accurately.

After the extreme decrease of structure stiffness due to the yield of tensile reinforcement, the load-displacement relation depends on the used cracked shear constant. The most appropriate cracked shear constant for 1-NWC is 0.05. It can be inferred that the influence of shear on ultimate failure of beam 1-NWC is insignificant.

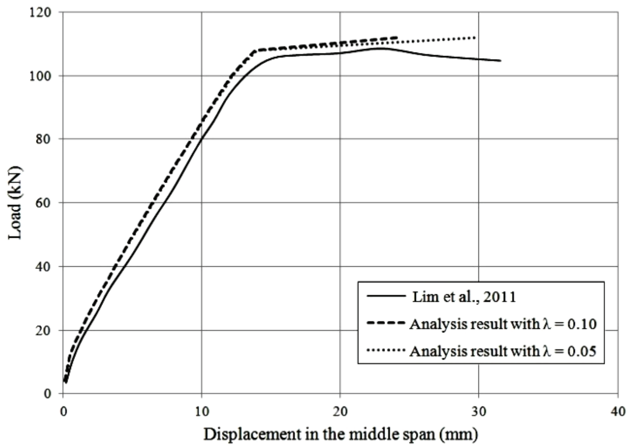

Figure 5. Load-displacement curves of beam 1-NWC.

The numerical model predicts the crack pattern of beam 1-NWC very well. As shown in Figure 6 and Figure 7, at ultimate failure, the height and direction of cracks and crushing of concrete in compression zone are estimated quite appropriately.

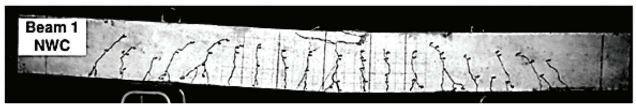

Figure 6. Actual crack pattern of beam 1-NWC.

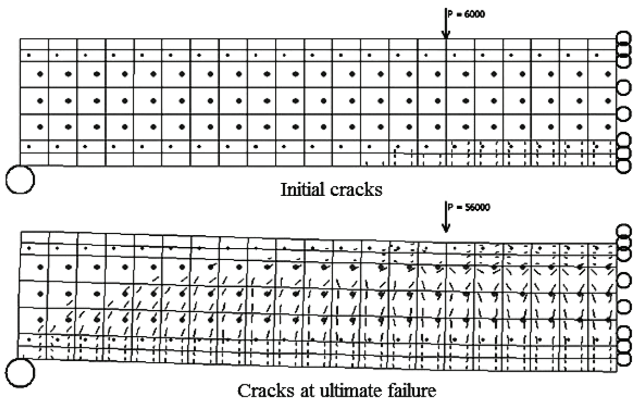

Figure 7. Predicted crack pattern of beam 1-NWC.

6.2 Beam 4B2-0.50

Geometrically, beam 4B2-0.50 has overall length 1770 mm, span length 1350 mm, width 140 mm, and effective height 210 mm. Material properties of the beam are concrete compressive strength f_c 40 MPa, yield strength of steel D19 435 MPa, and yield strength of steel D10 395 MPa. The beam has single reinforcement which is tensile reinforcement 2D19. Shear reinforcement along its span is D10-100. The beam, that is simply supported, is loaded at middle span with two point load system in which the distance between two point loads is 300 mm.

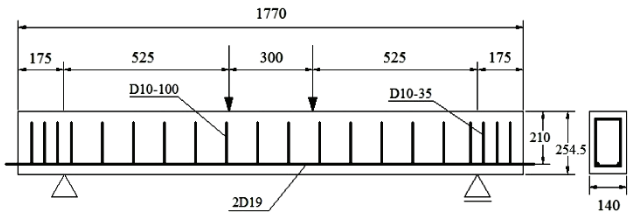

Figure 8. Reinforcement details of beam 4B2-0.50.

The applied load increment is 2000 N per point load. The used cracked shear constant λ are 0.10 and 0.50. The applied elastic modulus E_A and Poisson's ratio are 29,303.52 MPa and 0.20, respectively. Elastic modulus of steel D19 and D10 are 210,760 MPa and 192,350 MPa, respectively.

According to the physical experiment, initial cracks show up when load level is 24.2 kN and yield of tensile reinforcement occurs at 185.6 kN load level. While, based on the analysis, initial cracks occur when load level is 20 kN and yield of tensile reinforcement appears at 180 kN load level. Thus, the numerical model can approximate initial cracks of beam 4B2-0.50 and yield strength of its tensile reinforcement quite precisely.

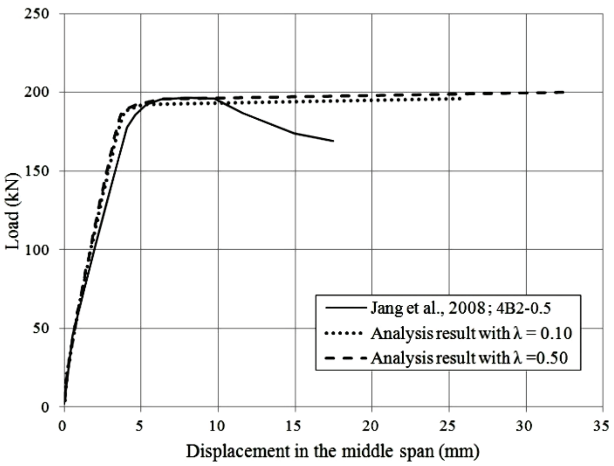

Figure 9. Load-displacement curves of beam 4B2-0.50.

The most fitting cracked shear constant for beam 4B2-0.50 is 0.50. It can be concluded that the influence of shear on ultimate failure of beam 4B2-0.50 is more significant than beam 1-NWC. It is because the shear deformation is more dominant in beam 4B2-0.50 since its shear span ratio (2.50) is less than shear span ratio of beam 1-NWC (5.45).

Figure 10. Actual crack pattern of beam 4B2-0.50.

Approximated crack pattern and actual crack pattern are similar enough. Figure 10 and Figure 11 show that, at ultimate failure, prediction of crushing in compression zone, diagonal flexure-shear cracks, and vertical flexural cracks is accurate enough.

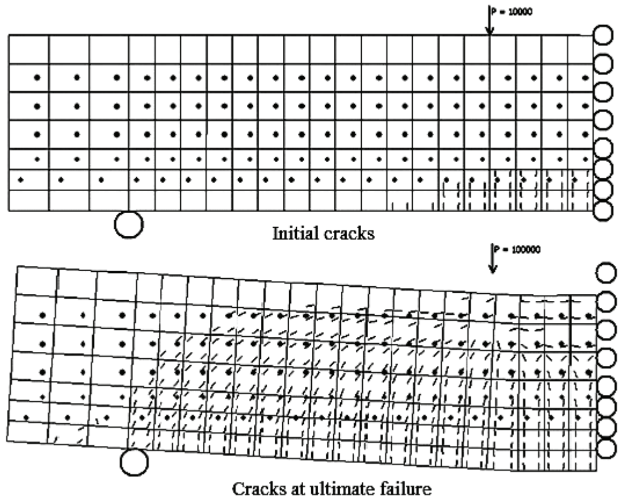

Figure 11. Predicted crack pattern of beam 4B2-0.50.

6.3 Beam 4B3-0.05 and Beam 4B3-0.10

Beam 4B3-0.05 and beam 4B3-0.10 have the same geometrical parameters and material properties which are overall length 2570 mm, span length 2220 mm, width 250 mm, effective height 300 mm, concrete compressive strength f_c 40 MPa, yield strength of steel D13 413 MPa, and yield strength of steel D10 395 MPa. Both beams have single reinforcement. Shear reinforcement along their spans is D10-70. The beam, that is simply supported, is loaded at middle span with two point load system in which the distance between two point loads is 300 mm. The difference of both beams is only on their tensile reinforcement where beam 4B3-0.05 has 2D10 as its tensile reinforcement and beam 4B3-0.10 has 2D13 as its tensile reinforcement.

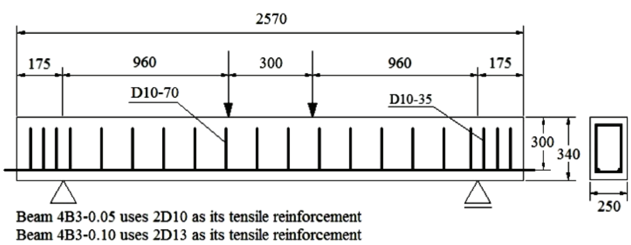

Figure 12. Reinforcement details of beam 4B3-0.05 and beam 4B3-0.10.

The applied load increment is 2000 N per point load. The cracked shear constant λ for both beams is set to 0.01. The applied elastic modulus E_A and Poisson's ratio are 29,303.52 MPa and 0.20, respectively.

Elastic modulus of steel D13 and D10 are 203,040 MPa and 192,350 MPa, respectively.

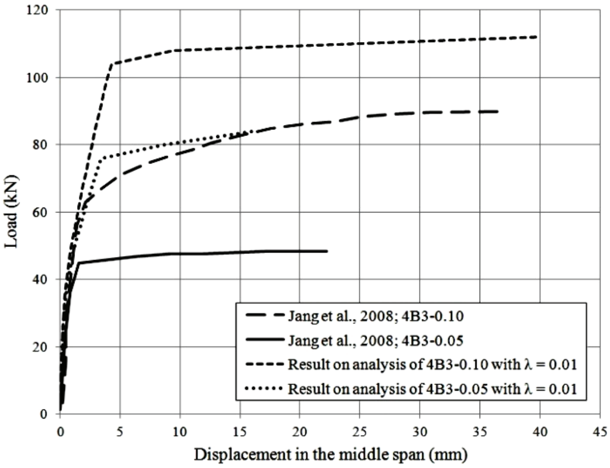

Figure 13. Load-displacement curves of beam 4B3-0.05 and beam 4B3-0.10.

The calculated load-displacement curve of beam 4B3-0.05 and 4B3-0.10 if compared to the load-displacement curve obtained from physical experiment shows distinct differences. Although the initial stiffness modulus of each beam is approximated accurately enough, the calculated ultimate load level of each beam is obviously deviated from its ultimate load level which is obtained from experiment.

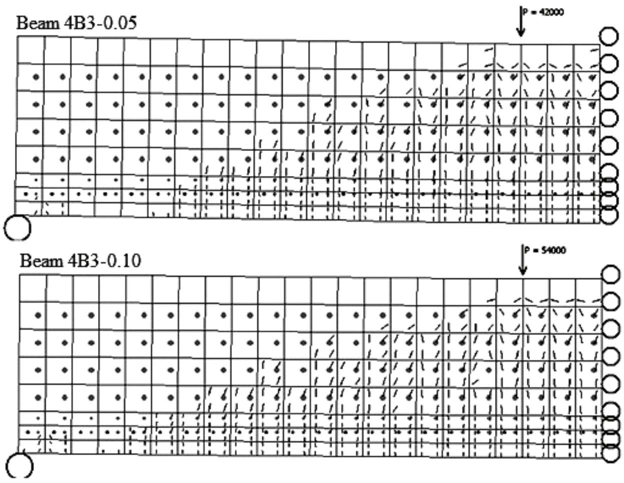

Figure 14. Predicted crack pattern of beam 4B3-0.05 and beam 4B3-0.10.

The predicted crack patterns also show dissimilarity in relation to actual crack patterns. As in Figure 15, some actual cracks are more concentrated in middle span zone instead of being distributed more evenly along span length (Figure 14). Due to the position of the actual cracks, they are categorized as flexural cracks. Obvious flexure-shear cracks do not exist in the actual crack pattern.

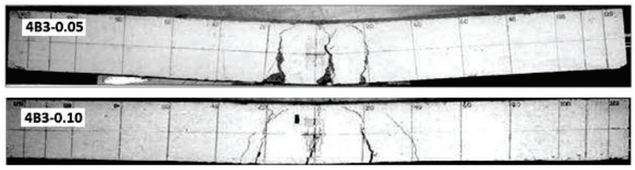

Figure 15. Actual crack pattern of beam 4B3-0.05 and beam 4B3-0.10.

The analysis calculates ultimate load of beam 4B3-0.05 and beam 4B3-0.10 that is significantly larger than ultimate load obtained from experiment. As information, ratio of steel ratio and balanced steel ratio (ρ/ρ_b) of beam 1-NWC, 4B2-0.50, 4B3-0.05, and 4B3-0.10 are 0.37, 0.562, 0.05, and 0.01, respectively. It is found that when ρ/ρ_b is quite small, the numerical model shows obvious different results if compared with results obtained from experiment. In the analysis, the reinforcement is considered distributed uniformly in a two dimensional concrete element in longitudinal direction as well as transversal direction. In fact, if the case is beam has considerable small ratio of ρ/ρ_b, the reinforcement cannot be assumed that it is spread evenly in transversal direction. This incorrectness leads the numerical model to show mistaken results.

7 CONCLUSIONS

The numerical model is able to predict load-displacement curve and crack pattern of reinforced concrete beams quite accurately although it will show obvious different results if ratio of steel ratio and balanced steel ratio (ρ/ρ_b) is very small,. It is due to the incorrect assumption applied to a two dimensional concrete element in which the reinforcement is considered distributed evenly in transversal direction that, in fact, is mistaken.

The cracked shear constant is applied to a cracked concrete element because the influence of shear and shear transfer within crack gap cannot be absolutely neglected. It is proved in this research work. The increase of appropriate cracked shear constant will occur when the shear deformation is significant. It happens when the shear span ratio of reinforced concrete beams is less or equal to 2.5 as shown in beam 1-NWC and beam 4B3-0.50 analysis.

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