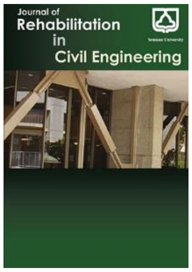
Journal of Rehabilitation in Civil Engineering 4-2 (2016) 79-92



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A New Approach for Numerical Analysis of the RC Shear Walls Based on Timoshenko Beam Theory Combined with Bar-Concrete Interaction

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ARTICLE INFO

Article history:
Received: 04 February 2017
Accepted: 03 April 2017

Keywords:
Nonlinear analysis,
Timoshenko beam theory,
Bar-Concrete interaction,
Shear deformation,
Reinforced concrete shear wall.

ABSTRACT

In this paper, a new approach for nonlinear numerical modelling of the reinforced concrete shear walls with consideration of bar-concrete interaction and shear deformation is proposed. Bar and concrete stress-strain relations, the bar-concrete interaction, the shear stress-strain relation and, also, their cyclic behavior including the strength degradation and stiffness degradation are adopted as known specifications. In the modeling, shear wall is divided into two types of joint and reinforced concrete (RC) elements. In the RC element, the effect of shear deformation is considered based on Timoshenko beam theory. Separate degrees of freedom are used for the steel bars and concrete part. The effect of bar-concrete interaction has been considered in the formulation of the RC element. The reliability of the method has been assessed through the comparison of numerical and experimental results for a variety of tested specimens under cyclic and pushover loading. A good agreement between experimental and analytical results is obtained for both cases of strength and stiffness during the analysis.

1. Introduction

Many analytical models have been devised for nonlinear analysis of reinforced concrete shear wall. One of the first methods is equivalent beam-column element. In this model, the wall is replaced by a column with equivalent cross section properties. The limitation of this approach is that assumes rotations occur around the central axis of the wall. Therefore, it ignores changes in neutral axis of the wall section and interactions.

Also, due to neglecting the progressive opening of the cracks associated with shifting of the neutral axis, rotations and displacements in this method are less than reality. These limitations led to use multiple elements method of the vertical component [1]. This procedure expresses more accurately the tensile hardening of concrete, progressive opening and closing of the cracks and shear nonlinear behavior of concrete in addition to removing restrictions of the shifting neutral axis. In order to consider the flexure-shear interaction in the reinforced concrete structural walls, a new model was presented by Massone and Wallace [2]. This model is overestimated for the flexural deformations and low estimated for the shear deformations. The other macroscopic modeling approaches that can be mentioned as two-component beam-column element, one-component beam-column element, multiple spring model, multiple-vertical-line-element and multi-axial spring model [3]. One of the most promising models for the nonlinear analysis of reinforced concrete elements is, presently, fiber theory model. This model, basically, neglects the shear deformations and adopts the perfect bond assumption between the bars and surrounding concrete. This assumption causes a considerable difference between experimental and analytical responses [4]. Mullapudi et al. [5] have used Timoshenko beam theory in fiber theory formulation in order to evaluate the behaviour of shear-dominated thin-walled RC[1] members with the restriction of perfect bond assumption between bars and concrete. Stramandinoli and La Rovere [6] have used the finite element model and studied the difference between Timoshenko-beam and Euler-Bernoulli beam theories in nonlinear analysis of reinforced concrete beams. Their research has concluded that in beams with dominant flexural behavior, Euler-Bernoulli theory in efficient and while the effects of inclined cracks caused by shear are important, Timoshenko beam model is appropriate to investigate element behavior. In the other hand, many studies have been done to date in the field of bar concrete interaction. Monti and Spacone [7] considered the bond slip effect of the bars in the fiber section theory. This model was used by Kotronis et al. [8] to simulate nonlinear behaviour of reinforced concrete walls subjected to earthquake ground motion. Belmouden and Lestuzzi [9] have used this model to predict nonlinear cyclic behaviour of reinforced concrete shear walls based on the tests conducted. Orakcal and Chowdhury [10] have studied bond slip effect in the reinforced concrete elements under cyclic loading by extension of multiple vertical line element model in accordance with fiber theory. Many of studies in this field acknowledge that bar-concrete interaction effect is significant and should be considered if an accurate model is expected [11].

Between the proposed numerical macro modelling methods for simulation of reinforced concrete shear walls, one or more limitations including linear shear deformation assumption, neglecting the shear deformation, complexity of the boundary conditions simulation, and ignoring the bond slip effect, show itself and is noteworthy. In this paper, a numerical model based on the fiber method is proposed for nonlinear analysis of reinforced concrete shear wall. The theory of the method is similar to fiber method but the perfect bond assumption between the reinforcing bars and surrounded concrete is removed. Separate degrees of

[1] Reinforced Concrete

freedom are used for the steel and concrete parts in nonlinear modelling of the reinforced concrete elements. Also the nonlinear shear deformation is considered in the formulations based on Timoshenko beam theory in combination with fiber theory [12].

2. Nonlinear modelling of RC shear wall with bar-concrete interaction

In the fiber theory, each member is divided longitudinally into several segments, and each segment is combined of parallel layers. Some layers would represent the concrete material and other layers would represent the steel material. Behaviour of concrete and steel are separately defined without consideration of interaction between them. In this research, in order to numerical modeling of shear walls, two types of element have been used as Figure1. One of them is the element basically used for modeling the body of shear wall and the other one is connection element applied for footing. In the method used on the basis of layered model, the perfect bond assumption between concrete and bars is removed and the possible effects of slip have been considered. In the joint element the effect of pull-out can be considered as the relative displacement between the steel bar and surrounding concrete and bond stress is referred to as the shear stress acting parallel to an embedded steel bar on the contact surface between the reinforcing bar and concrete. The number of degrees of freedom in the side of the joint element is compatible with the degrees of freedom at the ends of the wall element adjacent to the joint element. Although it is feasible to model the pull-out effects, the embedded length of steel bars has been considered sufficiently large to prevent interference of bar's pull-out from the foundation in the results of this research [13].

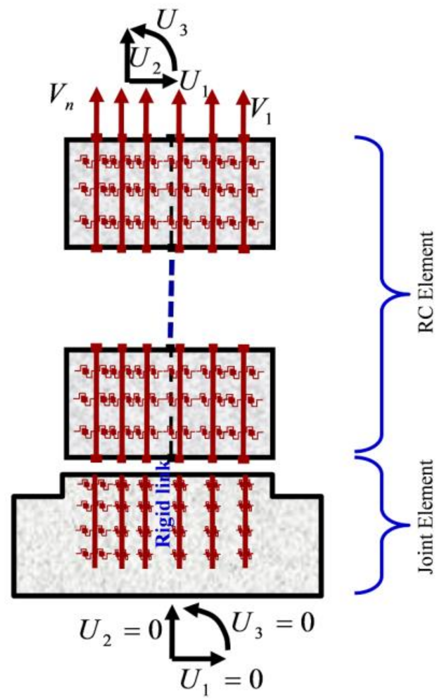

Figure 1. Numerical modeling of the reinforced concrete shear wall

Based on research carried out by Limkatanyu and Spacone [14] and Hashemi and Vaghefi [12], and by removing Euler-Bernoulli beam theory and replacing it with Timoshenko beam theory, the formulations has been rewritten. The slip effect between bars and surrounding concrete is considered without ignoring the compatibility of the strain between the concrete and bars. Timoshenko beam theory, assumes that the cross section remains plane and is not necessarily perpendicular to the longitudinal axis after deformation, but Euler-Bernoulli beam theory neglects shear deformations by assuming that, plane sections remain plane and perpendicular to the longitudinal axis during bending. In Figure2 the comparison between Euler-Bernoulli and Timoshenko beam theory is presented [15].

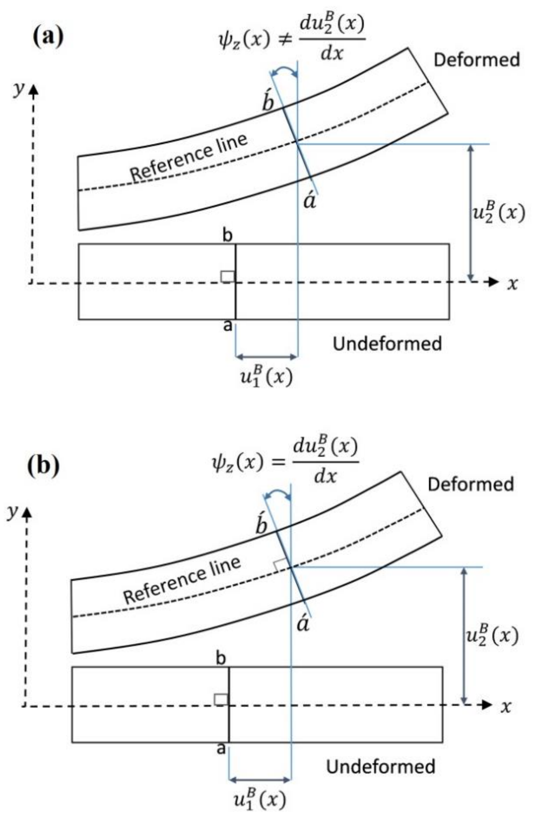

Figure 2. Comparison between (a) Timoshenko beam theory and (b) Euler-Bernoulli beam theory

In uniaxial bending conditions, stress values at any position of the cross section related to x position along the element can be calculated as Equations (1) and (2).

$$\sigma_{xx}(x,y) = -Ey\frac{d\psi_z(x)}{dx} \tag{1}$$

$$\sigma_{xy}(x,y) = G\left(\frac{\partial u_x(x,y)}{\partial y} + \frac{\partial u_y(x,y)}{\partial x}\right) \tag{2}$$

According to the Timoshenko beam theory Equation (2) can be rewritten as:

$$\sigma_{xy}(x,y) = G\left(-\psi_z(x) + \frac{d\,u_2^B(x)}{\partial x}\right) \tag{3}$$

In the recent Equations, $\psi_z(x)$ is rotational deformation and $u_2^B(x)$ is transversal displacement of the concrete element. σ_{xx}, σ_{xy} are defined as longitudinal and shear stresses in the section. E, G, u_x, u_y are defined as modulus of elasticity, shear modulus, longitudinal displacement, and transversal displacement, respectively. Bending moment about the z axis and shear force in y direction can be written as Equations (4) and (5), respectively.

$$M_z(x) = -\int_A y\,\sigma_{xx}dA = E\frac{d\psi_z(x)}{dx}\int_A y^2 dA = EI_z\frac{d\psi_z(x)}{dx} \tag{4}$$

$$V_y(x) = -\int_A y\sigma_{xy}dA = -\beta_y G_{shear}(-\psi_z(x) + \frac{du_2^B(x)}{dx})\int_A dA = -\beta_y G(-\psi_z(x) + \frac{du_2^B(x)}{dx})A = -\beta_y\sigma_{xy}A \tag{5}$$

In the Equations (4) and (5), β_y and I_z are shear correction factor and moment of inertia of the section, respectively.

According to the relation $\sigma_{xy} = G\gamma_y$ that shows shear stress-strain relation in y direction, from Equation (5) it can be concluded that:

$$-\frac{V_y(x)}{\beta_y GA} = \frac{du_2^B(x)}{dx} - \psi(x) \Rightarrow \quad \gamma_y = \psi_z(x) - \frac{du_2^B(x)}{dx} \tag{6}$$

Therefore, the difference between $\frac{du_2^B(x)}{dx}$ and $\psi_z(x)$ values in the section, will result the shear strain in the y direction which is neglected in the Euler-Bernoulli beam theory. More details about employed shear correction factor is presented in [16].

A length segment of an RC element is considered as a combination of a length segment of a 2-node concrete element and n number of steel bar elements (Figure3). 2-node concrete element follows the Timoshenko beam theory in order to consider both cases of shear and flexural deformations. 2-node bar elements are in fact truss elements with axial degrees of freedom. The effect of bond force between the concrete and each longitudinal bar is taken into account [11, 12].

Bar's slippage is allowed to occur, because the nodal degrees of freedom of the concrete element and that of the bars are different. Based on small deformation assumptions, all equilibrium conditions are considered. Considering axial equilibrium in the concrete element and steel bars, as well as the transversal and moment equilibriums in the segment dx, leads to a matrix form of equations given by Equation (7).

$$\partial_B^T \mathbf{D}_B(x) - \partial_b^T \mathbf{D}_b(x) - \mathbf{p}(x) = 0 \qquad (7)$$

Where: $\mathbf{D}_B(x) = \{\overline{\mathbf{D}}(x) : \overline{\overline{\mathbf{D}}}(x)\}^T$ is the vector of RC element section forces. $\overline{\mathbf{D}}(x) = \{N(x) \quad V_y(x) \quad M_y(x)\}^T$ is the vector of concrete element section forces. $\overline{\overline{\mathbf{D}}}(x) = \{N_1(x) \quad \dots \quad N_n(x)\}^T$ is the vector of bar axial forces. This vector has n rows. $\mathbf{D}_b(x) = \{D_{b1}(x) \quad \dots \quad D_{bn}(x)\}^T$ is the vector of section bond forces. $\mathbf{P}(x) = \{0 \quad p_y \quad 0 \quad 0 \quad \dots \quad 0\}^T$ is the vector RC element force vector. n is the number of longitudinal bars in the cross section. p_y is the value of external load. ∂_B , ∂_b are differential operators and given in Equation (8).

$$\partial_B = \begin{bmatrix} \overline{\partial}_B & 0 \\ 0 & \overline{\overline{\partial}}_B \end{bmatrix}, \overline{\partial}_B = \begin{bmatrix} \frac{d}{dx} & 0 & 0 \\ 0 & \frac{d}{dx} & -1 \\ 0 & 0 & \frac{d}{dx} \end{bmatrix},$$

$$\overline{\overline{\partial}}_B = \begin{bmatrix} \frac{d}{dx} & 0 & \dots & 0 \\ 0 & \frac{d}{dx} & \dots & 0 \\ \dots & \dots & \dots & \dots \\ 0 & 0 & \dots & \frac{d}{dx} \end{bmatrix} \qquad (8)$$

$$\partial_b = \begin{bmatrix} -1 & 0 & y_1 & 1 & 0 & \dots & 0 \\ -1 & 0 & y_2 & 0 & 1 & \dots & 0 \\ \dots & \dots & \dots & \dots & \dots & \dots & 0 \\ -1 & 0 & y_n & 0 & 0 & \dots & 1 \end{bmatrix}_{n*(n+3)}$$

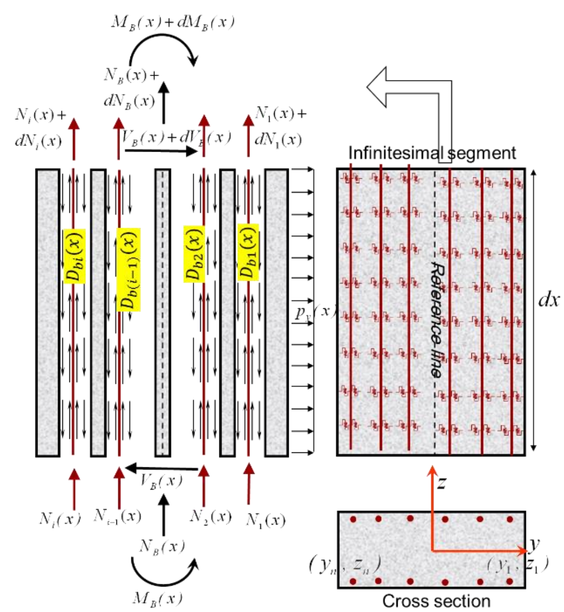

Figure 3. Free body diagram of infinitesimal segment of RC element

y_n is the distance of bar n from the section reference axis (Figure3). The RC element section deformation vector conjugate of

$\mathbf{D}_B(x)$ is $\mathbf{d}_B(x) = \left\{\bar{\mathbf{d}}(x) \vdots \bar{\bar{\mathbf{d}}}(x)\right\}^T$. In which $\bar{\mathbf{d}}(x) = \left\{\varepsilon_B(x) \quad \gamma_y(x) \quad \kappa_B(x)\right\}^T$ contains concrete element section deformations and $\bar{\bar{\mathbf{d}}}(x) = \{\varepsilon_1(x) \quad ... \quad \varepsilon_n(x)\}^T$ contains the axial strain of the bars. The displacement vector in the cross section of RC element is defined as $\mathbf{u}(x) = \left\{\bar{\mathbf{u}}(x) \vdots \bar{\bar{\mathbf{u}}}(x)\right\}^T$, in which $\bar{\mathbf{u}}(x) = \left\{u_B^1(x) \quad u_B^2(x) \quad \psi_z(x)\right\}^T$ contains concrete element axial, transversal and rotational displacements, respectively. $\bar{\bar{\mathbf{u}}}(x) = \{u_1(x) \quad ... \quad u_n(x)\}^T$ contains the axial displacements of the bars. From the small deformation assumption, the element deformations are related to the element displacements through the Equation (9).

$$\mathbf{d}_B(x) = \partial_B \mathbf{u}(x) \qquad (9)$$

The slip values of the bars in the section of RC element are determined by the following relation between the bar and concrete element displacements:

$$u_{bi}(x) = v_i(x) - u_1^B(x) + y_i\,\psi_z(x) \qquad (10)$$

Where, $v_i(x)$ is the bar axial displacement and $u_1^B(x)$ is the longitudinal displacement of concrete element. By introducing the bond deformation vector as $\mathbf{d}_b(x) = \{u_{b1}(x) \quad ... \quad u_{bn}(x)\}^T$, Equation (10) can be written in the following matrix form:

$$\mathbf{d}_b(x) = \partial_b \mathbf{u}(x) \qquad (11)$$

The weak form of displacement based finite element formulation is determined through the principle of stationary potential energy. The RC element nodal displacement (**U**), which is shown in Figure 4, serves as primary element unknowns and the section displacement **u**(x) are related to it through the displacement shape function matrix (**N**(x)). The relation between nodal displacements and internal deformations can be written through the transformation matrix as Equation (12).

$$\mathbf{d}_B(x) = \mathbf{B}_B(x)\mathbf{U},$$
$$\mathbf{d}_b(x) = \mathbf{B}_b(x)\mathbf{U},$$
$$\mathbf{B}_B(x) = \partial_B \mathbf{N}(x),$$
$$\mathbf{B}_b(x) = \partial_b \mathbf{N}(x) \qquad (12)$$

The nonlinear behaviour of RC element is derived from the nonlinear relation between the section forces ($\mathbf{D}_B(x), \mathbf{D}_b(x)$) and the section deformations ($\mathbf{d}_B(x), \mathbf{d}_b(x)$) through section and bond stiffness matrices ($\mathbf{K}_B(x)$, $\mathbf{K}_b(x)$). The section stiffness matrix included the axial, shear and bending stiffness of concrete element (*EA*(x), *GA*(x) and *EI*(x)) and also the axial stiffness of the bars ($E_n A_n(x)$). The bond stiffness matrix is diagonal and included the slope of the bond force-slip relationship of each bar ($k_{bn}(x)$). By using the fiber section method, the section stiffness matrix is derived. In this method, the stress-strain relationships of steel and concrete are needed. The bond stiffness matrix is derived through the bond stress-slip relation and perimeter of each bar. From finite element formulation, the stiffness matrix of RC element with the effect of bond-slip can be derived through the summation of two stiffness matrices and can be written in the form of Equation (13).

$$\mathbf{K}(x) = \mathbf{K}_B + \mathbf{K}_b = \int_L \mathbf{B}_B^T(x)\mathbf{k}_B(x)\mathbf{B}_B(x)dx + \int_L \mathbf{B}_b^T(x)\mathbf{k}_b(x)\mathbf{B}_b(x)dx \qquad (13)$$

The relationship between the external load vector, the internal resisting force vector and the nodal displacement vector in the nonlinear analysis algorithm can be written as Equation (14).

$$\mathbf{K}\Delta\mathbf{U} = \mathbf{P} - \int_L \mathbf{B}_B^T(x)\mathbf{D}_B(x)dx - \int_L \mathbf{B}_b^T(x)\mathbf{D}_b(x)dx = \mathbf{P} - \mathbf{Q} = \mathbf{P} - (\mathbf{Q}_B + \mathbf{Q}_b) \qquad (14)$$

Where $\mathbf{K}$ is the RC element stiffness matrix, $\mathbf{Q}$ is the resisting force vector of the element, $\mathbf{K}_B$ and $\mathbf{K}_b$ are the element and bond contributions to the stiffness matrix, respectively. Also, $\mathbf{Q}_B$ and $\mathbf{Q}_b$ are the element and bond contributions to the resisting force vector, respectively. At each load step of the nonlinear analysis, the resisting force vector of the section is driven according to existing deformations in each section of the element. Thereby, the resisting force vector of the element is derived by using numerical integration methods. The result of **P-Q** is the residual force vector and converges to a zero vector after some iteration at each load step.

A joint element is used as the footing connection of the RC shear wall. In this element the effect of pull-out is considered as the relative displacement between the steel bar and surrounding concrete and bond stress is referred to as the shear stress acting parallel to an embedded steel bar on the contact surface between reinforcing bar and concrete. Referring to Figure5, the slippage of the bars can be defined in the form of Equation (15), if the nodal displacement vector related to pull-out behaviour is defined as $\mathbf{U}_{PM} = \begin{bmatrix} U_1^1 & U_2^1 & U_3^1 & V_1^1 & \dots & V_n^1 \end{bmatrix}^T$.

$$\mathbf{slip} = \begin{bmatrix} d_{b_1} \\ \dots \\ d_{b_i} \\ \dots \\ d_{b_n} \end{bmatrix} = \begin{bmatrix} -1 & 0 & y_1 & 0 & \dots & 0 \\ -1 & 0 & y_2 & 0 & \dots & 0 \\ \dots & \dots & \dots & \dots & \dots & \dots \\ -1 & 0 & y_n & 0 & 0 & 1 \end{bmatrix}_{n*(n+3)} \times \mathbf{U}_{PM} \qquad (15)$$

In this equation, y_n is the distance of the n^{th} bar from the reference line. The relationship between the pull-out force and the slip of embedded bars derives from the bond stress-slip relationship, embedded length of the bar, conditions at the end of the bar and perimeter of the bar cross section. A computer program created in MATLAB software was used by the authors [17].

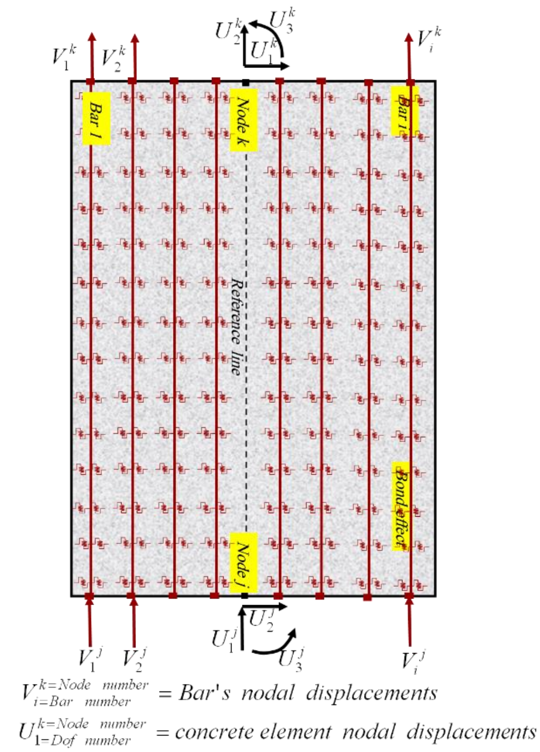

Figure 4. Reinforced concrete element

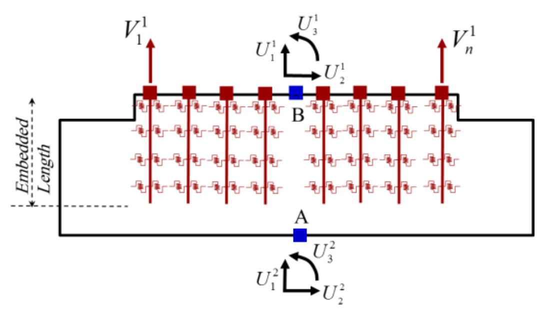

Figure 5. Numerical modelling of the joint element

3. Material behaviours

3.1. Concrete cyclic stress-strain relation

The monotonic envelope curve for confined concrete, introduced by Park et al. [18] and later extended by Scott et al. [19], is adopted for the compression region because of its simplicity and computational efficiency (Figure6). Also, it is assumed that concrete behaviour is linearly elastic in the tension region before the tensile strength and, beyond that, the tensile stress decreases linearly with increasing tensile strain. Ultimate state of tension behaviour is assumed to occur when tensile strain exceeds the value given in Equation (16).

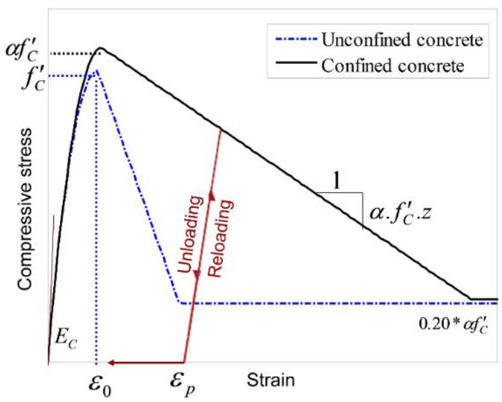

Figure 6. Concrete compressive stress-strain curve

$$\varepsilon_{ut} = 2 \times (\frac{G_f}{f_t}) \times \ln(\frac{3}{L}) / (3 - L) \quad (16)$$

Where L denotes the element length in mm and G_f is the fracture energy that is dissipated in the formation of cracks of unit length per unit thickness, and is considered as a material property. f_t is concrete tensile strength. For normal strength concrete, the value of G_f / f_t is in the range of 0.005-0.01 [20]. In this research, the average value of 0.0075 is assumed for G_f / f_t. The rules suggested by Karsan and Jirsa [21] are adopted for the hysteresis behavior of the concrete stress-strain relation in the compression region. In addition, the unloading-reloading branches that always pass the origin, regardless of the loading history, are assumed in the tension region [22].

3.2. Cyclic stress-strain relation of steel bars

The Giuffre-Menegoto-Pinto model is adopted to represent the stress-strain relationship of steel bars. This model was initially proposed by Giuffre and Pinto [23] and later used by Menegoto and Pinto [24]. This model is modified by Filippou et al [25] to include isotropic strain hardening (Figure7). The model agrees very well with experimental results from cyclic tests of reinforcing steel bars [26].

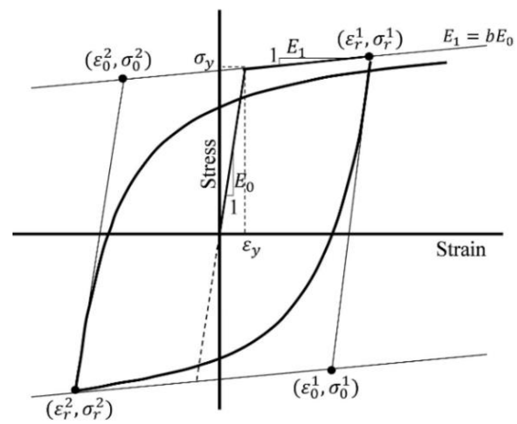

Figure 7. Cyclic stress-strain relation of steel bars

3.3. Cyclic bond stress- slip relation

Bond stress is referred to as the shear stress acting parallel to an embedded steel bar on the contact surface between the reinforcing bar and the concrete. Bond slip is defined as the relative displacement between the steel bar and the concrete. The adopted model to represent the bond slip effect between bars and concrete is proposed by Eligehausen et al. [27] shown in Figure 8. In this model, the effect of many variables, such as the spacing and height of the lugs on the steel bar, the compressive strength of the concrete, the thickness of the concrete cover, the steel bar diameter, and the end bar hooks, are considered. More details about unloading and reloading branches and bond strength degradation related to this model are given in [28].

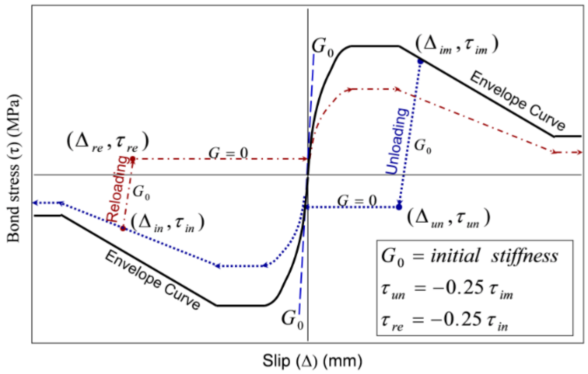

Figure 8. Cyclic bond stress-slip relation

3.4. Cyclic shear stress-strain relation

The adopted model to represent the shear stress-strain is that proposed by Anderson et al. [29]. This model replicates cyclic degradation in shear strength and stiffness (modulus) and energy dissipation for unloading and reloading state of behavior (Figure9).

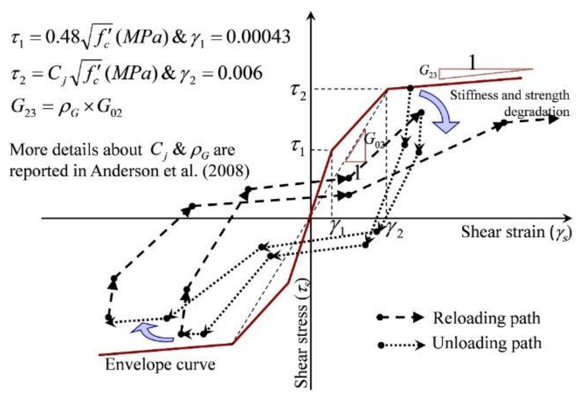

Figure 9. Cyclic shear stress- strain relation

4. Numerical investigation

4.1. Cyclic analysis

In order to analyze RC shear walls based on the proposed method, a computer program has been developed. The solution of equilibrium equations is typically accomplished by an iterative method through a convergence check. In this research the Newton-Raphson method is used as nonlinear solution algorithms. Also the Gauss-Lobatto method is used for numerical integration in which the number of integration points is equal to five.

For a reinforced concrete shear wall with geometric specifications according to Figure10 and details provided in the Table 1, numerical validation has been done. This specimen is a shear wall under uniaxial bending and constant axial load with magnitude of 630 kN. Lateral cyclic displacement was imposed at the free end. It was tested by Dazio et al. [30].

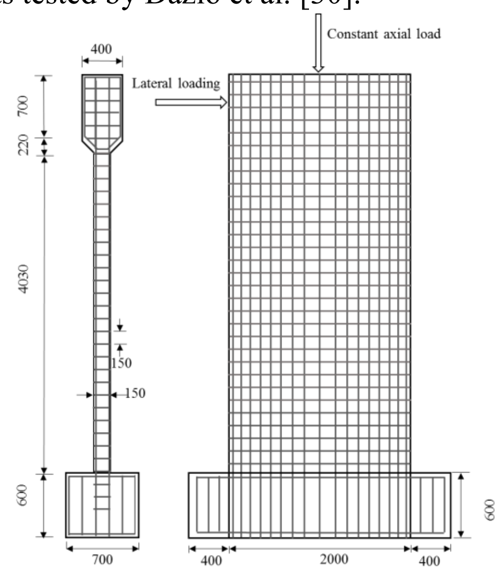

Figure 10. Geometry of Specimen1 (all dimensions in mm) [30]

The cross section of the specimen has 2 m length and 0.15 m width. Location of applying lateral load is 4.56 m above the foundation. The cross section of Specimen1 is shown in Figure11.

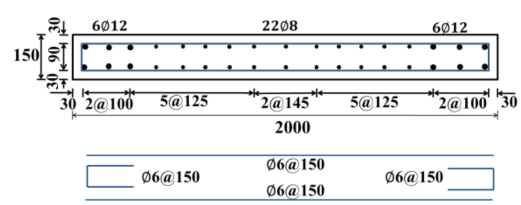

Figure 11. Reinforcement and section geometry of the Specimen1 (all dimensions in mm) [30]

Table1. Details of Specimen 1 [30]

Vertical Bars	12Φ12mm & 22Φ8mm
Horizontal Bars	Φ6@150mm
f_y of Vertical Bars (MPa)	576 for Φ12 & 583.7 for Φ8
f_y of Horizontal Bars (MPa)	518.9
f_y of closed ties and S- shaped ties (MPa)	518.9 for Φ6 & 562.2 for Φ4.2
f_c (MPa)	40.9
Concrete cover (mm)	30

In numerical modeling, the wall is subdivided into enough number of shorter elements. Because the formulation is displacement based and the response is depend on element size and it is needed the length of elements be enough short. As a simple suggestion, the length of the RC elements can be selected smaller than or equal to the average crack spacing in the wall [12]. In these cases, convergence of the calculated responses will be achieved in the numerical process. The minimum required embedded length is satisfied in Specimen1 in order to prevent pull-out of the bars from the footing connection and affect the results. In Figure12, numerical load-displacement response of the Specimen1 achieved from the Euler-Bernoulli and Timoshenko beam theories is presented and compared with experimental one. Numerical response using the Timoshenko theory, due to consideration of the shear deformation has been better matched with the experimental result. Results show, applying the Euler-Bernoulli theory, due to neglecting the shear deformation, leads to more stiffness and less deformation during the unloading and reloading paths.

Shear stress-strain cyclic behavior at the position with zero distance from the footing is calculated based on the employed theory and shown in Figure13. Shear strength and stiffness degradation effect is visible in cyclic behavior.

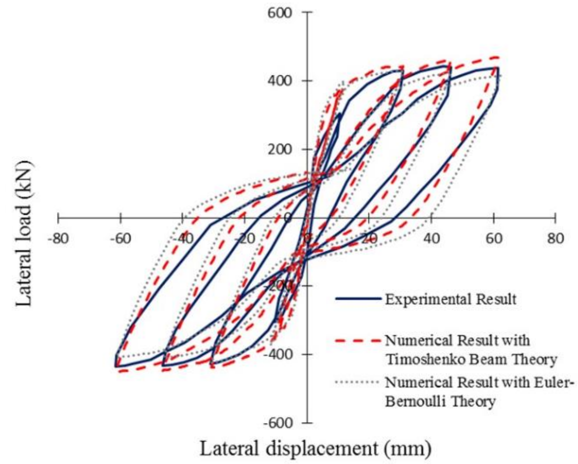

Figure 12. Experimental and numerical cyclic load-displacement responses for Specimen1

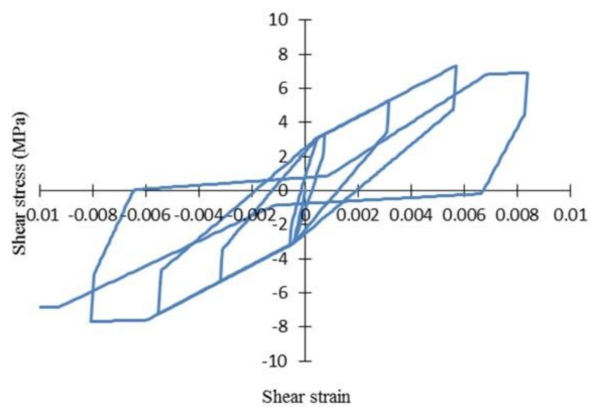

Figure 13. Cyclic shear stress-strain behavior at the position with zero distance from the footing of the Specimen1

4.2. Pushover analysis

Dimensions of tested Specimens 2 and 3, also arrangements of vertical and horizontal reinforcements are shown in Figures 14 and 15, respectively. The vertical and horizontal reinforcement's diameters are 8 and 6.25 mm, respectively. The yield strength of vertical and horizontal reinforcements are 470 MPa and 520 MPa, respectively. These shear wall specimens was tested by Leafs et al. [31]. Material properties and loading conditions of Specimens 2 and 3 is shown in Table2.

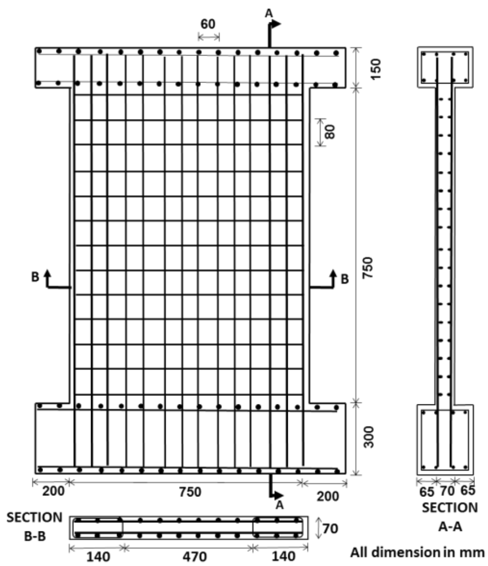

Figure 14. Geometry of Specimen2 (all dimensions in mm) [31]

Table 2. Material properties and loading conditions of the Specimens 2 and 3 [31]

	Axial load(KN)	$f_c^{/}$ (MPa)	f_t (MPa)	E_c (MPa)
Specimn 2	355	34.5	1.94	29362
Specimn 3	182	43	2.16	32824

In Figures 16 and 17, experimental results are compared with analytical ones in a pushover analysis for the Specimens 2 and 3. The results for both specimens show that the analytical ultimate capacity is in high conformity with the experimental response. Load-displacement response analysis of Specimen3 by using the Euler-Bernoulli theory is also calculated and shown in Figure17. The Numerical curve obtained by Euler-Bernoulli theory in comparison with that obtained from Timoshenko beam theory has overestimated and unrealistic stiffness.

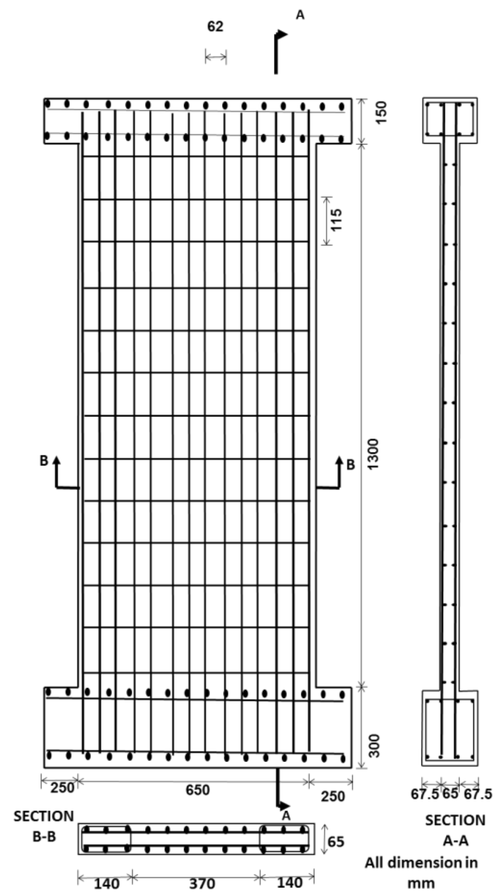

Figure 15. Geometry of Specimen3 (all dimensions in mm) [31]

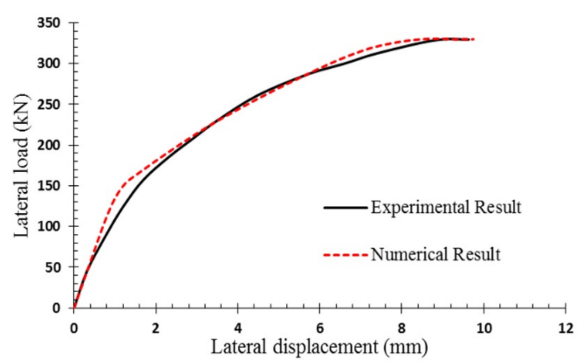

Figure 16. Experimental and numerical load-displacement responses for Specimen2

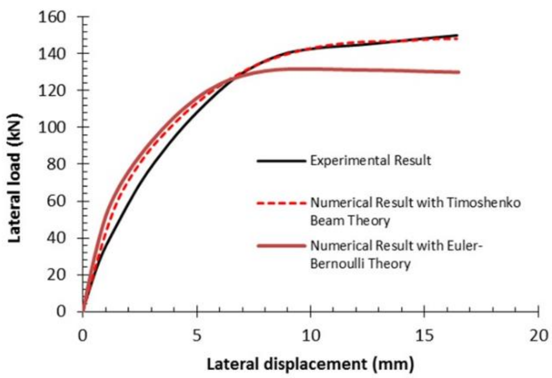

Figure 17. Experimental and numerical load-displacement responses for Specimen3

5. Conclusions

Most available numerical nonlinear methods which have been developed based on fiber theory, ignore shear deformation and usually are based on the perfect bond assumptions between bars and surrounding concrete. In this research, a numerical model based on fiber model is introduced for nonlinear cyclic and pushover analysis of RC shear wall and the effects of bar-concrete interaction and also shear deformation have been considered in the formulation. Formulation is displacement based and shape functions are used in order to express the internal displacements in term of nodal displacement. Two types of joint element and RC element are used for modelling of shear walls.

The reliability of the method is assessed through a variety of tested specimens under cyclic and pushover loading and good agreement between experimental and numerical results is obtained for both cases of strength and stiffness during analysis.

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