論文 Simplified Nonlinear Analysis of RC Members with consideration of Pullout Effect

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ABSTRACT: The calculated load-displacement behavior generally showed stiffer behavior as compared to the experimental results in the pre-peak part. The pullout of reinforcement has been found as the main reason for this phenomenon. Reinforcement behaves differently when embedded in concrete as compared to their behavior when it is tested as a bare bar. A developed simplified analytical tool for the RC member is used to understand this effect. A theoretical approach of decreased modulus of elasticity of reinforcement was applied and the behavior of the experiments was calculated under monotonic and cyclic loading. Good similarity has been observed between experimental and analytical results.

KEY WORDS: pullout effect, cyclic behavior, RC column, bond

1. INTRODUCTION

Due to recent major earthquake disasters the prediction of the behavior of reinforced concrete members is a hot topic of the research. Various researchers are trying to formulate more sophisticated analytical tools for more accurate simulation of different structural phenomenon.

A simplified analytical tool, based on finite difference method, is developed to analyze the reinforced concrete members under cyclic loading. Displacement-controlled algorithm is implemented with detailed cyclic material behavior. This tool is used successfully for calculating the load-displacement behavior of RC members under monotonic as well as cyclic loading. Similar to other analytical tools, member behavior that is calculated by using this method, showed comparatively stiffer behavior than experimental results in the pre-peak of the load-displacement curve.

This is due to pullout of reinforcement and mostly this phenomenon is common in the bridge piers, footing of the columns and the exterior beam-column joints. In case of the cyclic loading, the occurrence of the pullout of the reinforcement increases the hysteretic energy and also effects the unloading and reloading branches. For retrofit strategies of the earthquake disasters, the displacement corresponding to the load is important for defining the level of disaster. Hence it is important to work out the pullout of reinforcement in order to improve the analytical results.

Researchers generally measure the additional deformation due to the pullout of the reinforcement by displacement gauges attached at the beam column joint. The addition of this deformation in the analytical displacement gives the actual displacement. The accuracy of such data is quite difficult to maintain during the cyclic loading and also the analytical tools need proper implementation of this phenomenon. Shima et al[1] and Ishibashi et al[2] have proposed an empirical equation with bar diameter and clear distance among the main bars as the parameter for the calculation of the pullout displacement. The pullout displacement calculated by the empirical

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equations is quite conservative and needs further attention. The measured strain on the main bar at the column footing joint was also being used for calculating the pullout. The pullout displacement calculated from the strain in the main bar underestimates this behavior. It is obvious that the bar slip is involved in the pullout phenomena and its consideration is necessary in the calculation of the member behavior.

It is well known that the behavior of the reinforced concrete members under cyclic loading depends on the hysteretic behavior of the material and on several factors related to the interaction between reinforcing bar and concrete. Generally, the analysis of the reinforced concrete members is performed with perfect bond between steel and concrete. This assumption is true for low level of loads but as load increases the bond deterioration starts and the bar slip occurs. This leads to an additional displacement other than the deformation and generally refers as pullout of reinforcement.

In this paper, the simplified formulation is presented briefly with materials model. The pullout of the reinforcement is studied with the consideration of bond slip behavior in the material model of steel. A numerical test result of the effect of different degree of bond between concrete and steel on the material model of steel has been taken from Monti et al[3] and applied for the calculation of different experimental results using the simplified analytical method. The two RC specimens with and without axial load are simulated for monotonic and cyclic loading cases and this type of approach is found quite promising in improving the analytical results though the more concrete work is needed to understand the effect of bond characteristic on the steel material model.

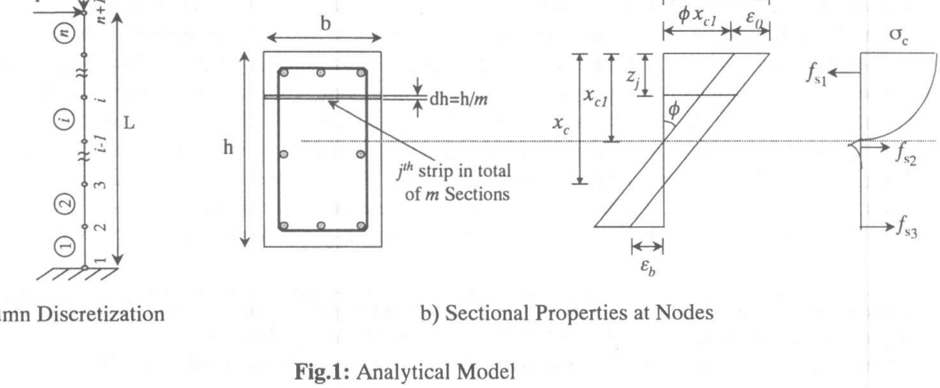
2. THE SIMPLIFIED FORMULATION

This formulation is based on relatively simple method of finite difference. The following are the assumptions for the formulations:

1. Plane section remains plane before and after bending
2. Concrete is homogeneous isotropic material
3. Deformation due to shear is neglected
4. There is no secondary moment in the column (P-Δ effect)

A reinforced concrete cantilever column of height L and cross-section dimensions of b and h is taken to present the analytical tool. In the analytical model, the column is divided into n elements with n+1 nodes as shown in **Fig.1a**. The cross-section is divided into m number of strips of thickness dh each. Lateral load P and varying axial load P_{axl} are applied at the top of the column. This varying axial load is taken as input from the experimental observations. Nodal variables of displacement y_i, rotation θ_i, curvature ϕ_i and strain at the top of the section ε_{ti} at i^{th} node are considered. Strain at the j^{th} strip of the cross-section can be defined as

$$\varepsilon_j = \varepsilon_{t_i} - y_j\phi_i \tag{1}$$



a) Column Discretization b) Sectional Properties at Nodes

Fig.1: Analytical Model

where $y_j = x_c - z_j$ and each strip of the section is further subdivided into parts of area A_{jm} according to the different material properties. These areas are core concrete, cover concrete and reinforcement. Stress σ_{jm} is calculated for each of these areas based on their material characteristics. l_i is the distance from the point of application of load P to the i^{th} node. The internal axial force P_{ax} and the external and internal bending moment, M_{iEX} and $M_{i_{IN}}$ at the nodes can be calculated as:

$$P_{ax} = \sum_j \sigma_j A_j \tag{2}$$

$$M_{iEX} = l_i P \tag{3a}$$

$$M_{i_{IN}} = \sum_j \sigma_j A_j z_j \tag{3b}$$

Using above equations we can get the following equation

$$\delta M_i = (\partial M / \partial \phi)_i \delta\phi_i + (\partial M / \partial P_{ax})_i \delta P_{axi} \tag{4}$$

In the above equation, $(\partial M / \partial P_{ax})_i \delta P_{axi}$ is the component due to varying axial load during the experiment. The nodal variables of displacement y, rotation θ and curvature ϕ between two consecutive nodes in the incremental form are assumed to be related to each other as follows,

$$\delta y_{i+1} - \delta y_i - (\delta\theta_{i+1} + \delta\theta_i)\Delta l/2 = 0 \tag{5}$$

$$\delta\theta_{i+1} - \delta\theta_i - (\delta\phi_{i+1} + \delta\phi_i)\Delta l/2 = 0 \tag{6}$$

Here $\Delta l = l_i - l_{i-1}$. The stiffness matrix in the incremental form can be constructed using **Eq.(4)**, **(5)** and **(6)**. There are n +1 variables of displacement δy_i, rotation $\delta\theta_i$ and curvature $\delta\phi_i$ at each node. The applied cyclic load δP is taken as a global variable. Thus there are n+1 equations from **Eq.(4)** and n equations each from **Eq.(5)** and **Eq.(6)**. This **Eq.(4)** will not be relevant at the n+1th node because of the boundary conditions of $\delta\phi_{n+1}$=0 and l_{n+1}=0. Therefore 3n equations have been taken with 3n +4 unknowns with 4 boundary conditions (at the support $\delta y_1 = 0$, $\delta\theta_1 = 0$ and at the top $\delta\phi_{n+1} = 0$, applied δy_{n+1}) and hence system of equations can be solved. In spite of stiffness matrix being unsymmetrical, there was no problem in the solution is noted[4].

The solution of stiffness matrix gives the nodal variables of displacement y_i, rotation θ_i, and the curvature ϕ_i. At each node, based on the calculated curvature ϕ_i, strain at the top of the section ε_{t_i} is calculated in an iterative manner such that external axial force balances the internal axial force of **Eq.(2)**. The top strain in the section is taken as convergence criteria for the moment curvature relationship calculations. The global convergence at each node is checked after convergence of the internal forces. The difference between moment due to internal forces and moment due to applied lateral force is taken as the criteria for global convergence. The unbalanced moment is iterated to get the converged moment.

3. MATERIAL MODELS

(1) Material model for reinforcement

The cyclic stress-strain model for reinforcement adopted in the analysis is presented in **Fig.2**. This is a multi-linear model that deals with one-dimensional stress-strain relationship for steel.

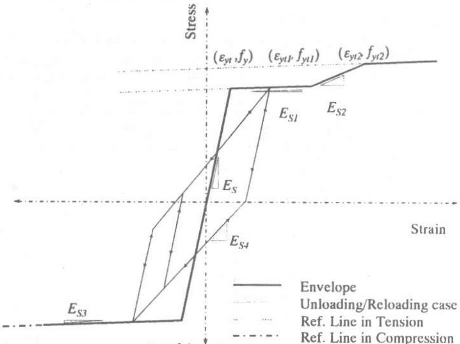


Fig.2: Reinforcing steel stress-strain model

More details can be found in Nasir et al[4].

Here, ε_{yt1} =0.007, ε_{yt2} =0.05, $f_{yc} = -f_{yt}$, E_s=2.1x10^5MPa, and E_{s4}=$m_1 E_s$ where m_1=0.05 for S specimen and m_1=0.1 for T specimen and has been adopted by trial error method in order to match with experimental results.

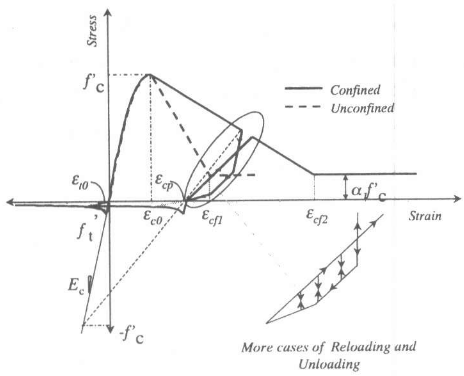


Fig.3: Concrete stress-strain model

(2) Material model for concrete

The stress-strain envelope curve for concrete is taken parabolic until peak and bilinear softening is taken as shown in **Fig.3** and more details can be found in Nasir et al[4]. Here ε_{c0} =0.002, is the strain at the peak stress f_c' and strain ε_{cfi} is the controlling parameter for the softening slope of the stress-strain curve. The values of i are taken as 1 for unconfined concrete and 2 for confined concrete. This factor ε_{cfi} physically depends on the tie spacing, number of ties and the strength of concrete. In tension, linear behavior until peak and gradual degradation in the post-peak range is adopted.

The accumulated plastic strain during cyclic loading ε_{cp} is calculated by focal point model for better numerical convergence[4]. It is taken as the ordinate of intersection of strain axis with line joining from the unloading point at the compression envelope curve, to the point ($-f_c'/E_c$ $-f_c'$) and is shown in **Fig.3**.

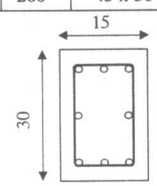
4. SIMULATION OF EXPERIMENTS

Two different experiments have been considered. Both are cantilever reinforced concrete columns with lateral load applied at the top of the specimen.

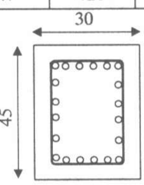
Specimen T is without axial load and specimen S is with axial load. More details of the specimens are presented in **Table 1**. In specimen T, one cycle of each displacement magnitude was applied and in specimen S, three cycles of each displacement magnitude had been applied at the top of the column. The pullout of reinforcement is measured but the data was not so reliable and is not presented here.

Table 1: Details of specimens

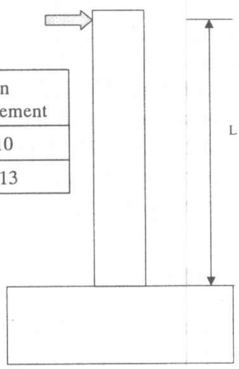
Specimen	Height L(cm)	Cross Section(cm)	Axial load N/mm^2	Concrete f_c' N/mm^2	Steel f_{yt} N/mm^2	Steel f_{yc} N/mm^2	Main Reinforcement
T	163	30 x 15	0.0	22	420	-420	8 D10
S	260	45 x 30	1.85	28.7	420	-420	20 D13



(a) T Specimen section



(b) S Specimen section



(c) Specimen

Fig.4: Specimen and cross-section details

5. PULLOUT OF REINFORCEMENT

The observation of most of the analytical result in comparison of the experimental results showed stiffer behavior in the initial stage. This is due to the pullout effect of the reinforcement, which is due to the bond slip between the reinforcement and the surrounding concrete. Hence the stress-strain relation of a bare bar, which during testing was ideally gripped at the ends, showed different behavior when it is embedded in concrete. The application of the standard anchorage to the bars also couldn't prevent the bar slip. Hence, the bond slip is important and it is needed to consider the effect of bar slip on the material model.

Monti et al[3] has presented the bond slip formulation for the beam element with the effect of bond slip for FE formulation, based on flexibility method. A numerical test using this formulation on a steel bar anchored on 20 diameters was performed to study the effect of the bond slip on the stress-strain behavior of steel. It is necessary to mention here that the effect of bond slip on the stress-strain behavior is purely theoretical and taken here from the reference[3] though it shows the probable effect of slip on the bar behavior. **Fig.5** shows the bond stress to bar slip relation and corresponding constitutive law with diameter of the bar as parameter.

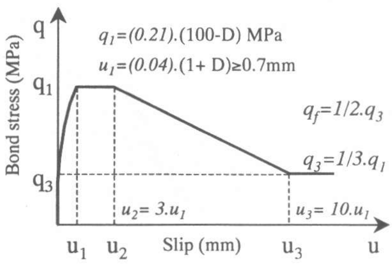


Fig.5: Bond slip constitutive law with parameter based on bar diameter, D[3]

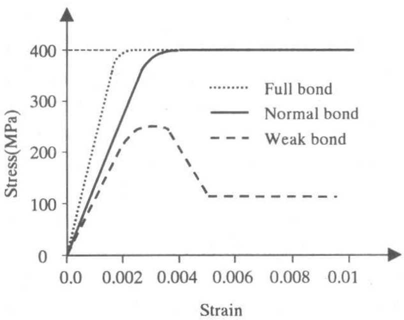
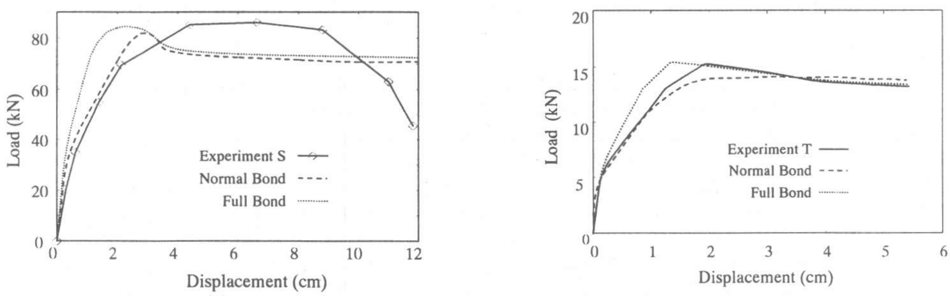


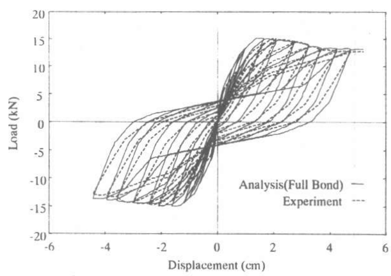
Fig.6: The stress-strain of Steel (anchorage length=20D)

The effect of the beam element formulation with bond slip consideration (**Fig.5**), on the material behavior of steel is shown in **Fig.6**[3]. In the numerical test, three degrees of bonds are considered. 'Full bond' refers to the steel stress-strain relationship when tested as bare bar. 'Normal' and 'Weak bond' refers to the steel stress-strain relationship when it is numerically tested with consideration of the bond slip constitutive law as mentioned in **Fig.5**. Normal bond case is selected when the steel bar yields before the bond reaches the plateau (represented by q1 in **Fig.5**) whereas weak bond refers to the case where bond breaks and starts softening before the steel reaches the yielding point. The case of weak bond arises if enough anchorage length is not provided. The specimens considered here were properly detailed and hence the case of the weak bond will not be discussed here. In full bond, modulus of elasticity E_s is taken 100% whereas considering normal bond in **Fig.5**, E_s of steel showed 33% reduction as compared to full bond[3]. For cyclic loading, the same reduction in E_s (67%E_s) as in the monotonic loading case has been adopted. Hence, the slopes of reloading/unloading lines, which are the parameter of E_s, have been changed accordingly.

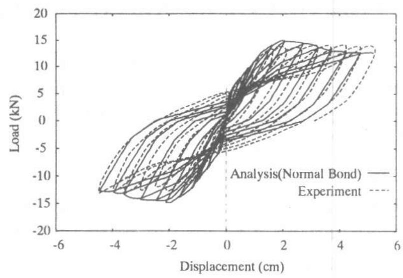


(a) Specimen S (b) Specimen T

Fig.7: Reinforcement model for steel with consideration of pull out effect of steel



(a) Specimen T with full bond



(b) Specimen T with normal bond

Fig.8: Reinforcement model for steel with consideration of pull out effect of steel

Cyclic loading cases need to be studied more critically as there might be a point in the stress-strain history when the anchorage slippage occurs in cyclic loading.

The experimental load-displacement envelope curve is compared with the monotonic analytical results with full and normal bonds and presented in **Fig.7**. As expected, the monotonic analytical results with full bond showed the stiffer behavior in the pre-peak part whereas the application of the normal bond between the concrete and steel gave almost the similar behavior in experimental and analytical results in both S and T specimens with a little lower peak load in the case of specimen T. For the case of specimen T, without axial load, cyclic behavior is calculated based on the full bond and normal bond as shown in **Fig.8(a)** and **8(b)** respectively. The normal bond case gave good matching in the initial stages and also in the loading and unloading branches of the hysteric curve. The case of specimen with axial load (specimen S) is having some convergence problem and need further attention.

6. CONCLUSIONS

A simple method for analysis of reinforced concrete members based on finite difference method has been used for the analysis of RC members. In this paper, the pullout of reinforcement is discussed using a simplified formulation. The stiffer analytical result in the pre peak part of the load-displacement behavior is taken care by considering the bond slip between the concrete and steel bar. This is a theoretical approach[3] just to understand the effect of the bond slip on the load-displacement behavior of the member.

The experimental results with and without axial load are simulated. The following conclusions can be made:

1. The simulation of the experimental behavior with consideration of the bond slip has given good matching between the analytical results under monotonic loading with the experimental envelope curve especially in the pre peak part of the load-displacement curve.
2. The application of simple bond slip for the cyclic loading simulation gives good matching in the case of specimen without axial load. The specimens with axial load are still under consideration.

REFERENCES

1) H. Shima, L. Chou and H. Okamura: Bond-Slip-Strain Relationship of Deformed Bars Embedded in Massive Concrete, Proc. of JSCE, pp.165~174, No. 378/V-6, 1987(Japanese).
2) T. Ishibashi, S. Yoshino: Study on Deformation Capacity of Reinforced Concrete Bridge Piers Under Earthquake, Proc. of JSCE, pp.57~66, No. 390/V-8, 1988(Japanese).
3) G. Monti and E. Spacone: RC Beam-Column FE including Bond-Slip, *Seminar on Post-Peak Behavior of RC Structures Subjected to Seismic Loads*, Japan Society for the Promotion of Science, Vol. 1, pp141-155, Oct. 1999
4) S. Nasir, S. Gupta and H. Umehara: Simplified Approach to the Non-Linear Behavior of RC Members, *12th world Conference on Earthquake Engineering,12WCEE*, 30th Jan-4th Feb. 2000, Auckland, New Zealand