Nonlinear Finite Element Analysis of Reinforced Concrete Beam-Column Connection with Interface Elements under Cyclic Loading

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Abstract To study the nonlinear response of corner beam-column junctions with inclusion of the effect of construction joint between the column and the beam cast at different times and subjected to cyclic and repeated loads, a computer program of three dimensional nonlinear finite element analysis, written by Al-Shaarbaty[1], (P3DNFEA) has been extended to account for the effect of construction joints on the behavior and to deal with concrete behavior under cyclic loads.

The 20-node isoperimetric brick elements have been used to model the concrete, while the reinforcing bars are modeled as axial members embedded within the brick elements. A nonlinear cyclic behavior model for concrete is developed in uniaxial and multiaxial states of stress. Also, a nonlinear cyclic behavior model for reinforcing bars is presented.

In completion, the behavior of concrete under cyclic loads is simulated by an elasto-plastic work hardening model followed by a perfectly plastic response. In tension, affixed smeared crack modeled has been used to simulate the behavior of concrete with a tension-stiffening model to represent the retained post-cracking tensile stresses in concrete. Closing and reopening of cracks during cyclic loading has been taken into consideration.

The nonlinear equations of equilibrium have been saved using an incremental-iterative technique based on the modified Newton-Raphson method. The convergence of the solution was controlled by a force convergence criterion. The numerical integration has been conducted by using 27-point Gaussian rule.

To represent the shear transfer between two concretes cast at different times, 20-noded interface layer brick elements are used with Fronteddu’s and Millard’s models to represent the aggregate interlock and the dowel stiffness, respectively.

Comparison between the results obtained from the finite elements and the available experimental results is made for a corner beam-column junction with inclusion of the effect of a construction joint. Good agreement is obtained. The maximum difference in ultimate load is 3.9%.

A parametric study dealing with construction joint is presented by taking various conditions of the junction. These include the axial load on the column, strength of concrete in the second cast.

Objective of the Project
The objective is to analyze reinforced concrete structures with considering the effect of construction joints and the effect of cyclic loading.
1. Introduction and Literature Review

The reinforced concrete beam-column joint is defined as the portion of the column within the depth of the beam framing into the column. An important result from the research done so far is that the notion of a rigid joint can be discarded forever. Thus, just as beams, columns and other structural elements exhibit flexibility in response to applied loading, so do the joints.

Sarsam and Phipps (1985)[2], reported on tests carried out by the author that there are five high strength exterior beam-column joints. The loading was applied in two stages. In the first stage, the column was loaded to a predetermined concentric load and this was kept constant throughout the test. In the second stage, the beam was loaded near its tip as a cantilever until either the joint failed or the beam failed. The specimens were grouped to examine different parameters.

Hamil et al (2000)[3], used two-dimension nonlinear finite element method for the analysis of high strength reinforced concrete beam-column connection. Quadrilateral elements formed from a pair of triangular elements with nodes at each corner were used to represent concrete while the reinforcement was represented by bar element. Perfect bond between concrete and steel was assumed to occur. The developed model was shown to be sensitive to changes in concrete strength, the detailing arrangements of the tension steel and the presence or absence of joint ties. From their investigation, the strength of a joint without column ties was proportional to the square root of the concrete’s compressive strength.

Hamza (2005)[4], studied the behaviour of beam-column joints with nonlinear interface elements. She assumed the interface as a thin brick element with Millard dowel forces.

2. Classification of Beam-Column Joint

Structural joints are classified into two categories in accordance with the loading conditions on the joint.

1. A joint for which the primary design criterion is strength and no significant inelastic deformations are expected.
2. Joint connecting members for which the primary design criterion is the sustained strength under reversals in the inelastic range. The requirements for joints are dependent on the deformations at the joint imposed by the loading condition.

3. Outline of the Computer Program

The computer program P3DNFEA (three-dimensional nonlinear finite element analysis) has been used in the present work. This program is originally developed by Al-Shaarabaf. The main objective of the program is to analyze reinforced concrete members under general three-dimensional states of loading up to failure.

In the present research work, the computer program has been modified to analyze beam-column joints under cyclic loading. A nonlinear cyclic behaviour model for concrete is used for uniaxial and multiaxial states of stress. Also, a nonlinear cyclic model for reinforcing bars is presented. Closing and reopening of cracks during cyclic loading has been taken into consideration.

4. Termination of the Analysis

The finite element analysis is terminated when any of the following criteria is satisfied.

1. The stiffness matrix is no longer positive definite.
2. A reinforcing bar has been fractured.
3. Excessive concrete crushing takes place.
4. The number of increments exceeds a maximum specified number.
5. The number of iterations exceeds a maximum specified number.
5. Crack Model for Concrete in Tension
A smeared fixed-cracking Is used to represent the crack model and it is widely used in connection with the finite element analysis. This implies that the cracks are distributed at a cracked sampling point [Al-Shaarbafl][1]. It is assumed that the concrete becomes orthotropic after the first crack has occurred. Cracks are assumed to form in a plane perpendicular to the direction of the maximum principal tensile .
6. Modeling of Reinforcement

In this study the reinforcing steel is modeled as a linear elastic, homogeneous material that can be assumed to have the same strength in tension and compression behavior and its mechanical properties in comparison to the properties of concrete are well known and well understood. Reinforcing bars are usually long and slender and therefore can be generally assumed to be capable of transmitting axial forces only.

In the current study, an elastic-linear work hardening model simulates the uniaxial stress-strain behavior of steel bars [Raid Ahmed 2005].

![Idealized elastic-plastic stress-strain relationship for reinforcing steel](image)

**Figure 2.** Idealized elastic-plastic stress-strain relationship for reinforcing steel

7. Closing and Re-Opening of Cracks

For a member under cyclic or repeated loading, cracks at sampling points may close. For a closing crack it is assumed that the orthotropic of the sampling point under consideration is maintained. Unloading and re-opening of cracks are assumed to follow a secant path. The secant path modulus, $E_1$, can be evaluated from the previously stored maximum strain developed normal to the cracked plane. This secant modulus may be used to calculate the retained stress as:

$$\sigma n = E_1 \varepsilon_n$$

Where $\varepsilon_n$ is the maximum strain developed normal to the cracked plane.

8. Shear Retention Models

In the finite element analysis of reinforced concrete members, a shear retention model is usually used to take into account the capacity of the cracked concrete to transfer shear across the crack. In the present study, a reduction factor has been used across the crack plane, to reduce the shear stiffness at the cracked sampling points. Before cracking a value of unity is assigned to the shear reduction factor. As the crack propagates, the shear reduction factor is taken to be linearly decreasing with the strain normal to the cracked plane, which represents the crack width. When the cracks have sufficiently opened, a constant value is assigned to account for dowel action. The shear retention model can be expressed as:

1) For $\varepsilon_n < \varepsilon_{cr}$

$$\beta = 1.0$$  \hspace{1cm} (1)
2) For \( \dot{\varepsilon}_{cr} < \dot{\varepsilon}_n < \gamma_1 \dot{\varepsilon}_{cr} \)

\[
\beta = \frac{\gamma_2 - \gamma_3}{\gamma_1 - 1.0} \left[ \gamma_1 - \frac{\varepsilon_n}{\varepsilon_{cr}} \right] + \gamma_3
\]

(2)

3) For \( \dot{\varepsilon}_n > \gamma_1 \dot{\varepsilon}_{cr} \)

\[
\beta = \gamma_3
\]

(3)

Figure 3. Shear retention for concrete

9. Outline of The Computer Program

The computer program P3DNFEA (three-dimensional nonlinear finite element analysis) has been used in the present work. This program is originally developed by Al-Shaarbagh. The main objective of the program is to analyze reinforced concrete members under general three-dimensional states of loading up to failure.

In the present research work, the computer program has been modified to analyze beam-column joints under cyclic loading. A nonlinear cyclic behaviour model for concrete is used for uniaxial and multiaxial states of stress. Also, a nonlinear cyclic model for reinforcing bars is presented. Closing and reopening of cracks during cyclic loading has been taken into consideration.

10. Interface Element

The behaviour of a composite concrete specimen depends upon the interaction between the two concrete cast at different times. There can be separation, closing of gap, and slipping between the two parts. In the present study a thin layer element is used to represent this behaviour.

10.1 Thin Layer Element

An isoperimetric finite element formulation, which is treated essentially like a solid element, can be used to represent the behaviour of the interface region [Desai and Zaman (1984)], since the element is treated essentially like any other solid element, its incremental stress-strain relationship is expressed as:

\[
\{d\sigma\} = [D]\{d\varepsilon\}
\]
Where $[D]$. 

### 11. Shear-Friction Concept

The shear-friction concept provides a convenient tool for the design of members for direct shear where it is inappropriate to design for diagonal tension, as in precast connections, and brackets or corbels. The approach is to assume that a crack has formed at an expected location, as illustrated in Fig. (4). As slip begins to occur along the crack, the roughness of the crack surface forces the opposite faces of the crack to separate. The separation is resisted by reinforcement ($Av_f$) across the assumed crack. The tensile force ($Av_f f_y$) developed in the reinforcement by this strain induces an equal and opposite normal clamping force, which in turn generates a frictional force ($Av_f f_y \mu$) parallel to the crack to resist further slip, where $\mu$ is the coefficient of friction.

![Figure 4. Idealization of the shear-friction concept](image-url)
12. Application of Finite Element Analysis to Beam-Column Joints
A three dimensional nonlinear finite element analysis has been carried out on corner beam-column joints by implementing the modified computer program. The three-dimensional model describe the behaviour of the analyzed corner beam-column joint under cyclic loading. The example has been chosen from the available experimental studies for the numerical analysis. The example was tested by [Sarsam (1983)] [5].

13. Beam-Column Joint Specimen (under monotic loads)
Nine specimens of beam-column joints (5 exterior and 4 interior) were tested by [Sarsam (1983)]. The plane exterior ones-Ex series were made of two pours. The first pour was made on one day. This pour included the specimen up to the level of the top of the joint. The second pour was made on the next day for the top column. Thus, a horizontal construction joint existed at top of joints .

13.1 Description of Beam-Column Joint Specimen
The column was reinforced with four 16 mm longitudinal bars and 8mm closed links at 85mm centre to centre spacing, giving three joint links. The beam was reinforced with two 16mm bars on the tension side and two 12mm bars on the compression side. Beam links were 8mm of closed type and spaced at 130mm centre to centre .

Ex1 specimen is used in the present study, its dimensions are shown in Table (1) and Fig. (5). Material properties and additional material parameters of this specimen are shown in Table (2). The column was first loaded to a predetermined value of (Nc), prior to any beam loading, the next stage involved loading the beam up to ultimate load .

| Dimensions | Beam | Column | Lc | Av | Column Load (kN) |
|------------|------|--------|----|----|------------------|
| h(mm)      | b(mm)| h(mm)  | b(mm)|    |                  |
| Specimen Ex1 | 303  | 152    | 205 | 152| 1531             |
|             |      |        |     |    | 1422             |
|             |      |        |     |    | 292.6            |

Table 1. Dimensions of Sarcasm's Specimens

Table 2. Material properties & Material Parameters of Sarcasm's Specimens

| First pour (age=64 days) | Material properties | Material parameters |
|--------------------------|---------------------|---------------------|
| Modulus of elasticity, E(MPa) | 35500               | Tension-stiffening parameter: \( \alpha_1 = 35 , \ \alpha_2 = 0.35 \) |
| Compressive strength, \( f'_c \) (MPa) | 56.3                |                      |
| Tensile strength, \( f_t \) (MPa) | 4.5                 | Shear-retention parameter: \( \gamma_1 = 25, \ \gamma_2 = 0.5, \ \gamma_3 = 0.1 \) |
| Poisson's ratio, \( \nu \) | 0.2                 |                      |

| Second pour (age=63days) | Material properties | Material parameters |
|--------------------------|---------------------|---------------------|
| Modulus of elasticity, E(MPa) | 30600               | Tension-stiffening parameter: \( \alpha_1 = 25 , \ \alpha_2 = 0.25 \) |
| Compressive strength, \( f'_c \) (MPa) | 45.8                |                      |
| Tensile strength, \( f_t \) (MPa) | 3.93                |                      |
| Poisson's ratio, \( \nu \) | 0.2                 | Shear-retention parameter: \( \gamma_1 = 25, \ \gamma_2 = 0.5, \ \gamma_3 = 0.1 \) |

Steel reinforcement
Longitudinal bar $\Phi 16$ 
Longitudinal bar $\Phi 12$ 
Stirrup bar $\Phi 8$

|               | Young's modulus (MPa) | Yield stress (MPa) |
|---------------|-----------------------|--------------------|
| Longitudinal bar $\Phi 16$ | 208000                | 504                |
| Longitudinal bar $\Phi 12$ | 198000                | 507                |
| Stirrup bar $\Phi 8$       | 197000                | 517                |

Figure 5. Experimental beam-column joint specimens of Sarsam (1983)

13.2 Finite Element Idealization
The concrete of specimen Ex1 is idealized by using 5820-noded brick elements (including 1 interface element at the top level of the joint), (for half of this specimen). To simulate the procedure of loading that occurred during the experimental test, the column axial load has been firstly applied in equal increments of 10% of the maximum column load for this specimen. Later, for Ex1 two different sizes of increments have been used for beam loading. The beam was loaded initially by increments of 37.5 kN up to 75% of the expected collapse load (40 kN). Then reduced increments of 1.43 kN each were
applied until failure load has been reached. Both the initial and post-cracking stiffness are reasonably predicted, table (2). For this ordinary reinforced concrete beam the tension-stiffening parameters $\alpha_1$ and $\alpha_2$ was set equal to 55 and 0.5 respectively. While the shear-retention parameters $\gamma_1$, $\gamma_2$ and $\gamma_3$ was set equal to 10, 0.5 and 0.1 respectively.

13.3 Analysis of Ex1 Specimen (under static load)
In order to analyze the specimen, the effect of the thickness (t) equal to 0.014mm, and 1.4mm have been carried out. The results show that the type of failure of the specimen Ex1 is beam hinging for the range (0.014-0.14)mm for thickness of interface element, fig. (6). A response stiffer than the experimental results was obtained when the thickness is reduced within the range, and the best fit to the experimental results was obtained at $t=0.14$mm with effective thickness of Gaussian point of 0.038mm, in which the effect of nonlinearities along the loading stages is clear. The failure load of numerical results is 37kN while the failure load of experimental results is 36.04 kN, so that the error ratio is 3.9%. In the present study the value of thickness of the interface element equal to 0.14mm is fixed for Ex1 specimen, Fig. (7).

**Figure 6.** Experimental and analytical response to vary interface thickness for Ex1

**Figure 7.** Experimental and analytical response for $t=0.14$mm for Ex1
14. Beam-Column Joint Specimen (under cyclic loads)
Also Ex1 has been taken. This was made of two pours. The first pour was made on one day. This pour included the specimen up to the level of the top of the joint. The second pour was made on the next day for the top column. Thus, a horizontal construction joint existed at top of joints.

Figure 8. Finite element mesh of half of Ex1

14.1. Finite Element Idealization
By making use of the geometric and loading symmetry, a segment, which represents one-half of the beam-column joint, has been used for the finite element analysis. This segment has been discretized into 5820-noded brick elements (including 1 interface element at the top level of the joint).
The column axial load has been firstly applied in equal increments of 10% of the maximum column load for this specimen. Later, four different sizes of increments have been used for beam loading. As in Table (2), For this ordinary reinforced concrete beam the tension-stiffening parameters $\alpha_1$ and $\alpha_2$ was set equal to 55 and 0.5 respectively. While the shear-retention parameters $\gamma_1$, $\gamma_2$ and $\gamma_3$ was set equal to 10, 0.5 and 0.1 respectively.

14.2. Analysis of Ex1 Specimen (under cyclic load)

Fig. (9) shows the numerical repeated load-beam tip deflection curve. Also the figure reveals that the first half-cyclic, a tri-segmental curve was usually recognized. The first segment represents the elastic-uncracked stage of behaviour. The second represents the elastic-cracked stage. While, the third stage represents the yielding of main reinforcement. During the sequence of half cycles, these segments will disappear and a smooth behaviour is seen which is characterized by cracking and post-yielding stages.

15. Parametric study

15.1 The Effect of Column Axial Load

In order to inspect the effect of column axial load on the behaviour of construction joint, a numerical study has been carried out, one with experimental column axial load ($N_c=292.6$ kN), and the other with half column axial load ($N_c/2$) for the case of construction joint at the top level of the beam-column joint. It can be observed from Fig. (11) and (12) that the shear and normal this feature may be the following: higher compressive stresses (at $N_c=292.6$ kN), in spite of the more intimate interlocking they secure, produced a shortening of the protruding asperities and subsequently reduce overriding resistance. This mechanism is less apparent at $N_c/2$. On the contrary, due to loss of some confinement for $N_c/2$, the response of the specimen at $N_c/2$ is softer than the response for $N_c=292.6$ kN, Fig. (10).
Figure 10. Load-beam tip deflection curve

Figure 11. Normal strains distribution in joint
Three numerical tests have been carried out by using percentages of steel across the construction joint (diameter of the bar)(16mm), (18mm), and (20mm) for the construction joint at the top level of the beam-column joint, these tests occurred with the original designed specimen. From Fig. (13), the deflection decreases with the increase in the steel percentage across the construction joint (column reinforcement), the contribution in this result is the decreased strains in joint due to increase in dowel stiffness. Figs. (14) and (15) show the shear and normal strains in joint. It can be notice that these strains decrease with the increase in the steel percentage across the construction joint.

Figure 12. Shear strains distribution in joint

15.2 The Effect of the Percentage Steel across the Construction Joint

Figure 13. Effect of diameter of crossing steel on load-beam tip deflection
16. Conclusions

The following conclusions can be drawn from the present study:

1. A good assessment can be obtained for the behaviour of corner beam-column joints by using the developed program of the current study (DPACJ) and is suitable to predict the behaviour of reinforced concrete under cyclic and repeated loads.

2. The performance of the interface element, used in this study to model the shear transfer between two concretes cast in different times, is quite good.

3. A stiff response can be obtained with decrease the thickness of the interface element.
4. The response of a specimen can be expected within a certain range of thickness of the
interface element. This range depends on the finite element mesh, nonlinear behaviour of
material, and the combination of stresses.
5. The construction joint would affect only on the joint. On the other hand, the mode of failure
for the corner beam-column joint in this study beam hinging, this type of failure conforms to
the design requirements.
6. The presence of column axial load would decrease the aggregate interlock stiffness. However,
it secures a good confinement for the beam, and so as the result of increase it.
7. The increased of steel percentage across the construction joint would decrease the strains in
joint. Consequently, a slightly decrease of deflection occurred.

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