CONTINUUM MODEL FOR RC INTERIOR BEAM-COLUMN CONNECTION REGIONS

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ABSTRACT :

Due to complexity of reinforced concrete beam-column connection behavior, experimental investigations have not provided definitive answers as to the impact of local nonlinear material inelastic response on the development of global mechanisms that determine connection behavior. The current study investigates the application of the state-of-art commercial continuum finite element analysis software DIANA to simulate interaction between local and global mechanisms that govern failure and thereby to better understand the nonlinear response of reinforced concrete beam-column connections. The ability along with limitations of the state-of-art continuum finite element software DIANA in simulating the response of reinforced concrete interior beam column connections exhibiting all local inelastic mechanisms is presented in this research study. Recommendations are proposed for more generalized continuum study of reinforced concrete beam-column connections.

KEYWORDS: RC beam-column joint, connection region, continuum finite element, plasticity, bond-slip, concrete crack model.

1. INTRODUCTION

Reinforced concrete interior beam-column connections are one of the least studied critical components of a building or bridge structure. It should be mentioned that a connection region comprises of the joint region along with the adjoining area within the beam and column where the inelasticity is concentrated. Post earthquake reconnaissance and experimental investigations have revealed that strength and stiffness loss in reinforced concrete interior beam-column joint regions may result in strength and stiffness loss and, potentially, collapse of the entire structure. Even though a lot of experimental investigations have been performed on these structural components, the complexity of the mechanics involved prevents simulation of all local responses in predicting the global mechanisms that determine connection behavior. A number of component based models have also been developed (such as Mitra and Lowes 2007); however, it should be noted that most of these models are based on observations and assumptions made in the experimental investigations.

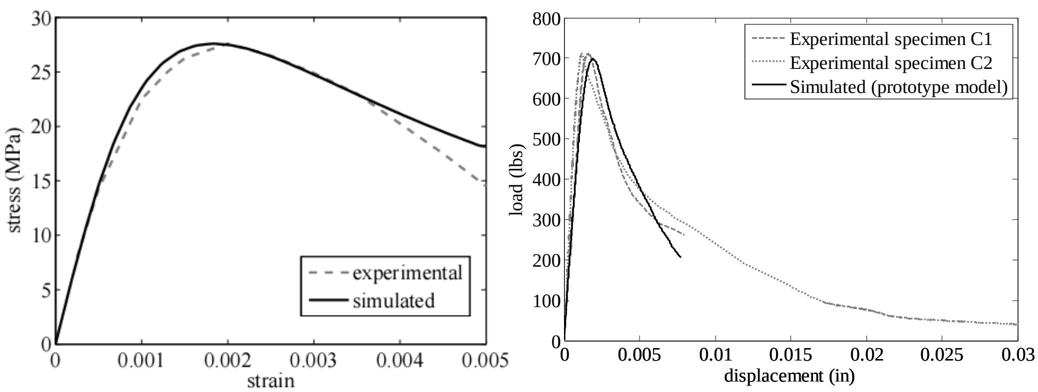
The local inelastic mechanisms governing behavior of the connection are cracking of concrete, crushing of confined and unconfined concrete, closing of concrete cracks under load-reversal, shearing across concrete crack surfaces, yielding of reinforcing steel and damage to bond-zone concrete. The objective of this current research is to primarily consider all of these local inelastic mechanisms in predicting correctly the global failure mechanism as observed from experimental investigations.

There exists very few previous research which considers continuum finite elements to model and simulate behavior of reinforced concrete beam-column connection regions, however these investigations did not account for all the local inelastic mechanisms governing beam-column connection response. Research by Will et al. (1972) was one of the first continuum finite element studies of joint regions. The investigation assumed brittle fracture for concrete and assumed linear elastic response for concrete in tension, compression and also the reinforcing steel and bond-link elements. Noguchi (1981) utilized a discrete crack approach to represent cracks in concrete. However, as had been identified later by many researchers (such as Rots and Blauuwendraad 1989), one of the major drawbacks of the discrete approach for use in concrete structures is the crack propagation path needs to be well defined a-priori to the analysis. Pantazopoulou and Bonacci (1994) utilized modified compressive field theory (MCFT), which primarily considers reduction in compressive strength due to tension in orthogonal direction, to represent behavior of concrete. Even though MCFT has been used successfully in many applications, the viability of utilizing the theory as a generalized material model for reinforced beam column joints is questionable (LaFave and Shin 2005, Lowes et al. 2005). Moreover, the direct portability of this theory for three dimensional analysis in which joints are subjected to complex 3-dimensional stress states are not easy. The model by Pantazopoulou and Bonacci (1994) however did consider frictional contact theory for simulating bond-slip between reinforcing steel and concrete, without relying on empirical bond-slip curves obtained from experimental investigations of pull out tests of reinforcement bars from concrete. Baglin and Scott (2000) and Hegger et al. (2004) utilized commercial finite element software SBETA and ATENA respectively to simulate response of beam-column connections, however these models considered perfect bond between reinforcing steel and concrete. A generalized continuum model for beam-column connections is presented in this research study, developed using state-of-art commercial continuum finite element software DIANA 9.1. The study also points out the limitations of the software in representing the inelastic mechanisms involved in a beam-column connection response and suggests recommendations for improvement.

2. MATERIAL MODEL CALIBRATIONS AND BENCHMARK TESTS

The material model utilized for the purpose of the simulation of concrete response is Drucker-Prager plasticity for compression and decomposed strain multiple fixed crack model (de-Borst and Nauta 1986) with Hordijk softening (Hordijk 1991) for tensile response. The combined model represents a multi-surface plasticity type of model. In a Drucker-Prager yield surface the octahedral shear stress depends linearly on the octahedral normal stress. In the current application, concrete is subjected to only moderate hydrostatic pressure loading (Lowes 1999) and thereby Drucker-Prager model is adequate for defining the yield surface of concrete. However, this is an assumption and may not be suitable for three-dimensional analysis of joints. In a three dimensional analysis the center of the joint region may be experiencing high hydrostatic loads and thereby a parabolic dependence of octahedral shear stress on the octahedral normal stress may be required through use of higher parameter yield

surfaces such as Ottossen (1977) and William and Warnke (1975).
Beyond definition of the yield surface, definition of a plasticity-based constitutive model also requires specification of the plastic flow rule. The flow rule defines the orientation of the plastic strain. Typically, to obtain improved numerical stability and due to lack of precise data for model calibration, associative flow is assumed (Lubliner et al. 1989). The primary drawback of this approach is overestimation of plastic dilation (Chen and Han 1988) at compressive loading, which can be a significant problem for cases of high hydrostatic pressure. For the present problem, this could be expected to result in over-estimation of the confining effect of transverse steel on concrete strength. Evolution of the plastic yield surface i.e., the hardening/softening rule, is the final component of a plasticity-based model. For the current study, a hardening/softening function is calibrated such that the concrete response under uniaxial compression matches an established and well-accepted empirical curve proposed by Popovics (1973). Based on previous micromechanics studies on concrete, it was assumed that concrete exhibits a linear response until compressive stress equals 30% of the maximum compressive strength. The hardening/softening envelope was defined by a nonlinear relationship between plastic strain and the cohesion parameter. Based on DIANA 9.1 manual on materials and considering the ratio between the uniaxial compressive strength and the equal biaxial compressive strength to be 1.16 (Kupfer and Gerstle 1973) the friction angle for the Drucker-Prager model was taken as 10 deg.
A good correlation between simulated and observed compressive stress strain response (Figure 1a) for a 25.4 mm square concrete specimen block with unconfined compressive strength of 27.6 MPa, and initial stiffness of 31.7 GPa. (Karsan and Jirsa 1969).



a) Uniaxial compressive response (Karsan and Jirsa 1969)

b) Response in three point notched beam test (Martin et al. 2007)

Figure 1. Simulation of concrete uniaxial response

Tension loading of a concrete structure results in development of cracks. A number of methodologies exist for modeling of cracks in concrete (the reader is referred to Mitra (2007) for detailed discussion of these methodologies). Based on availability of phenomenological smeared models for modeling of concrete cracks in the commercial software DIANA 9.1 utilized for modeling, a decomposed-strain multi-directional fixed crack model (de Borst and Nauta 1986) has been used to represent cracks in concrete because of it's simplicity and it's ability to be used with plasticity models. The model is based on decomposition of the total strain increment at a gauss-quadrature point into a concrete and a crack-strain increment. This decomposition permits the combination of the phenomenon of crack formation with other non-linear phenomena such as plasticity to represent the behavior of concrete in compression. As the name suggests, multiple cracks are allowed to form at a gauss quadrature point. A crack is said to originate once the cracking criterion is satisfied (i.e. user specified threshold angle is exceeded and also the maximum tensile strength is exceeded). The threshold angle refers to the angle from the plane of the first crack to a plane where the next crack can originate. Thereby, if the threshold angle is 180 deg. then this model reduces down to a single fixed crack model, whereas theoretically if the threshold angle is made equal to 0 deg and the effect of other open cracks at the gauss-quadrature points are erased then it represents a rotating-crack model (refer Gupta and Akbar(1984), Crisfield and Wills (1989)). In the current study the threshold angle was considered as 60 deg. Details about this model can be found in de

Borst and Nauta (1986) and also in the DIANA 9.1 manual on material modeling. De-Borst (2002) demonstrated that this decomposed strain multiple-fixed crack model is theoretically similar to damage plasticity models (Lubliner et al. 1989) and microplane model (Bazant and Prat 1988). Post peak tension softening in the model has been calibrated using the Hordijk strength deterioration model (Hordijk 1991).
Typically, experimental three-point bend tests on a notched beam are carried out to determine the tensile response and fracture energy of a specimen. Good correlation was observed between an experimental investigation at University of Washington (Martin et al. 2007) and numerical simulations (Mitra (2007) and paper no. 05-01-1075 in 14wcee), as shown in Figure 1b.
For the current two-dimensional study, reinforcing steel was assumed to act as a truss element and behave elastoplastically without Bauschinger effect. Von Mises plasticity was used to characterize the constitutive behavior of reinforcing steel. Bond between reinforcing steel and the concrete continuum element was modeled primarily by one dimensional interface elements whose material relationship was calibrated to match the uniaxial bond-slip model proposed by Eligehausen et al. (1983).
Good correlation was observed between simulated and experimental anchorage bond-zone response by Viwathanatepa et al. (1979) utilizing the above specified concrete response in tension and compression along with response of reinforcing steel and bond-slip response. The anchored bar is subjected to monotonically increasing elongation at one end of the exposed bar. The comparison of the observed and the computed response for these models provides a means of evaluating the adequacy of the proposed bond model for predicting bond-zone response under severe loading conditions similar to those that develop under earthquake loading of reinforced concrete buildings. Figure 2a shows the crack patterns developed in the simulation which matches with crack patterns observed in the experimental investigation. Figure 2b shows the correlation between the simulated and the experimental global response.

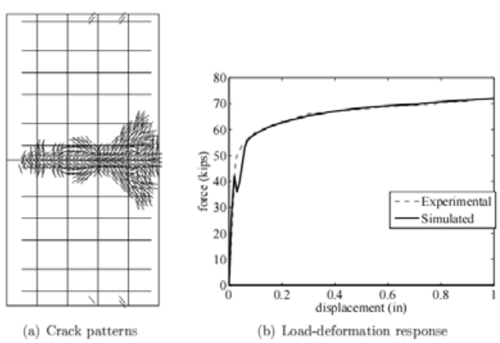


Figure 2. Simulation of anchorage bond slip response (Viwathanatepa et al. 1979)

Flexure tests of Burns and Seiss (1962) sample was simulated to demonstrate behavior with perfect and with the Eligehausen calibrated bond model as specified in the previous paragraphs. The difference in the crack pattern was observed between the case of “perfect bond element” and the case of provided material bond model, as shown in figure 3. If the bond is not perfect, then the cracking is not continuous and is much more discrete.


a) Perfect bonded specimen b) Specimen with Eligehausen bond model

Figure 3. Crack patterns as observed in flexural tests

3. SIMULATION OF JOINT RESPONSE

DIANA 9.1 software was used to simulate the experimental response of two beam-column joints which represent the range of expected global behavior for joints: one exhibiting brittle joint failure prior to flexural yielding of beams and another exhibiting flexural yielding of beams followed by joint damage. Material models along with their validations for the simulation have been discussed in details in the previous section. The proposed continuum model with loading and boundary conditions is shown in figure 4. Constant axial load is applied at the column top using load-control. A monotonically increasing lateral load is applied at the top of the column by displacement-control.

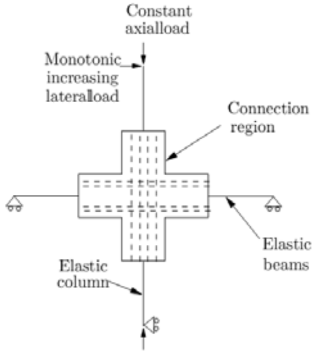


Figure 4. Simulated joint specimen with loading and boundary conditions

The joint region along with a plastic hinge region (taken equal to the depth of beam/column section), combined being referred to as the connection region, was discretized with concrete continuum elements. Four node quadrilateral plane stress elements with 2x2 gauss-quadrature integration were considered for the continuum elements. Embedded reinforcement elements with perfect bond representing stirrups were also used within the concrete continuum region. The longitudinal bars in beams and columns were modeled as 2 dof bar elements which were connected to the concrete elements through a line interface bond element. Beams and columns outside the connection region were represented by elastic line elements with an effective stiffness, equal to that of cracked concrete, as per ACI 318-05. The elastic line elements were specially connected to the concrete continuum elements so that there was proper transfer of moments from the beam/column line elements to the continuum elements. Material models described in the previous paragraphs were utilized for constitutive relations for the concrete, steel and bond.

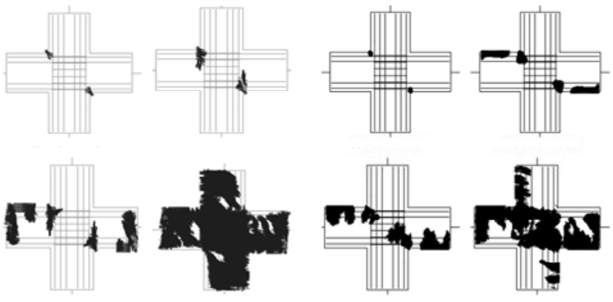
Two specimens from Oka and Shiohara (1992) were chosen for purpose of simulation, namely OSJ5 and OSJ10. Both specimens had a higher than average joint shear stress and bond stress demand. The concrete compressive stress was significantly different for the two specimens.

4. RESULTS AND DISCUSSION

Specimen OSJ5 exhibited a beam yielding followed by connection failure type of response. The connection failure could either be due to anchorage/bond failure of the longitudinal beam bars or could be due to shear failure within the joint region or it could also be due to simultaneous activation of both the failure mechanisms. In the literature for the OSJ5 specimen, the bond/anchorage response is not documented. Specimen OSJ10, on the other hand, exhibits a joint shear failure and thereby high compressive stresses are expected within the joint region resulting in a diagonal shear band type failure.

In simulations for the joint response, numerical convergence problems were observed when using all the material models as listed in the previous section. Details associated with the problem have been described later in the manuscript. Thereby, in our reduced modeling effort, all other material models were kept except for the compressive response of concrete which was made as linear elastic. The implication of this change in the simulated response would be that the post-yield characterization can not be done properly. However, the pre-peak response should meet that observed from the experimental investigations. Moreover, the pattern of cracks and sequence of mechanisms observed as in the experimental investigations should also be observed in the numerical simulation.

The crack patterns in OSJ5 simulation indicate that cracks initiate at the corner regions and propagate along the joint interface. Finally at a later displacement level, diagonal cracks are observed in the joint region. The pattern of crack propagation is shown in figure 5a. In the case of OSJ10, the cracks initiate at the joint corner perimeter and then progresses diagonally in the joint region (figure 5b). The crack patterns observed in OSJ10 correlates well with the crack pattern propagation obtained from simulation and is typical of joint shear type of failure. It should be noted that the pattern of cracks observed in the simulation of OSJ10 differs significantly from the pattern of cracks observed in the simulation of OSJ5.



a) OSJ5 crack propagation b) OSJ10 crack propagation

Figure 5. Simulated crack propagations

For OSJ5 specimen, it was observed that flexural steel yielded prior to concrete reaching compressive strength within the joint region. The distribution of normal stresses in the reinforcing steel is shown in figure 6. The distance 300-600 mm represents the region within the joint. Outside the perimeter of the joint, the reinforcing steel has reached yield strength. On the other hand, reinforcing steel was not observed to reach yield strength for the OSJ10 specimen response even though concrete elements had reached its compressive strength.

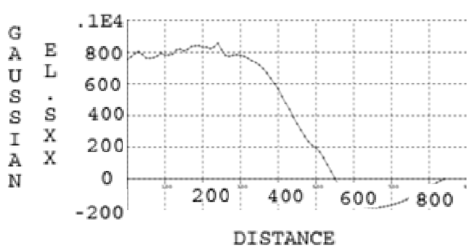
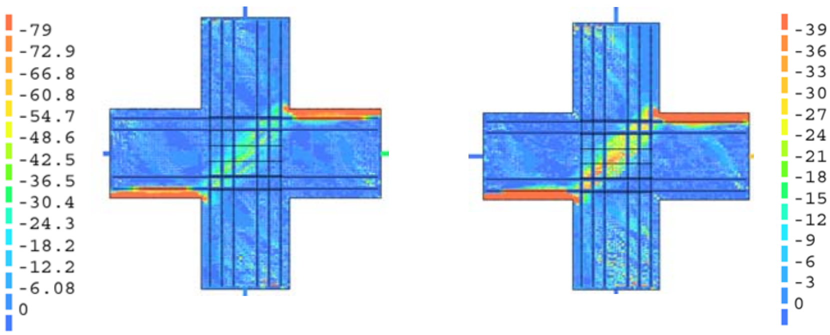


Figure 6. Stress distribution along the top reinforcement bar

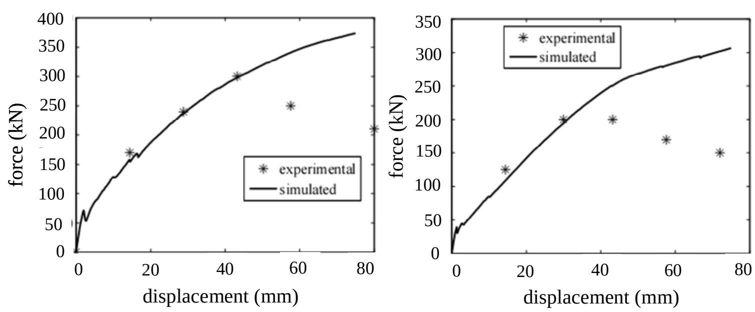
The compressive stresses observed in the joint region, at a displacement level beyond the peak load level, are shown in figure 7. It can be observed that the concrete within the joint region did not reach the maximum compressive strength of concrete (79 Mpa) for the OSJ5 specimen; however, the concrete have reached its maximum compressive strength (39 Mpa) for the OSJ10 specimen. The concrete regions reaching its maximum compressive strength will exhibit spalling as a result of crushing. For the OSJ5 specimen, it is observed that spalling occurs at the beam regions adjacent to the joint and for the OSJ10 specimen, it occurs at the diagonal region within the joint.

Figure 8a shows simulated and observed load deformation response of specimen OSJ5. Good correlation could be observed in the pre-peak region. The simulated load-deformation response was observed not to soften after the observed peak strength was achieved. This anomalous behavior in the simulation could be partly explained due to usage of elastic material model for concrete and also partly due to use of Eligehausen bond model with a maximum bond stress of 2.46 $\sqrt{f_c}$ for all the bond elements, which is clearly an overestimated value to be used for bond stress within the joint. The maximum bond stress value in Eligehausen bond model experiment was obtained by applying tensile force to a reinforcing bar anchored to a concrete block. The stress distribution in a joint is more complex than the simple idealization of anchorage failure in Eligehausen experiment and thereby proper bond stress values are to be estimated, which can probably be done by utilizing principles in contact mechanics. Good correlation in the pre-peak response between observed and simulated load-deformation response could be observed in Figure 8b for OSJ10 specimen but could not be obtained in the post-peak region. Good correlation in the post-peak regime could have been obtained for the OSJ10 specimen, which exhibits joint shear failure, if strength reduction model for concrete is considered instead of the currently used elastic model for compression.



a) OSJ5 specimen b) OSJ10 specimen

Figure 7. Compressive stress distribution in the connection region



a) OSJ5 specimen b) OSJ10 specimen

Figure 8. Load deformation response of the simulated specimen

With an objective to identify the reason for numerical non-convergence, an analysis was carried out for OSJ10 specimen in which a Drucker-Prager model was added in for concrete in compression in place of the elastic model and other material models kept as in the previous simulations. The reason for lack of numerical convergence was identified as the weird response of the concrete tensile region, which unfortunately did not follow the Hordijk tensile softening curve as specified since element convergence could not be attained at the crack. The numerical algorithm in DIANA failed stating that stresses in the main and the crack material are unequal. Similar observations were also made by Wang et al. (1990) who concluded that the problem of convergence for multiple-fixed crack model with plasticity is similar to the numerical convergence problems associated with multi-surface plasticity models. It is well known that a standard radial return-mapping algorithm fails for multi-surface plasticity problems due to presence of more than one plasticity yield surfaces in its vicinity. In this problem too, multiple yield surfaces of cracking and crushing of concrete originate at a local level, thereby resulting in numerical convergence failures. Thereby, even though we could represent one response of concrete, i.e. cracking but both the responses cracking and crushing of concrete could not be captured simultaneously due to numerical unstable algorithms, and lack of better material models in DIANA 9.1. Since it can be realized that these kinds of problems with all the local instabilities is a strong nonlinear problem, the author recommends use of explicit nonlinear finite element softwares such as ABAQUS and/or LS-DYNA for simulation of these complex mechanisms considering all local inelastic mechanisms.

5. CONCLUSIONS

A material model has been suggested in this study which is capable of considering all the local inelastic mechanisms involved in determination of a beam-column joint response. Based on the above study, it has been demonstrated that the current continuum finite element model software, such as DIANA 9.1, with the suggested material model, is capable of representing mechanistic behavior for moderately complex problems such as three point bending, push out response of a reinforcing bar anchored in concrete, bending response of beams and so to name a few. However, it should also be noted that the current capabilities of DIANA using the suggested material model is not capable of representing extremely complex mechanisms such as the exact behavior within the joint region demonstrating all the local inelastic mechanisms within the joint. It has also been demonstrated that if one of the local inelastic mechanisms is simplified then the analysis may converge and global response might be obtained partially. It should also be noted that a large literature exists on number of simulations of concrete structures which have been done considering empirical curves for compressive response of concrete along with degradation rules to account for tension cracks. Even though the global response can be obtained using those empirical equations but the author believes that these are not representative to identify the exact local inelastic mechanisms in complex situations such as that within a connection region. For the bond response, models from first principles of contact mechanics also need to be developed. Within the perspective of commercial finite element softwares, better numerical algorithms needs to be developed which can be utilized to solve situations encountered in multi-surface plasticity models. The author also suggests that these complex local inelastic responses as well as the global response may be obtained through use of explicit nonlinear finite element softwares such as ABAQUS and LS-DYNA. The author is currently involved in simulating the response using ABAQUS.

ACNOWLEDGEMENTS

The author would like to acknowledge his advisor Prof. Laura N. Lowes at University of Washington. The numerical work described here was carried out in part under her guidance as part of the authors PhD research.

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