COMPARISON OF REFINED NUMERICAL MODELING FOR SUBSTANDARD BEAM-COLUMN JOINTS

Özgür YURDAKUL1, Ciro DEL VECCHIO2, Marco DI LUDOVICO3 and Özgür AVŞAR4

ABSTRACT

Advanced numerical modeling for reinforced concrete structures is nowadays available. Nevertheless, reproducing the non-linear behavior of substandard beam-column joints is still a challenging task. Indeed, difficulties arise even more in capturing the contribution of different nonlinear mechanisms (and their mutual effects) on the global cyclic response. This paper deals with the advanced numerical modeling of substandard reinforced concrete beam-column joints designed by reinforcement details non-conforming to current seismic codes. Experimental data from the different testing programs on joint subassemblies characterized by lack of transverse reinforcement in the joint, low strength concrete and plain round/deformed bars were collected from available literature studies. Refined numerical models, which were developed in VecTor2 and ATENA Science finite element method (FEM) software, are investigated in detail to identify the factors affecting the overall response. Two different modeling strategies and their results are closely compared. The advantages-disadvantages, difficulties in implementing the numerical model and their capability to capture the experimental behavior are discussed in detail. A focus on the influence of the modeling assumption on the joint panel shear behavior and the bond-slip of longitudinal reinforcements is provided. Practical suggestions were given to drive the structural engineers in developing efficient and reliable numerical models for substandard RC joints.

Keywords: beam-column joint; shear; substandard; low strength concrete; numerical model

1. INTRODUCTION

Existing reinforced concrete buildings designed with details non-conforming to current seismic codes are vulnerable to seismic actions. Structural deficiencies such as the use of low strength concrete, lack of transverse reinforcement in the joint panel and improper anchor length may lead to devastating brittle failure of the joint under moderate seismic actions. This results in poor energy dissipation and a sudden strength and stiffness degradation that may significantly compromise the seismic response of the structural system. Thus, investigating the response of substandard joints under seismic action has paramount importance. This could be achieved by either experimental tests or numerical analyses on validated numerical models. Many experimental studies were performed to investigate the response of substandard joints (Hassan 2011; Ilki et al. 2011; Del Vecchio et al. 2014; Yurdakul and Avşar 2016; among many others). On the other hand, the recent development in the computer-aided nonlinear
analysis demonstrated that the response of beam-column joints under multiaxial complex stress field can be accurately reproduced (Kulkarni et al. 2008; Haach et al. 2008; Hawileh et al. 2010; Del Vecchio et al. 2016; Najafgholipour et al. 2017). However, further developments are still needed. Indeed, reproducing the nonlinear behavior of substandard beam-column joints in terms of global or local cyclic response is challenging. Difficulties arise more in capturing the failure mode correctly, which in many cases is a combination of different non-linear effects (e.g. shear failure of the joint panel, slip of longitudinal reinforcements, flexure-shear interaction in the members framing into the joint). The current study aims at investigating the seismic response of substandard RC beam-column joints through numerical method. The proposed numerical models were validated against the experimental response of substandard RC beam-column joints which experienced shear failure mode. Those were selected from different experimental programs. The refined numerical models, which were generated in VecTor2 (Wong et al. 2013) and ATENA Science (ATENA Program Documentation, Part 8. 2015) finite element method (FEM) software packages, were implemented to observe the progress of crack developments, its patterns and global hysteretic response. Moreover, the performance of proposed numerical models was discussed in terms of accuracy and effectiveness in capturing the experimental behavior, modelling advantages-disadvantages, difficulties in modelling of substandard joints, theory and constitutive material laws. Finally, practical suggestions were given to drive the structural engineers in developing efficient and reliable numerical models for substandard RC joints.

2. Numerical Study

The nonlinear behavior of substandard beam-column joint is often characterized by the joint panel shear failure, which makes the prediction of the seismic response of this structural element very challenging. Nowadays, refined numerical models are available in user-friendly computer tools which allow reproducing the hysteretic response of RC members characterized by premature shear failure with satisfactory accuracy. In this study, two different modeling strategies developed in FE environment are proposed to reproduce the cyclic response of substandard beam-column joints. The proposed models aim at reproducing the nonlinear response in terms of strength, stiffness, local stresses in the joint panel, crack patterns, as well as the global response. The numerical models were generated in ATENA Science (ATENA Program Documentation, Part 8. 2015) and VecTor2 FEM (Wong et al. 2013) software. Two different models are closely compared in terms of basic assumptions, adopted modeling options and results. The advantages-disadvantages and the modeling difficulties are also discussed.

2.1 Selected experimental tests

The experimental tests used to validate the proposed numerical models are collected from two different testing programs. The selected substandard specimens are similar in terms of concrete compressive strength (low strength concrete) and lack of joint transverse reinforcement. On the other hand, the type of steel bars used as longitudinal reinforcement is different. While the specimen EJ-R was built with plain round bar, deformed bars were used in the specimen T_C3. This significantly affected the seismic response and the crack pattern. The specimens were tested under constant axial load and cyclic displacement imposed at the top column and at the beam end for the EJ-R and T_C3 specimen, respectively. A summary of the material properties, geometry, test setup and other experimental data are reported in Table 1. More detailed information about the tested specimens i.e. EJ-R and T_C3 can be found in Yurdakul and Avşar (2016) and Del Vecchio et al. (2014), respectively.
Table 1. Details of the experimental tests selected for the model validation

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Specimen</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Compressive Strength, $f_c$ (MPa)</td>
<td>EJ_R</td>
</tr>
<tr>
<td>Type of Longitudinal Reinforcement</td>
<td>Plain</td>
</tr>
<tr>
<td>Elastic Modulus, $E_s$ (GPa)</td>
<td>190.9</td>
</tr>
<tr>
<td>Yield Strength, $f_y$ (MPa)</td>
<td>295.5</td>
</tr>
<tr>
<td>Ultimate Strength, $f_u$ (MPa)</td>
<td>437.5</td>
</tr>
<tr>
<td>Ultimate Strain, $\varepsilon_u$ (mm/mm)</td>
<td>0.21</td>
</tr>
<tr>
<td>Beam Cross-Section</td>
<td>250 x 500mm</td>
</tr>
<tr>
<td>Column Cross-Section</td>
<td>250 x 500mm</td>
</tr>
<tr>
<td>Axial Load Ratio ($\Sigma=N/A_g f_c$)</td>
<td>0.1</td>
</tr>
<tr>
<td>Test Setup</td>
<td>Loading on the column</td>
</tr>
<tr>
<td>Loading Protocol</td>
<td>1 repetition per cycle</td>
</tr>
<tr>
<td>Failure Mode</td>
<td>Joint shear</td>
</tr>
</tbody>
</table>

2.2 *ATENA Science numerical modelling*

In order to simulate the actual response of the tested specimens in 3D, the modelling approach, the constitutive law of the materials and its parameters suitable for application to substandard beam-column joints are selected. The geometry of RC members is defined by means of hexahedral element (*CCIsoBrick*). *CC3DNonLinCementitious2*, which is a fracture-plastic concrete model, is used to combine constitutive models for tensile (fracturing) and compressive behavior (*ATENA Program Documentation, Part 1 2014*). This allows reproducing the post-elastic behavior of cracked concrete satisfactorily. The compressive strength of the concrete, $f_c$, was obtained by material characterization tests (Table 1). The Menetrey and Willam (1995) criteria is used to reproduce the evolution of the failure of the concrete. The surface sharpness in Menetrey and Willam (1995) failure surface is controlled by parameter $e \in [0.5,1.0]$. $e = 0.52$, as recommended in the manual of the software, was used in all analyses. The compressive behavior is based on hardening/softening variable, $f_c(\varepsilon_{eq})$, namely the compressive stress as a function of strain. This function, which is characterized by an elliptical hardening and linear softening behavior, was defined according to Van Mier (1986) experimental observations as shown in Figure 1a and b. While the hardening part is strain-based, the linear softening depends on the plastic displacement, $w_p$. The relation between deformation (i.e. plastic displacement in compression, $w_{pl}$, and crack opening in tension $w_c$), $w$, and strain, $\varepsilon$, was obtained by crack band theory, i.e. $w = \varepsilon L_t$ (Bazant and Oh 1983). It should be noted that $L_t$ is the crack band size, which is equal to the finite element size in case of finite element applications (Bazant and Oh 1983). A Gauss function was adopted to describe compressive strength degradation for cracked concrete. The parameters were derived from the experimental data published by Kollegger and Mehlhorn (1988), which also includes Vecchio and Collins (1986) test data (ATENA Program Documentation, Part 1 2014). The strength reduction factor, $c$, in Figure 1c was defined according to Dyngeland (1989) for specimen EJ-R and Kollegger and Mehlhorn (1988) for specimen T_C3.
The tensile behavior of plain concrete is assumed to be elastic in the uncracked regions. The exponential softening relation within the stress in the crack, \( \sigma \), and crack width, \( w \), is defined according to Hordijk (1991) in the cracked region (Figure 2a). The crack opening at full stress, \( w_c \), can be obtained by using \( G_f - w \) relationship. The fracture energy of concrete, \( G_f \), which is the energy required to generate a crack in the unit surface, is the area under Stress–Crack Width curve. A smeared crack approach in a combination of crack band theory by Bazant and Oh (1983) is used to model the behavior of fractured concrete. The inclination of cracks is assumed fixed after the first cracking according to Cervenka (1985) and Darwin and Pecknold (1974). Therefore, the axis of principal stress and the axis of principal strain do not overlap, which creates shear stress on the crack face. A second model using the rotated crack model at the base of the Modified Compression Field Theory (MCFT) by Vecchio and Collins (1986) is also proposed. In this model, the crack direction is computed step-by-step as a function of the stress field. Moreover, Rankine criterion (Rankine 1857) is used for concrete cracking. A reduction in the shear stiffness due to cracking of concrete was employed according to Kolmar (1986). By the increasing strain, the shear modulus is reduced; moreover, the dependence of reduction in the shear stiffness to transverse reinforcement ratio is also taken into account (Figure 2b). It should be noted that different reduction factors were used for beams or columns and joints (i.e. the strength reduction factor for the concrete in the joint panel is the function corresponding to 0% shear reinforcement while it is a function taking into account corresponding shear reinforcement ratio for the beam and column). The mesh size plays a crucial role in accuracy and reliability of the model since its efficacy is generally strongly correlated to mesh optimization. Different mesh sizes were used until the variation in the computed maximum strength is minimized. This resulted in a linear-hexahedral element (i.e. square shape) with 50mm side.

Figure 1. (a) Concrete in compression (b) Compression softening (c) Strength reduction in the cracked concrete (ATENA Program Documentation, Part 1 2014)

The reinforcing steel was described by a bilinear elasto-plastic model with hardening behavior. It was modelled as truss element. Since the plain round bar was used in the specimen EJ-R, the effect of slippage of the reinforcement is also considered by using the CEB-FIP Model Code (2010) model, which considers a nonlinear increase in bond stress with the increasing slip. Then, after a specific value of

Figure 2. (a) Tension softening (b) Shear retention factor (ATENA Program Documentation, Part 1 2014)
0.1mm slip, a constant bond stress was assumed. It should be noted that the slippage of beam reinforcement at beam-ends was intended to prevent by welding of beam reinforcing bar hooks in EJ-R before concreting the specimen. This effect should also be considered in modelling of bond slip behavior in the FEM model. Therefore, slippage at reinforcement ends was restricted at a certain extent. No bond-slip law was employed for specimen T_C3 because of the existence of deformed bars.

The material properties were summarized in Table 2.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>ATENA Science model</th>
<th>VecTor2 model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic Modulus, $E_c$</td>
<td>$4700\sqrt{f_c}$ (ACI 318M-11 2011)</td>
<td>$3320\sqrt{f_c} + 6900$ (ACI 363R-92 1992)</td>
</tr>
<tr>
<td>Tensile Strength, $f_t$</td>
<td>$0.30f_c^{2/3}$ (CEB-FIP Model Code 2010)</td>
<td>$0.33\sqrt{f_c}$ (Vecchio and Collins 1986)</td>
</tr>
<tr>
<td>Tension Softening</td>
<td>Exponential Function (Hordijk 1991)</td>
<td>Linear Function</td>
</tr>
<tr>
<td>Fracture Energy, $G_f$</td>
<td>$73f_{ct}^{0.18}$ N/m (CEB-FIP Model Code 2010)</td>
<td>$0.075kN/m$ (Wong et al. 2013)</td>
</tr>
<tr>
<td>Compressive Strain, $\varepsilon_{co}$</td>
<td>$f_c/E$ (Van Mier 1986)</td>
<td>$1.8 + 0.0075f_c$ (mc) (Wong et al. 2013)</td>
</tr>
<tr>
<td>Compression Softening</td>
<td></td>
<td>Vecchio F. J. (1992)</td>
</tr>
<tr>
<td>Maximum Aggregate Size</td>
<td>20 mm in EJ-R</td>
<td>20 mm in EJ-R</td>
</tr>
<tr>
<td></td>
<td>18 mm in T_C3</td>
<td>18 mm in T_C3</td>
</tr>
<tr>
<td>Average Crack Spacing</td>
<td>Crack Band Approach (Bazant and Oh 1983)</td>
<td>Collins and Mitchell (1997)</td>
</tr>
<tr>
<td>Concrete Compression Pre/Post Peak</td>
<td>Van Mier (1986)</td>
<td>Popovics (1973)/Mander et al. (1988)</td>
</tr>
<tr>
<td>Cracking Criterion</td>
<td>Rankine Failure Criteria (Rankine 1857)</td>
<td>Mohr-Coulomb (Stress)</td>
</tr>
<tr>
<td>Hysteretic Response</td>
<td>Unloading to origin</td>
<td>Palermo and Vecchio (2003)</td>
</tr>
</tbody>
</table>

### 2.3 VecTor2 numerical modelling

VecTor2 (Wong et al. 2013) is a program based on the MCFT/DSFM (Vecchio and Collins 1986) for
nonlinear finite element (FEM) analysis of reinforced concrete 2D structures which allows to assess the nonlinear response of shear critical RC structures (including strength and stiffness degradation, post-peak behavior and failure mode). The VecTor2 model was developed by using a pre-processor unit FormWorks (Wong et al. 2013) that simplifies the meshing and the input of the model parameters. Similar to other continuum elements, mesh size plays an important role in computational efficiency and accuracy. A mesh size in the range of about 25 mm and approximately square elements have been adopted as suggested in the related studies (Wong et al. 2013; Del Vecchio et al. 2015, 2016). The software can accommodate only 2D elements. A specific thickness complying with the experimental data has been set for all members. Longitudinal reinforcements are modelled with truss elements. Link elements were adopted to model the bond-slip behavior using the Embedded deformed bar option available in the software option (Wong et al. 2013). To account for the stress concentration at the anchorages of longitudinal beam bars, the Hooked bar option has been adopted for the link element in correspondence of reinforcements ends. Transverse reinforcements in the beam and columns were modelled as smeared reinforcements with appropriate in-plane (ρt) and out-of-plane (ρc) average ratios. The concrete cover was modelled as unconfined concrete. The material properties are defined in accordance with the material tests (Table 1). All analyses are performed with the use of basic default material behavior models and analysis options (see Table 2 for the details). The concrete constitutive model by Popovics (1973) and Mander et al. (1988) is adapted to reproduce the concrete compressive behavior. The hysteretic model of the concrete suggested by Palermo and Vecchio (2003) is used in this study to reproduce the loading/unloading behavior and the cyclic-load induced damage sustained by the concrete. To account for the effect of beam anchorages on the shear response of the joint panel Priestley (1997), the maximum lateral pressure provided by the bar bents has been computed and an equivalent amount of transverse reinforcement offering the same lateral pressure has been inserted in the model (Del Vecchio et al. 2016).

3. RESULTS AND DISCUSSION

In following paragraphs, a close comparison between the results of the proposed numerical models and the experimental results is proposed with reference to the specimens EJ_R (Yurdakul and Avşar 2016) and T_C3 (Del Vecchio et al. 2014). The results are discussed with reference to the hysteretic response and crack pattern focusing on the peculiarities of the models.

3.1 Hysteric Response by ATENA Science

The cyclic response in the ATENA models is obtained imposing a displacement history (reverse cyclic displacement) at the top column (for EJ_R specimen) or at the beam tip (for T_C3 specimen) along with a constant axial load on the columns. The restraints in the numerical models are set compliant with the experimental test setup. In the numerical models, different modelling approaches were compared. The stirrups in the column and beam were modelled in one of the models (e.g. numerical model CN_CC1) while the rest of the models were considered in the confined concrete model available in ATENA Science software (ATENA Program Documentation, Part 8. 2015). Moreover, the effect of different crack models was also investigated (e.g. CN_1 fixed crack theory, CN_2 rotated crack theory).

3.1.1 Specimen EJ-R

The outcomes of the FEM model in terms of hysteretic behavior (V-Drift) and crack pattern are presented in Figure 3a-d and Figure 4, respectively. These figures also report the comparison of models with available experimental results. In all models, i.e. CN_1, CN_2 and CN_CC1, the initial stiffness is higher than the one measured experimentally. The differences are significant in the positive loading direction while the gap reduces to the 20% in the negative loading direction. The peak and post-peak response including pinching effects and the sudden strength, stiffness degradation matched with the experimental response with a satisfactory agreement in the model with fixed crack theory and confined concrete model, i.e. CN_1
While the behavior in the initial cycles was very similar to the model with confined concrete (CN_1), the model with stirrups (CN_CC1) underestimated the capacity after maximum load since the cracks took place between the stirrups. A rather gradual stiffness degradation was also monitored until the last loading step. At drift ratio of 4%, the model with a stirrup (CN_CC1) and the model with confined concrete (CN_1) have reached almost the same load value. This phenomenon might be attributed to the concrete crushing in the joint core. As it became more critical in the subsequent drift ratios, the overall response was dominated by the crushing properties.

Less accurate results are provided by the model CN_2. Indeed, the model with rotated crack theory overestimated the column shear strength at all the steps of the analysis and especially for large drift demand where a significant strength and stiffness degradation was observed during the experimental tests.

![Model assumptions](image)

**Model assumptions**

- **CN_1**
  - **Confinement**: Confined concrete model
  - **Crack Model**: Fixed crack theory

- **CN_CC1**
  - **Confinement**: Stirrups
  - **Crack Model**: Fixed crack theory

- **CN_2**
  - **Confinement**: Confined concrete model
  - **Crack Model**: Rotated crack theory

Figure 3. EJ-R vs. (a) CN_1 (b) CN_CC1 (c) CN_2 (d) Model assumptions for EJ-R

In terms of crack pattern, all models have successfully captured the concentration of cracks in the joint panel. The results of CN_1 are presented in Figure 4a-c along with the experimental observation for increasing drift demand. In the initial loading steps, the cracks developed in the joint panel and at the beam-to-joint interface according to the experimental test. In the following cycles, the existing cracks in the beam close and the shear cracks in the joint panel widen (Figure 4b and c). The severe damage due to spalling of concrete cover in the joint is well captured by the model for large drift demand. This
is due to the pushing force of the concrete core as a result of its shear fracturing (Figure 4c).

![Figure 4](image)

**Figure 4.** Comparison of experimental and numerical crack pattern demand for the specimen EJ_R at (a) first joint cracking (b) 2% drift ratio (c) 4% drift ratio

### 3.1.2 Specimen T_C3

Different modelling approaches were employed in T_C3. The compressive strength was reduced in cracked concrete as mentioned earlier. The strength reduction factor, \( c \), in Figure 1c was proposed as 0.80 according to Dyngeland (1989) and 0.45 according to Kollegger and Mehlhorn (1988). In the calibration of the numerical model, the reduction factor of 0.45 was used for numerical analyses of T_C3 while 0.80 in the corresponding analyses of EJ-R. In case of very low concrete strength such as 8-10MPa, the value of 0.45 results in numerical instability and low capacity as very low strength is reduced more than half of its strength. Thus, the reduction factor of 0.80 was selected in EJ-R. For T_C3, the reduction factor of 0.45 yields more accurate results in terms of capacity without any divergence in solution. It is because the concrete strength in T_C3 is rather higher than EJ_R.

Another parameter which is different in the analysis of specimen EJ-R and T_C3 is the critical displacement value in compression, \( w_d \) (Figure 1b). This value affects the softening function of concrete in compression. As a result, the lateral load in the cycles with more than 1% drift ratio where softening of concrete starts (especially cycles after post-peak) can be affected by the critical displacement value. The parameter \( w_d \) for the selected specimens is set according to available literature (Van Mier 1986; Duran et al. 2017) as a function of the concrete compressive strength.

The third difference in the numerical model is the loading scheme. While the specimen T_C3 was loaded from beam tip with 3 repetitions per cycle, EJ_R was loaded from column tip with 1 repetition per cycle.
Finally, a bond-slip law according to CEB-FIP Model Code (2010) was defined for the specimen EJ_R constructed with plain bar, no bond slip law was employed for T_C3 due to the existence of deformed bars. The model developed in ATENA Science (ATENA Program Documentation, Part 8. 2015) well captures both initial stiffness and peak load of the specimen T_C3, while the pinching effect at large drift demand does not match well with experimental results (Figure 5a). This results in higher energy dissipation in the numerical models with rotated crack theory (C3_1). A possible explanation for this mismatching might be large residual displacement in the longitudinal steel reinforcements due to the local yielding as a consequence of the strain compatibility between steel and concrete across a large shear crack in the joint panel. Indeed, a large strain in reinforcing bar is observed across the crack. Further simulations are needed to improve the numerical predictions including proper bond-slip models for deformed bars. In case of fixed crack theory (C3_2 model), the model predicts a sudden strength and stiffness degradation at a drift about the 1% (Figure 5b). This could be related to the sensitivity of the fixed crack model to the number of repetition per each cycle.

![Figure 5. T_C3 vs. (a) C3_1 (Rotated crack theory) (b) C3_2 (Fixed Crack Theory)](image)

3.2 Hysteric Response by VecTor2

The cyclic response of EJ_R and T_C3 specimens is reproduced by using the VecTor2 model imposing the experimental recorded displacement history at the beam tip and top column, respectively. A constant axial load is applied at the top of the column about 0.1A_f and 0.2A_f, respectively for EJ_R and TC_3 specimen. The results of the numerical models are reported in the following and compared with the experimental response in terms of hysteretic behavior and crack pattern.

3.2.1 Specimen EJ_R

The VecTor 2 model well captures the experimental response in terms of peak strength, crack pattern and failure mode (see Figure 6). The joint shear failure significantly affected the subassembly response resulting in a limited peak strength (significantly lower than the strength corresponding to the yielding of longitudinal reinforcement). The significant joint cracking resulted in a marked strength and stiffness degradation for interstorey drift higher than 1.5%. This behavior is well captured by the VecTor2 model which is also able to account for the significant slip of smooth longitudinal reinforcement. It is worth mentioning that the adapted bond-slip law was according to Eligehausen et al. (1983) while its parameters are obtained from available experimental tests on smooth reinforcement presented by Fabbrocino et al. (2005). The numerical response at large drift demand (i.e. 3% and 4%) is less accurate respect to the ATENA Science model due to convergence issue related to the interaction between significant shear cracking and the slip of longitudinal reinforcements.
3.2.2 Specimen T_C3

The VecTor2 model well captures the experimental response of T_C3 specimen both in terms of hysteresis and crack pattern (Figure 7). The numerical prediction well matches all significant parameters which are relevant for the seismic assessment, including initial stiffness, peak strength, strength and stiffness degradation, pinching effects and energy dissipation. Furthermore, a crack pattern very similar to the experimental one is also predicted. In particular, the first cracking in the joint panel (depicted in Figure 7) occurs for a column shear of about 33kN (drift 0.5%), significantly lower than the maximum strength. At this step, diffused hairline cracks appear in both directions. At the same step, a flexural crack appears on the beam in the section of the maximum bending moment.

The joint peak strength is characterized by deep and large diagonal cracks in the order of millimeters. As seen in Figure 7a, a corner-to-corner diagonal crack appears in the joint panel for the positive loads.

Reverse cyclic actions produce a change in the crack orientation; however, due to the strong nonlinear
behavior of the cracked concrete, the opposite diagonal cracks are not completely closed. The crack pattern at the joint panel failure, after which a significant drop in the shear strength can be observed, shows marked cracks in both directions and the spalling of concrete cover. Due to the severe damage in the joint panel, large shear cracks, in the order of centimeters, can be observed in both directions.

4. CONCLUSION

This study investigates the response of substandard RC beam-column joints with lack of stirrups in the joint panel, non-seismic reinforcement details and low strength concrete by numerical methods. In order to compare the numerical predictions, two experimental tests were selected from available literature studies. Two different modeling strategies by using two different FEM environment were developed. The modeling strategy and adopted constitutive laws are discussed along with advantages-disadvantages, the difficulties and the capability of each model to capture the experimental response. Based on the results of this study, the following conclusions can be drawn.

- The model developed in ATENA Science (ATENA Program Documentation, Part 8. 2015) well matches with the experimental results of EJ_R joint. In particular, CN_1 based on the fixed crack theory and confined concrete model provides the best matching in terms of hysteretic response.
- The confinement model significantly affects the numerical response. When the stirrups are explicitly modelled, the capacity is significantly lower than the model where the confinement effects of the stirrups are considered in the definition of concrete characteristics.
- For the T_C3 joint, the ATENA Science model based on the rotated crack theory well captures the experimental response except for the large drift demand. The pinching effect is not well represented because of strain compatibility issue between locally yielded reinforcing steel in a large shear crack in the joint panel.
- On the other hand, the VecTor2 analytical model provides a good match with experimental results of EJ_R and T_C3 joint with respect to all significant parameters in the seismic assessment including initial stiffness, peak strength, strength and stiffness degradation, pinching effects and energy dissipation. Furthermore, the crack pattern at different stress level can be accurately predicted.
- The VecTor2 model resulted less accurate in reproducing the cyclic response of EJ_R specimen at large drift demand due to the interaction of the shear damage and reinforcement slip in the joint panel.
- When the performance of VecTor2 (Wong et al. 2013) and ATENA Science (ATENA Program Documentation, Part 8. 2015) are compared, no significant difference was found before peak response. Significant differences can be observed after the post-peak behavior.

5. ACKNOWLEDGMENTS

This study has been accomplished by the support of the Educational and Research Centre in Transport, University of Pardubice and Anadolu University Scientific Research Projects Commission under grant No. 1210F169. This study was performed in the framework of PE 2016-2018 joint program DPC-ReLUIS.

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