Analytical model of retrofitted reinforced concrete beam column joints after experiencing severe damage from earthquake load simulation using SAP2000

N Gosal\textsuperscript{1}, I Imran\textsuperscript{2}, and M Riyansyah\textsuperscript{2}

\textsuperscript{1}Post Graduate Program of Faculty of Civil Engineering, Bandung Institute of Technology, 40132 Bandung, Indonesia
\textsuperscript{2}Structural Engineering Department Faculty of Civil Engineering, Bandung Institute of Technology, 40132 Bandung, Indonesia

*nelsongosal@hotmail.com

Abstract. An earthquake resistant structure should be able to perform as expected when it’s struck by an earthquake such as it may experience slight damage and should be able to be repaired to its original state when struck by a design level earthquake. However, there are still debates whether reparation should also be viable when structures are experiencing severe damage. There are still a little to no studies about repairing severely damaged structures, therefore a study was made to see how severely damaged structures behaved after being repaired. Two beam column joints are used as specimens, which were severely damaged by an earthquake simulation before getting repaired. Both their initial and repaired condition are then modeled analytically using SAP2000. Their behaviors such as plastic hinges, joint’s shear deformation, bond slip, and rebars’ residual strain and stress due to previous loading are also implemented to the model. Although the lateral strength of the specimens are similar to their analytical model, the other behavior, especially their stiffness is not similar, therefore needing another parameter implemented in the model.

1. Introduction

Nowadays, earthquake resistant structures are designed to have certain performance during certain level of earthquake. Structures are expected to experience no damage when they are struck by frequent earthquakes. When a design level earthquake strikes, structures may be slightly damaged but should be able to be repaired. However, there are still a little to no studies whether repairing severely damaged structures is viable. Therefore, previous study by Nelson et al. (2018)\textsuperscript{[1]} have been analyzing nonlinear behaviors of reinforced concrete structures that were being repaired after experiencing severe damage from an earthquake. The study shows that by repairing only the beam section of the structure can cause lower stiffness due to the rebars’ bond degradation inside the joint and even the cracks forming on the joint. Knowing this, an analytical model was made using SAP2000 to analyze the behavior of the beam column joints, including both the bond slip and the joint’s shear deformation.

Two beam column joints that are used as specimens by Kurniawan et al. (2018)\textsuperscript{[2]}, are shown in Figure 1 below. Each specimen is different than the other by the longitudinal rebars implemented, using D16 and D19 – which are now being named as SD16-0 and SD19-0 respectively. As can be seen from
Figure 2, both specimens are then being setup in PRI ITB and subjected to a cyclic loading module defined by ACI 374.2-13 up to drift level of 5%. The damaged specimens are then being repaired by replacing the damaged concrete on the beam with new stronger concrete without replacing the longitudinal rebars. Repaired specimens, which is now named SD16-R and SD19-R, are then being tested using the same setup and loading module as before. An analytical model was also made for each specimen as their experimental results are then being compared with their analytical results.

![Diagram of Specimen Setup and Cyclic Module](image)

*Figure 1. Specimens' Details (Kurniawan et. al.), (a) Elevation View of the Specimen, (b) Beam and Column Section for SD19 (Top) and SD16 (Bottom)*

*Figure 2. Cyclic Loading Preparation (a) Specimen Setup, (b) Cyclic Module*

2. **Modelling of Beam Column Joint Component**

Beam column joints in SAP are usually modeled by joining column frames with beam frames, assuming joint elements are rigid. However, failures such as bond slips and joint shear failure are not considered using this method. Therefore, this study also considers the possibility of such failures so that more accurate results can be obtained. Lowes et al. [3] proposed a reinforced concrete beam column joint model where the joint is modeled as a shear panel with several springs to connect it with beam and column frames as shown in Figure 3.
2.1. Shear Panel Model
Shear panel model is used to evaluate the deformation of the specimens through joint shear failure. Therefore, it should have a shear to deformation relationship of a beam column joint. Sung et al. [4] had proposed a modified beam column joint model from Youssef and Ghobarah [5] where its performance is controlled by its shear strength. Such model can be seen in Figure 4.

![Figure 4. Beam Column Joints Model Proposed by Sung et al.](image)

In this model, shear behavior of beam column joints is modified as a diagonal strut and tie model, where its force – deformation relationship is obtained through these equations:

\[
P_{\text{strut}} = \frac{V}{2\cos\theta} \quad (1)
\]

\[
\delta_{\text{strut}} = \delta_{\text{shear}} \times \cos\theta \quad (2)
\]

where \(P_{\text{strut}}\) is the axial force acting on the diagonal struts, \(V\) is the shear force acting on the beam column joints, \(\delta_{\text{strut}}\) is the axial deformation of the diagonal struts, \(\delta_{\text{shear}}\) is the shear deformation of the beam column joints, and \(\theta\) is the angle between the diagonal strut and the horizontal line.

Referring to ASCE 41-17 [6], shear – deformation relation of beam column joints is defined where the nominal shear strength of a beam column joints can be acquired through this equation:

\[
V_n = \lambda y \sqrt{f'_{c}} A_j \quad (3)
\]

where \(V_n\) is the nominal shear strength of the joint, \(\lambda\) is concrete coefficient, \(\gamma\) is a constant that depends on the joint’s confinement and its position (interior joints or exterior joints), \(f'_{c}\) is the strength of the concrete, and \(A_j\) is the effective cross section area of the joints. Nonlinear shear – deformation relationship, as shown in Figure 5, for joints is also made using the curve defined in ASCE 41-17 where
a and b are rotation angles and c is a residual strength ratio which values are also defined in ASCE 41-17.

![Figure 5. Shear - Deformation Relation](image)

Sung et al. then proposed new behavior model for the diagonal struts using the shear – deformation relation. Using the initial stiffness of 0.4 $E_c A_g$, the new behavior model can be seen in Figure 6.

![Figure 6. Axial Hinge Behavior Proposed by Sung et al.](image)

Diagonal struts are then modeled using link element with non-linear axial behavior that is calculated through this method.

### 2.2. Bar Slip Model

As can be seen from Figure 3, bar slips are defined by bar slip spring which have zero length. Lowes et al. [3] had proposed the parameters of the spring using these equations:

$$ slip = \frac{2\tau_E}{E} \frac{l^2_s}{d_b} \text{ for } f_s < f_y $$

(4)

$$ slip = \frac{2\tau_E}{E} \frac{l^2_e}{d_b} + \frac{f_y}{E} l_y + \frac{2\tau_y}{E_h} \frac{l^2_y}{d_b} \text{ for } f_s \geq f_y $$

(5)

From equation (4) and (5), $f_s$ is the bar stress at the joint perimeter, $f_y$ is the bar’s yield strength, $E$ is the bar’s elastic modulus, $E_h$ is the bar’s strain hardening modulus while assuming it has a bilinear stress – strain relation, while $l_s$, $l_e$, and $l_y$ is the length along the bar which its stress is less than, equals to, and more than the yield stress respectively. These lengths are then acquired from the following equations:
\[ l_{fs} = \frac{f_s}{\tau_{ET}} \frac{A_b}{\pi d_b} \]  
\[ l_e = \frac{f_y}{\tau_{ET}} \frac{A_b}{\pi d_b} \]  
\[ l_y = \frac{f_s - f_y}{\tau_{YT}} \frac{A_b}{\pi d_b} \]

where \( \tau \) is the average bond strength of the joint, and is defined in Table 1.

**Table 1. Average Bond Stress Defined by Lowes et al.**

| Bar stress, \( f_s \) | Average bond strength (MPa) |
|------------------------|-----------------------------|
| Tension, \( f_s < f_y \) | \( \tau_{ET} = 1.8\sqrt{f_c} \) |
| Tension, \( f_s > f_y \) | \( \tau_{YT} = 0.4\sqrt{f_c} \) to \( 0.05\sqrt{f_c} \) |
| Compression, \( f_s < f_y \) | \( \tau_{EC} = 2.2\sqrt{f_c} \) |
| Compression, \( f_s > f_y \) | \( \tau_{YC} = 3.6\sqrt{f_c} \) |

While section’s tension stress is mostly carried by the reinforcement bars, the compressive stress is distributed between the reinforcement bars and the concrete. Therefore, in order to make more accurate model of the stress – slip relation, compressive stress that works on the section is also defined by Lowes et al. [3] using the following equation:

\[ C = f_s' A_s' \left( 1 + \left( \frac{0.85 f_c' w}{E_s A_s'} \left( \frac{2(1 - j)}{0.003 \beta \left( 1 - \frac{d'}{d} \frac{\beta}{2(1 - j)} \right)} \right) \right) \]  
\[ \left( 1 + \left( \frac{0.85 f_c' w}{E_s A_s'} \left( \frac{2(1 - j)}{0.003 \beta \left( 1 - \frac{d'}{d} \frac{\beta}{2(1 - j)} \right)} \right) \right) \]

where \( f_c' \) is the nominal strength of the concrete, \( f_s' \) is the compression bar stress, \( A_s' \) is the compression bar area, \( E_s \) is the bar’s elastic modulus, \( w \) is the width of the section, \( d \) and \( d' \) is the depth of the section to the tension reinforcement and compression reinforcement respectively, and \( j \) is a constant that is taken 0.85 for beams and 0.75 for columns.

2.3. *Residual Strain and Stress Model*

Reinforcement bar’s strain and stress relation is modeled using the model proposed by Menegotto and Pinto [6]. Constants such as the curvature of the curve, \( R \), and the post yield slope, \( b \), is assumed as the model is then compared with the stress – strain relation obtained from experimental data that is done by Kurniawan et al. [2] as shown in Figure 7.
Due to lack of reinforcement bars’ strain data from the first experiment, an analytical approach was done to assume the reinforcement bars’ residual strain. This was done by analyzing the reinforcement bars’ last strain during the 5% drift due to the first cyclic test is up to 5% drift. After obtaining the 5% drift strain, all reinforcement bars are assumed to unload until they have no residual stress. The damaged tilted specimens are then straightened out so that the beam elements are perpendicular again to the column elements. Strain rate of the beams’ reinforcement bars are recorded during this process, and will be used to assume the residual stress and strain of the repaired specimens’ reinforcement bars. New reinforcement bars’ stress-strain relations are then obtained according to each of their residual stress and strain. Those relations are then modeled as a bilinear curve with parameters as follows:

\[
\sigma'_y = \sigma_y - \sigma_{res} \tag{10}
\]

\[
\sigma'_u = \sigma_u - \sigma_{res} \tag{11}
\]

\[
\epsilon'_y = \frac{\sigma'_y}{E} \tag{12}
\]

\[
\epsilon'_u = \epsilon_u - \epsilon_{res} \tag{13}
\]

where \(\sigma'_y, \sigma'_u, \epsilon'_y, \) and \(\epsilon'_u\) are reinforcement bars’ modified yield stress, ultimate stress, yield strain, and ultimate strain respectively, \(\sigma_y, \sigma_u, \) and \(\epsilon_u\) are reinforcement bars’ initial yield stress, ultimate stress, and ultimate strain respectively, while \(\sigma_{res}\) and \(\epsilon_{res}\) are reinforcement bars’ residual stress and strain respectively.

3. Modelling of Beam Column Joint Element

![Figure 8. Analytical Model in SAP2000](image)
A beam column joint is made on SAP2000 using models implemented from before, as shown in Figure 8. In this case, bond slips are modeled using small nonlinear links as SAP2000 can’t implement nonlinear zero-length spring. Bond slip links and shear interface links are only implemented on beams’ interface, assuming column’s interface is rigid and doesn’t affect the performance. Shear interface on beam is also modeled as a rigid link, assuming it doesn’t affect both specimens’ performance.

3.1. Initial Specimens
Two specimens were tested by Kurniawan et al. [2] as specified in Figure 1 using cyclic displacement control loading, until each of them reached the drift level of 5%. Both backbone curves of the test are then acquired and compared with their analytical model counterparts. A pushover analysis is then done to observe both models’ force – deformation relation.

![Observed and Simulated Response of Kurniawan et al. (2018) Beam Column Joint Specimens (a) SD16-0 and (b) SD19-0](image)

3.2. Repaired Specimens
Both specimens are then repaired by replacing the damaged concrete on the beams with a new, stronger one without replacing their longitudinal rebars. The reparation is only done to both specimens’ beams due to there are only small hairline cracks forming on both specimens’ columns and joints. The repaired specimens are then retested and compared with their initial counterparts. Due to the residual stress and strain in both beams’ longitudinal rebars, different stress – strain diagrams are implemented for the longitudinal rebars, using from Equation 10 to Equation 13. Shear panel stiffness is also reduced to 0.7 of its initial stiffness, assuming that cracks forming on the specimens caused reduction in the joint’s stiffness.
Figure 10. Observed and Simulated Response of Repaired Specimen (a) SD16-R and (b) SD19-0

4. Model Evaluation

Figure 9 shows the comparison of a pushover analysis with the backbone of each tested specimen. It seems that both specimens’ peak lateral forces are almost the same with the analytical model, with a difference ranging from 8% to 10%. This proves that the analytical model can predict the lateral strength of the specimens although its initial stiffness isn’t the same. Events such as spalling of both beams, and also significant yield of the specimens are also recorded and compared. From these comparisons, it seems that the analytical model may also predict lateral force when spalling event occurs although it may not accurately predict the force needed for the specimens to reach their significant yield.

Figure 10 shows the comparison of the repaired counterparts of the specimens. From the analytical model, reducing shear panel stiffness to 0.7 of its initial stiffness may not significantly affect the stiffness of the model. Furthermore, an assumption to the residual stress and strain was made due to lack of data from the previous experiment, causing a difference in the specimens’ lateral strength ranging from 5% to 28%. However, the analytical model due to residual stress and strain can predict the changes in lateral strength including the increase strength of SD16-R during the negative loading as can be seen in Figure 11 and Figure 12. An effort to match the analytical results with the experimental results were then made. This effort was made assuming that the integrity of the joint is reduced due to the damages that are left untreated. On the model’s shear panel, the horizontal rigid bars are replaced with a concrete beam section which has the same width and half of the height of the specimen’s beams while the vertical rigid bars are replaced with a concrete column section which also has the same depth and half of the width of the specimen’s column. The parameter that is refined is the rigidity of those bars, which is modeled by reducing their EA and EI by certain constant. As can be seen in Figure 13, the results show that by reducing the rigidity can also cause reduction in specimen’s stiffness, therefore implementing this parameter may predict the stiffness of a damaged joint due to its loss of integrity. It is also can be seen that reducing the rigidity up to 0.2 of its initial state can represent the specimen’s stiffness more.
**Figure 11.** Comparison Between SD16-0 and SD16-R (a) Analytical Comparison and (b) Experimental comparison

**Figure 12.** Comparison Between SD19-0 and SD19-R (a) Analytical Comparison and (b) Experimental comparison

**Figure 13.** Analytical Response with Joint Rigidity Reduction a) SD16-R and b) SD19-R
5. Conclusions
An analytical model was made based on previous studies to be compared to their experimental counterparts. Based on the comparison that was made, more parameters other than the joint’s shear deformation and bond slip spring must be implemented to get more accurate results. Nevertheless, it seems that the analytical model may predict the specimens’ lateral strengths. Other conclusions are as follows:

- Analytical model that was made can predict the lateral strength of the specimen with an error ranging from 8% to 10%
- Implementing residual strain and residual stress by modifying the rebars’ properties can predict the changes in specimens’ lateral stiffness, although it may not predict the strength more accurately due to assumption was made on the residual strain.
- Reducing shear panel stiffness to 0.7 of its initial stiffness may not significantly reduce the stiffness of the model.
- Reducing the rigidity of the shear panel may represent the damaged joint due to its loss of integrity. Reduction up to 0.2 of its initial state may predict the behavior more accurately.

6. References
[1] Nelson G, Iswandi I and Muhammad R 2018 Studies on nonlinear behavior of retrofitted reinforced concrete beam column joints after experiencing severe damage from earthquake load simulation ConCERN-2 accepted
[2] Kurniawan S, Iswandi I, Maulana D, Aris A and Muhammad R 2018 Application of high strength reinforcing bars in earthquake resistant structure elements 4th International Conference in Rehabilitation and Maintenance in Civil Engineering
[3] Laura N L and Arash A 2003 Modeling reinforced-concrete beam-column joints subjected to cyclic loading J. Struct. Eng. 129(12) 1686
[4] Sung Y C, Lin T K, Hsiao C C and Lai M C 2013 Pushover analysis of reinforced concrete frames considering shear failure at beam-column joints Earthq. Eng. & Eng. Vib. 12 373
[5] Youssef M and Ghobarah A 2001 Modelling of RC beam-column joints and structural walls Journal of Earthquake Engineering 5(1) 93
[6] Menegotto, M. and Pinto, P.E. 1973 Method of analysis of cyclically loaded RC plane frames including changes in geometry and non-elastic behavior of elements under normal force and bending Preliminary Report IABSE 13 15
[7] ASCE 2017 Seismic Evaluation and Retrofit of Existing Buildings (ASCE/SEI 41-17) (Virginia: American Society of Civil Engineers)