Research Article
Cyclic Behavior of Reinforced Concrete Slab-Column Connection Using Numerical Simulation

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Received 23 March 2022; Revised 22 April 2022; Accepted 5 May 2022; Published 25 May 2022

Nonlinear finite element (FE) simulation was employed to look into the behavior of flat slab-column connections exposed to lateral cyclic loading. The study focused on the effect of drop panel and concrete grade by taking into consideration the element size sensitivity on the response of the joint. The study considered twelve specimens for the investigation. Four concrete grades having a magnitude of 47.7, 60, 75, and 90 MPa were selected. Three element sizes of 30 mm, 40 mm, and 60 mm were undertaken to access the mesh size sensitivity of the outputs. Six specimens are with drop panels, and others are without drop panels. The concrete-damaged plasticity model has been used to model the specimens. Reduced eight noded bricks (C3D8R) and two noded trusses (T3D2) elements have been used for modeling concrete and steel, respectively. Cracking pattern, slab vertical deflection, strain distribution, ductility, load-drift ratio, cracking load capacity, yielding load capacity, and ultimate load capacity are the main outputs of the FE simulation. The study confirmed a considerable increase in load-carrying capacity and a decrease in ductility as the concrete grade increased. In another way, the load-carrying capacity and ductility were slightly increased and decreased, respectively, as the slab-column connection is provided with a drop panel. As element size decreases, the result obtained becomes more realistic.

1. Introduction

Reinforced concrete flat slab framed system typically consists of a floor slab directly supported by a column without horizontally spanning beams. This allows the construction of building frames with reduced height, which in turn results in easy use of formwork during construction. Generally, this all makes the flat slab framed structure the most economical relative to other framed systems. Since there is no beam in a flat slab framed building, attention is needed during the analysis and design of column strips to control flexures, punching shear, and combination failure, especially in seismic areas. If properly designed for all possible load combinations, a flat slab-column connection can perform satisfactorily when subjected to lateral cyclic loading.

Some past studies investigated the behavior of flat slab-column connection under both monotonic and cyclic loading, using numerical and experimental methods. Nonlinear FE analysis has been conducted to study the performance of connections subjected to seismic loading without providing shear reinforcement [1]. In the study, cyclic degradation which reduces connection stiffness, unbalanced moment capacity, and ductility has been investigated. Additionally, local behaviors such as joint rotations, crack patterns, and flexural rebar strain were also accessed accurately. The FE simulation showed a good agreement with an experimental result. Flexures and punching failure are common in flat slab-column connections of framed buildings [2]. The paper proposed a novel numerical strategy for modeling punching failure, which limits the force transfer from slabs to supporting columns, using FE code ADAPTIC. The methodology showed a capability of accurately capturing both the load-rotation response and the failure point of all proposed specimens.
An experimental investigation has been also conducted to study the effect of discontinuous slab reinforcements on the performance of slab-column connection under lateral cyclic loading [3]. The study testified that the consequences are significantly more severe when compared to a slab with continuous reinforcement. The slab-column connection has lower stiffness, ductility, and drift capacity, especially under joint application of gravity and cyclic loading, and hence, the modified shear reinforcement detail is required [4]. An experimental assessment has also been conducted to study the behavior of nonmonolithic slab-column connections by using four connection techniques: shear dowels, welding dowels, steel brackets, and concrete key-lock [5]. Full-scale testing of flat slab structure is also reported, by considering different longitudinal and shear reinforcements under seismic load at service and ultimate action [6]. In another way, the effect of varying gravity load and reinforcement ratio has been studied experimentally to access the behavior of the connection under both monotonic and cyclic loading [7]. Specifically, a number of studies have been conducted on the way of improving the punching shear capacity of the joint by applying different techniques [8–15].

In addition, the behavior of slab-column connections was tested under a reverse cyclic moment with different levels of gravity shear load [16]. The study confirmed that, as the gravity shear ratio increased, the ductility of the joint was reduced, and as the joint was subjected to cyclic loading, the shear capacity of the connection is reduced when compared to monotonic loading. The result of the test was compared with the failure load predicted with code provisions of ACI318-14 and EC2-2004. The slab-column connections with partially debonded reinforcement were also tested under lateral loading [17]. The investigator testified that the joint with partially debonded reinforcement showed more lateral drift capacity as compared to the joint without partially debonded reinforcement. In other cases, numerical simulation was undertaken to study the dynamic performance of slab-column joint under blast loading [18]. The accuracy of numerical analysis was improved by taking a reasonable simulation method, material model, and mesh size. A model with drop panel and high confinement showed better performance under blast loading.

Furthermore, in the past few decades, an experimental and analytical investigation has been conducted on the performance of slab-column connection with normal concrete strength and considering with and without opening [19–23]. The concrete damaged plasticity model was used for modeling the joint to conduct a numerical investigation. From the authors’ knowledge and based literature review, the study concerning high-strength concrete is very rare or limited. However, the demand for using high concrete strength is increasing in practical building construction. As an example, recently, a concrete compressive strength approaching 138 MPa has been used in cast-in-place of tall-building structures [24]. Therefore, an extensive performance investigation is needed in detail to control any possible failure of slab-column connection with high-strength concrete specifically in high seismic areas. Even if the FE simulation is difficult and time-consuming work, it is the most economical method and accurate that every interested individual can be accessible at any time without limitation.

This study endeavors the investigation of the effect of high-strength concrete grades and drop panels by considering element size sensitivity, on the response of flat slab-column connection under cyclic loading using advanced FE simulation. It focuses on the concrete strength degradation, slab deflection, cracking pattern, strain distribution, and ductility of the connection under simultaneous application of gravity shear load on the floor slab and cyclic lateral loading at the top end of the column. A new methodology has been employed to model concrete damage parameters [25]. The slab-column connection needs a critical consideration to be safe under cyclic seismic loading, which needs a detailed investigation of all possible load combinations and failures. Therefore, this study provides valuable data on connection performance including failure mechanism and zone of development at different loading stages. The study provides detailed information that is taken into consideration on how to analyze and design the joint with high-strength concrete under cyclic loading.

2. Description of the Finite Element Models

The study employed a FE scheme that comprises a solid element for the concrete and a truss element for the steel reinforcement. The compressive and tensile stress-strain response of the concrete is simulated using a new methodology of the uniaxial constitutive law proposed by [25]. Concrete is modeled using 8-node linear solid (brick) elements with reduced integration and hourglass control (C3D8R) available in Abaqus FE simulation software. In another way, steel reinforcement is meshed using two-node linear truss elements (T3D2). The stress-strain response of steel reinforcement is described by a uniaxial bilinear material model (elastoplastic) that accounts for strain hardening [26].

2.1. Concrete Material Idealization. The uniaxial concrete stress-strain law is selected for this particular study because the uniaxial constitutive relation model has the capability to predict the tension stiffening, cracking opening, and stiffness degradation of concrete structure under cyclic loading [27]. The mathematical model presented in equations (1) through (10) for the uniaxial stress-strain is taken from [25]. The uniaxial compressive and tensile constitutive model combines the ascending and the descending segments, as shown in Figure 1. The compressive stress-strain curve has three segments with different mathematical equations presented in equations (1)–(3) and Figure 1(a): the first segment covers from 0 up to 40% $f_{cm}$, which is linear, the second segment is from 40% $f_{cm}$ to $f_{cm}$, and the third segment is the descending branch.

As shown in Figure 1(b), concrete tensile stress-strain is linear and ascending in the initial segment and nonlinear and descending in the second segment. The model can capture the important characteristics of concrete materials, including material deterioration due to cracking and crushing. The
descending segments of both compressive and tensile stress-strain are affected by the mesh size of the concrete element.

first segment
\[ \sigma_c = E_c \varepsilon_c. \] (1)

The second segment
\[ \sigma_c = \frac{E_{c1} \varepsilon_c f_{cm} - (\varepsilon_c f_{cm})^2}{1 + (E_{c1} f_{cm} f_{cm} - 2) \varepsilon_c f_{cm}} \] (2)
\[ f_{cm} = f_{ck} + 8 \quad \text{and} \quad \varepsilon_{cm} = 0.0022, \]
\[ E_{c1} = 1000 f_{cm}^{1/3}, \quad E_o = \left(0.8 + 0.2 \frac{f_{cm}}{88}\right) E_{c1}. \] (3)

The third segment
\[ \sigma_c = \left(2 + 2 \frac{f_{cm} \varepsilon_c}{f_{cm}} - \frac{\varepsilon_c^2 \gamma_c}{2 f_{cm}}\right)^{-1}, \] (4)
\[ \gamma_c = \frac{\pi^2 f_{cm} \varepsilon_c}{2 \left[G_{ch}/l_{eq} - 0.5 f_{cm} (\varepsilon_c (1 - b) + b f_{cm}/E_o)\right]^2} \]
\[ b = \frac{\varepsilon^{pl}_{c}}{\varepsilon^{el}_{c}} \] (5)

In equation (5), the characteristic length \(l_{eq}\) represents the mesh size, the type of finite element, and the crack direction, and the value of \(b\) is determined by iteration, which affects the softening, branch of the compressive stress-strain relation. Initially, the value of \(b\) is assumed to be 0.9 based on experimental observation.

Regarding the tensile behavior of concrete, the ratio between crack width at a given loading and the maximum tensile strength is shown in the following equation:

\[ \sigma_t(w) = \left[1 + \left(\frac{w}{w_c}\right)^{2} \right] \frac{E_i w}{w_c} \frac{1 + C_i^2}{\tan d} \] (6)
\[ f_{cm} = 0.3016 f_{ck}^{2/3} \]
\[ C_1 = 3, C_i = 6.93, \]
\[ w_c = 5.14 \frac{G_F}{f_{cm}} \]
\[ G_F = 0.073 f_{ck}^{0.8} \]
\[ G_{ch} = \left(\frac{f_{cm}}{f_{tm}}\right) G_F \]
\[ \varepsilon_t = \varepsilon_{tm} + \frac{w}{l_{eq}} \] (7)

Then, concrete compressive and tensile damage evolutions have been calculated using as per the following equations:

\[ d_c = 1 - \frac{1}{2 + a_c} \left[2 (1 + a_c) \exp(-b_c \varepsilon_c^{ck}) - a_c \exp(-2b_c \varepsilon_c^{ck})\right], \] (8)
\[ a_c = 7.873, \quad a_t = 1, \quad b_c = \frac{1.97 (f_{ck} + 8)}{G_{ch} l_{eq}}, \quad \text{and} \quad b_t = \frac{0.453 (f_{ck} + 8)}{G_f l_{eq}}. \] (9)

\[ \frac{0.453 (f_{ck} + 8)}{G_f l_{eq}}. \] (10)

Finally, crushing strain, cracking strain, and plastic strain (for both compressive and tensile) of concrete have been determined with equations (11) through (16) provided in [28–30].
\[ \sigma_c = (1 - d_c) E_{co} (\varepsilon_c - \varepsilon_c^{pl}), \] (11)
\[ \varepsilon_c^{ch} = \varepsilon_c - \varepsilon_c^{el} = \frac{\sigma_c}{E_{co}}, \] (12)
\[ \varepsilon_c^{pl} = \varepsilon_c^{ch} - \frac{d_c}{1 - d_c} \frac{\sigma_c}{E_{co}}, \] (13)
\[ \sigma_t = (1 - d_t) E_{co} (\varepsilon_t - \varepsilon_t^{pl}), \] (14)
\[ \varepsilon_t^{ch} = \varepsilon_t - \varepsilon_t^{el} = \frac{\sigma_t}{E_{co}}, \] (15)
\[ \varepsilon_t^{pl} = \varepsilon_t^{ch} - \frac{d_t}{1 - d_t} \frac{\sigma_t}{E_{co}}, \] (16)

Figure 2 illustrates the concrete damaged plasticity model input used for concrete strength with 47.7 MPa under different element sizes, which consider both the elasticity and plasticity behavior of the concrete. The plasticity region is mostly affected by the element size of the model, which is used to access the mesh size sensitivity of the response. On the other hand, Figure 3 represents the concrete damaged plasticity model input data for all concrete strength with a 30 mm element size. Mesh size-based sensitivity parameters are presented in Table 1, which was computed as per the procedure stated in the paper [22]. Concrete damage parameters have been selected for the study based on Abaqus default value recommendation [28]. The magnitude of each concrete damage parameter has been shown in Table 2.

2.2. Steel Material Idealization. The uniaxial stress-strain response of steel reinforcement recommended in the paper [31] was employed, which is based on a bilinear modeling approach. The first slope of the stress-strain curve represents the elastic modulus, and the second slope represents the hardening modulus of elasticity. Different diameters of steel reinforcements were used for the modeling. The diameters and properties of steel reinforcements considered in the model have been illustrated in Table 3.

2.3. Geometric Descriptions. The same geometric dimension has been considered for all specimens; however, half scale of the total specimens was modeled with and without a drop panel. Table 4 shows model specimens employed in the FE simulation. The slab has a depth of 120 mm, a length of 3000 mm, and a width of 1500 m. The model contains six specimens with drop panels and six specimens without drop panels. The drop panel has a total depth of 180 mm with slab, 600 mm width along transverse, and 1000 mm length along the longitudinal direction. The column was modeled with a 300 mm by 300 mm cross section and a total length of 1750 mm. Four concrete grades having a magnitude of 47.7 MPa, 60 MPa, 75 MPa, and 90 MPa were considered.

2.4. Detailing Reinforcement. Figure 5 illustrates the detailed reinforcement of the model specimen taken from [32]. The spacing of reinforcement was closely spaced in the column strip, in both transverse and longitudinal directions to control punching shear failure, as shown in Table 5. Steel having a diameter of 15 mm, 8 mm, and 7 mm has been used for modeling column longitudinal reinforcement, slab longitudinal and transverse reinforcement, and column stirrups, respectively. A 12 φ13 mm longitudinal steel reinforcement and φ7 mm with 50 mm center-to-center spacing of stirrups was provided for the column.

2.5. Modeling Loading, Boundary Condition, Meshing, and Interaction. As it is well known, adequate boundary conditions and fine meshing are needed to get an equilibrium solution. Figure 6 shows the loading, boundary condition, and meshing of the flat slab-column connection of the FE model. Pin support has been applied at the column’s bottom end. Except for horizontal translation, restraint has been applied at the right and left ends of the slab. This constraint condition allows the response of the model under lateral cyclic loading. Three mesh sizes of 30 mm, 40 mm, and 60 mm have been used for meshing, to access mesh size sensitivity response. The number of elements becomes 3000, 10,260, and 24,000 as meshing sizes of 60 mm, 40 mm, and 30 mm are used, respectively, for models without drop panels. In another case, the number of elements becomes 3170, 11,010, and 25,360 as meshing sizes of 60 mm, 40 mm, and 30 mm are used, respectively, for models with drop panel. Distributed gravity load having 0.1 gravity shear ratio magnitude has been applied on the flat slab floor, and a cyclic loading history is shown in Figure 7, which was applied at the top end of the column [32]. For this particular study, the truss elements of the reinforcement bar are embedded inside the concrete solid elements by using embedded constraint which is assumed that no bond slip or deboning occurred between the steel bars and concrete.

2.6. Validation of Finite Element Model. The study was validated against previously performed experimental results on the flat slab-column connection under cyclic loading [32]. The test specimen was a half-scale model, designed to have a low reinforcement ratio, drop panel, and without shear reinforcement. The FE model was established exactly based on all conditions undertaken in the experimental test. Figure 8 illustrates the comparison of FE analysis and experimental results for the validation, which also shows the geometric dimension of the test specimens. The validation was performed by considering different mesh sizes of 30 mm, 40 mm, and 60 mm. The model with 30 mm, 40 mm, and 60 mm element sizes showed 10.98%, 15.32%, and 19.78% differences as compared to the experimental result. The model with a small element size showed good agreement with the experimental result.
Figure 2: Concrete damaged plasticity model input based on different element sizes for concrete grade with 47.7 MPa. (a) Compressive stress vs crushing strain. (b) Compressive damage vs crushing strain. (c) Tensile stress vs cracking strain. (d) Tensile damage vs cracking strain.

Figure 3: Continued.
Figure 3: Concrete damaged plasticity model input data for all concrete grades with 30 mm element size. (a) Compressive stress vs crushing strain. (b) Compressive damage vs crushing strain. (c) Tensile stress vs cracking strain. (d) Tensile damage vs cracking strain.

Table 1: Element size sensitivity parameters.

<table>
<thead>
<tr>
<th>$f_{cm}$ (MPa)</th>
<th>Element size (mm)</th>
<th>$G_{ch}$ (N/mm)</th>
<th>$G_F$ (N/mm)</th>
<th>$b$</th>
<th>$a_c$</th>
<th>$a_t$</th>
<th>$b_c$</th>
<th>$b_t$</th>
</tr>
</thead>
<tbody>
<tr>
<td>47.7</td>
<td>30</td>
<td>27.034</td>
<td>0.1464</td>
<td>0.896</td>
<td></td>
<td></td>
<td>104.28</td>
<td>1080.5</td>
</tr>
<tr>
<td>47.7</td>
<td>40</td>
<td>27.034</td>
<td>0.1464</td>
<td>0.821</td>
<td>7.873</td>
<td>1</td>
<td>139.04</td>
<td>1440.7</td>
</tr>
<tr>
<td>60</td>
<td>30</td>
<td>31.105</td>
<td>0.153</td>
<td>0.819</td>
<td>7.873</td>
<td>1</td>
<td>114.00</td>
<td>1241.2</td>
</tr>
<tr>
<td>60</td>
<td>30</td>
<td>36.085</td>
<td>0.159</td>
<td>0.796</td>
<td>7.873</td>
<td>1</td>
<td>122.83</td>
<td>1411.8</td>
</tr>
<tr>
<td>75</td>
<td>30</td>
<td>41.016</td>
<td>0.164</td>
<td>0.779</td>
<td>7.873</td>
<td>1</td>
<td>129.68</td>
<td>1563.2</td>
</tr>
<tr>
<td>75</td>
<td>30</td>
<td>41.016</td>
<td>0.164</td>
<td>0.779</td>
<td>7.873</td>
<td>1</td>
<td>129.68</td>
<td>1563.2</td>
</tr>
</tbody>
</table>

Table 2: Concrete damage parameter.

<table>
<thead>
<tr>
<th>Eccentricity ($\gamma$)</th>
<th>Dilation angle ($\psi$)</th>
<th>$K$</th>
<th>$\sigma_{sw}/\sigma_{co}$</th>
<th>Viscosity parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>31</td>
<td>0.667</td>
<td>1.16</td>
<td>0.0001</td>
</tr>
</tbody>
</table>

Table 3: Reinforcement bars diameter and properties.

<table>
<thead>
<tr>
<th>Rebar type</th>
<th>Bar size (mm)</th>
<th>Area (mm$^2$)</th>
<th>Elastic region</th>
<th>Inelastic regions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$f_y$ (MPa)</td>
<td>$\epsilon_y$ (%)</td>
</tr>
<tr>
<td>Threaded</td>
<td>12.78</td>
<td>128.01</td>
<td>394.89</td>
<td>0.197</td>
</tr>
<tr>
<td>Plain</td>
<td>7.96</td>
<td>46.42</td>
<td>324.87</td>
<td>0.217</td>
</tr>
<tr>
<td>Plain</td>
<td>6.89</td>
<td>7.27</td>
<td>327.07</td>
<td>0.192</td>
</tr>
</tbody>
</table>

Table 4: The number of specimens for FE simulation.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>SCCD11</th>
<th>SCCD21</th>
<th>SCCD31</th>
<th>SCCD41</th>
<th>SCCD12</th>
<th>SCCD22</th>
<th>SCCD32</th>
<th>SCCD42</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete grade</td>
<td>47.7</td>
<td>60</td>
<td>75</td>
<td>90</td>
<td>47.7</td>
<td>60</td>
<td>75</td>
<td>90</td>
</tr>
<tr>
<td>Drop panel</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>
Figure 4: Geometric dimension of the model specimens. (a) Side view without drop panel. (b) Side view with drop panel. (c) Top view without drop panel. (d) Top view with drop panel.

Figure 5: Detail reinforcement for the specimens.

Table 5: Detail reinforcement of flat slab.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Top reinforcement</th>
<th>Bottom reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Column strip</td>
<td>Middle strip</td>
</tr>
<tr>
<td>Trans-</td>
<td>$13\Phi 8\text{ mm }c/c\text{ 50 mm}$</td>
<td>$24\Phi 8\text{ mm }c/c\text{ 120 mm}$</td>
</tr>
<tr>
<td>Long-</td>
<td>$13\Phi 8\text{ mm }c/e\text{ 50 mm}$</td>
<td>$6\Phi 8\text{ mm }c/e\text{ 135 mm}$</td>
</tr>
</tbody>
</table>
3. Result and Discussion

3.1. Cracking Pattern. The region around the slab-column joint was extremely affected, and it is a critical area that needs extensive investigation and consideration during analysis and design. Since the cracking pattern obtained from FE simulation is almost similar for all specimens, the cracking pattern of a sample specimen (SCCD11) has been presented as shown in Figure 9. Under cyclic lateral loading, the near column joint region is subjected to reverse compression and tension action. Gravity loading on the slab increased tension cracking on the face of the column and slab at the connection region opposite to the direction of lateral loading.

3.2. Steel Strain. The strain measurement has been taken from different points of the model specimen to examine the strain distribution. It contains strain in the longitudinal top steel reinforcement of the slab and the longitudinal steel reinforcement of the column located near to slab-column joint. Since the strain distributions during FE analysis were similar for all specimens, the strain distribution of one specimen (SCCD11) was selected for its ease of presentation. Figure 10(a) shows the strain of the top of the longitudinal reinforcing bar of the slab near the face of the column at different stages of loading.

Figure 10(b) shows the strain of the longitudinal reinforcing bar of the column near the slab-column joint at different stages of loading. The strain in column longitudinal reinforcement is slightly greater than that of slab reinforcement. As a general, the conclusion has been made that at the same level of loading, the strains in both slab and column longitudinal reinforcements are decreased as concrete grade increases. The strain in the slab and column reinforcements also slightly decreased when the drop panel was used compared to the slab-column connection without the drop panel.

3.3. Load-Drift Ratio Curve. Table 6 summarizes the effect of drop panel and concrete grades on the initial cracking, yielding, ultimate and failure load for slab-column connection obtained from FE simulation.
3.3.1. Effect of the Drop Panel. The study considered and investigated the effect of drop panels on the performance of slab-column connection as subjected to cyclic lateral loading. Figure 11 illustrates the backbone curves of slab-column connection with and without drop panels by considering different concrete grades. The presence of a drop panel slightly caused the increase in the ultimate load capacity of the joint when compared to concrete grade. In overall records, the drop panel increased the ultimate load from 3.19% to 7.53% with respect to the slab-the-column connection without the drop panel. This arose from the increase of punching shear capacity as a drop panel is provided.

3.3.2. Effect of Concrete Strength. Figure 12 shows the recorded backbone curves from hysteretic load-drift curves with cyclic lateral load by considering different concrete grades.
grades. Figures 12(a) and 12(b) present the backbone/envelope curves of slab-column connections without and without drop panels, respectively. For both slab-column connections, with and without drop panels, the slope of the curves was increased as the concrete grade increased. This implies that at the same level of drift ratio, a large lateral load was resisted by connection with larger concrete strength. As concrete grade increased from 47.7 MPa to 90 MPa, the ultimate load resisted by the slab-column connection increased by 69.28% and 63.99%, for the joint without and with drop panel, respectively. This implies that as concrete grade increases, the punching shear capacity is increased. After the specimens reached the ultimate strength, an evident strength degradation was observed at every loading cycle. The
backbone curve is dropped after the ultimate load resistance and eventually converges to a stable value. In addition, when the load reached the ultimate capacity, the specimens still possess a certain capability of bearing and deformation.

3.4. Slab Deflection. The slab deflection has been briefly investigated by considering the presence and absence of drop panels and different concrete grades. Figures 13 and 14 show the vertical deflection of the slab parallel to the free edge of the slab due to concrete grades and drop panel, respectively. The deflection of the slab is decreased by 31.97% and 27.44% in the joint without and with drop panel, respectively, as the concrete grade is increased from 47.7 MPa to 90 MPa. The deflection is decreased up to 11.07% when the drop panel is used.

3.5. Ductility. The envelope curves in this study are obtained based on the hysteretic curve. The ductility values reflect the deformation capacity under lateral cyclic loading, and it is the ratio between deflection at failure and yielding. Ductility is the most determinant behavior of the structure in a seismic area where the building is mostly subjected to cyclic loading. Table 6 shows the ductility capacity of each specimen. The ductility capacity of the slab-column connection is decreased up to 34.36% and 27.05%, as the concrete grade increased from 47.7 MPa to 90 MPa without and with drop panels, respectively. Therefore, during the design of slab-column joint, special care should be given to detailing to get sufficient ductility under cyclic loading. Over reinforcement results in concrete crushing means failure of structure without warning. But, under reinforcement will result in steel yielding before crushing of concrete which can shows sufficient
warning time before failure. The use of drop panels also decreased ductility capacity up to 16.67% with respect to the slab-column connection without drop panels.

4. Conclusion
This paper considers the investigation of the behavior of reinforced concrete slab-column connection using nonlinear FE analysis in the Abaqus software package. Twelve specimens have been selected for the model. The effect of high-strength concrete grades (47.7 MPa, 60 MPa, 75 MPa, and 90 MPa) and drop panels have been studied. The mesh size sensitivity analysis is also accessed to check the validity of the model. The study mainly focused on cracking patterns, strain distribution, slab deflection, and load-drift ratio envelope curve. The investigation confirmed a significant and slight effect of concrete grade and drop panel on the slab-column connection response under cyclic loading, respectively. As concrete grade increased, the drift ratio and slab deflection decreased significantly. In another way, the drop panel slightly affects the slab deflection and drift ratio when compared to concrete grade. Furthermore, the simultaneous application of gravity load on the slab floor with cyclic loading at the top end of the column resulted in the increase of cracking width on the face of the slab and column opposite to the direction of lateral cyclic loading. However, it decreased the drift of the top end column. The FE analysis result also showed a good agreement with a previously conducted experimental investigation, during validation.

Abbreviations

\[ a_c \]: Dimensionless coefficients in compression damage function

\[ a_t \]: Dimensionless coefficients in tensile damage function
$b$: Ratio $b/n$ compressive and tensile strain

$b_c$: Dimensionless coefficients in compression damage function

$b_t$: Dimensionless coefficients in compression damage function

$C_1$: Cohesion coefficient in tensile stress function

$C_2$: Cohesion coefficient in tensile stress function

$d_c$: Compression damage parameter

$d_t$: Tension damage parameter

$\sigma_c$: Concrete compressive stress

$\sigma_{bo/\sigma_{co}}$: Biaxial stress ratio

$\sigma_f (w)$: Tensile stress at given crack width

$E_o$: Undamaged modulus of deformation

$E_{ci}$: Tangent modulus of deformation of concrete for zero stress

$\varepsilon_c$: Compressive strain

$\varepsilon_{cmi}$: Strain at maximum compressive stress

$\varepsilon_{ch}$: Crushing strain

$\varepsilon_{ci}$: Elastic damaged compressive strain

$\varepsilon_y$: Steel yield strain

$\varepsilon_{p}$: Steel ultimate strain

$\varepsilon_{pl}$: Compressive plastic strain

$\varepsilon_{t}$: Tensile strain

$\varepsilon_{el}$: Elastic undamaged tensile strain

$\varepsilon_{et}$: Elastic damaged tensile strain

$\varepsilon_{tm}$: Strain at maximum tensile stress

$\varepsilon_{ck}$: Cracking strain

$\varepsilon_{ct}$: Tensile plastic strain

$f_{cm}$: Maximum compressive strength

$f_{tm}$: Maximum tensile strength

$f_{ck}$: Characteristic concrete compressive strength

$G_{ch}$: Crushing energy per unit area

$G_{p}$: Fracture energy per unit area

$\gamma$: A coefficient that is used in concrete stress-strain computation

$\psi$: Dilatation angle

$K$: Tensile-to-compressive meridian ratio

$l_{eq}$: Mesh size (element size)

$w_c$: Critical crack width

$w$: Cracking width

$f_y$: Steel yield strength

$f_{ul}$: Ultimate steel strength.

**References**


[16] A. Abdelkhalik, T. Elafandy, A. Abdelrahman, and A. Sherif, "Tests of slab-column connections subjected to reversed cyclic

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**Data Availability**

All data have been included in the research paper.

**Conflicts of Interest**

The authors declare that they have no known competing financial interests.

**Acknowledgments**

The authors would like to thank all those who shared resources and information.


