Nonlinear analysis of exterior precast beam-column J-Bolt and cleat angle connections

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ABSTRACT

In this study a 3-D nonlinear FE model was developed to study the response of an exterior precast beam to column connection subjected to reverse cyclic loading. Tests of a one-third scale exterior beam column precast concrete connections was conducted. Two types of connections were compared. The connections included a monolithic connection and two precast beam - column connections (i) using J-bolt (ii) using Cleat Angle. ANSYS finite element software was used for the non-linear analysis of the precast beam column connection. For the nonlinear analysis, one-third scale model was developed. Two types of elements were used including solid elements and contact elements. The finite element analysis results compared well with the experimental data. It is concluded that if the material constitutive relation and failure criterion can be selected suitably, the finite element model can accurately predict the overall seismic behavior and the inelastic performance of these two kinds of joints

Keyword: Nonlinear analysis, precast concrete, dry connection, J-bolt, cleat angle.

1. Introduction

Precast concrete systems have many advantages like speed in construction, good quality due to factory production, economy in mass production. Despite many advantages of precast concrete, it is not widely used throughout the World, especially in regions of high seismic risk. The reason behind this is a lack of confidence and knowledge base about their performance in seismic regions as well as the absence of rational seismic design provisions in major model building codes (Priestley, 1991). High storey precast frame panel buildings performed poorly in the 1988 Spitak, Armenia earthquake due the lack of adequate seismic design considerations such as ductility in precast joints (Hadjian, 1993).

A significant number of parking structures suffered extensive damage and a number of precast concrete parking structures collapsed in the 1994, Northridge earthquake. One of the reasons for the collapse was lack of proper diaphragm connections (Mitchell et al., 1995). In the 1995 Kobe earthquake, most of the precast prestressed concrete structures performed well, only three sustained severe structural damage. The structural damage was due to insufficient connection detailing (Muguruma et al., 1995). The lessons learnt from the past earthquakes are that the connections are the weakest link. Hence more research is required in the study of connections.
1.1 Literature survey

Dolan and Pessiki (1984) demonstrated the behavior characteristics of a welded monotonically loaded precast concrete connection can be simulated using models. Tests of one-quarter scale models of a single beam to column connection were conducted. Good agreement was found between the strength and the normalized moment rotation response of the model and the prototype. The effects of weld quality and design eccentricities had similar consequences in both model and prototype.

Ersoy and Tankut (1993) tested precast concrete beams with dry joints designed for multistory buildings located in a seismic area under reversed cyclic loading. The original beam consisted of two steel plates one at top, the other at the bottom, welded to the anchored steel plates in the column bracket and the beam. The design was later revised by adding side plates. The main variables were presence of side plates and joint width. The authors concluded that the joint width is an important parameter and therefore tolerances should be checked carefully during erection. The strength, stiffness and energy dissipation of the member with side plates were comparable to those of monolithic member.

Kachlakev et al (2001) studied the behavior of four concrete beam members with externally bonded Carbon Fiber Reinforced Polymer (CFRP) fabric using ANSYS. Solid65, Link8, Solid46 and Solid45 were used to model concrete, steel reinforcement, FRP composites and steel plates respectively. Symmetry allowed one quarter of the beam to be modeled. It was concluded that in the load strain plots, the strain in the linear stage from the FE analysis correlated well with those from the experimental data. The yield load of steel from FE analysis was 14% lower than that of the test results. In the linear range, the load deflection plot was stiffer when compared to the experimental results. The first cracking loads obtained form ANSYS was higher than the test data. ANSYS underestimated the ultimate load of the beams by 5% to 24%.

Ibrahim and Mubarak (2009) studied the behavior of externally prestressed continuous concrete beams subjected to symmetrical static loading. A numerical model based on the finite element method using ANSYS. The elements Solid65 and Link8 were used to model concrete and steel reinforcement. The prestress in the finite element was given as an initial strain in the link element. Solid45 element was used for steel plates at the support and loading location to avoid stress concentration problems. The anchorage zone was modeled as steel plate which was connected to the tendon element. The finite element analysis showed good agreement with the experimental results throughout the entire range of behavior and failure mode.

Pirmoz and Danesh (2009) studied the effect of the seat angle stiffness on moment-rotation response of the bolted top-seat angle connections using finite element method ANSYS. All components of the connection such as the beam, column, angles and bolts head are modeled using eight node-first order Solid45 elements and bolt shanks are modeled using Solid64 elements, which can apply a thermal gradient on it to pretension the bolts. The effect of interactions between components, such as slippage of bolts and frictional forces, are modeled using surface contact algorithm. ANSYS can model contact problems using contact pair elements Conta174 and Targe170, which pair together in such a way that no penetration occurs during the loading process. Thus the effect of adjacent surface interactions, including angle-beam flange, angle/beam flange-bolt head/nut, bolt hole bolt shank and effect of friction, are modeled using the mentioned contact elements. In the finite element analysis the difference between test data and numerical models grows in nonlinear portion of curves.
From the literatures, it is observed that the precast connections can be detailed to emulate the behaviour of monolithic connections. It is also understood that the dry connections have better energy dissipation characteristics. In the present study, a precast beam column connection using J-Bolt is investigated by conducting experiments and using Finite Element Modeling. The finite element package ANSYS for modeling. Information about the elements to be used in modeling the various materials was also studied. It was concluded that, for modeling precast connection in ANSYS the most appropriate elements to be are Solid65, Link8, Contact174, Target170, and Solid45. Also input data for material model to simulate concrete to behave in multi-linear, elastic and inelastic stages are decided from the literature study.

2. Analysis of the structure

For the design of the beam, column and the beam-column connection a three-storey reinforced concrete building had been analyzed. The shear forces, bending moments and axial forces in the exterior beam-column joint in the first floor had been calculated for the various load combinations including earthquake loading. Figure 1 shows the plan and elevation of the building showing the exterior beam-column joint considered (joint A). The maximum axial force calculated in the column is 287 kN. Seismic analysis had been performed using equivalent lateral force method recommended by IS 1893 (Bureau of Indian Standards (BIS) 2002). The design and detailing of beam, column and exterior joint had been done based on the guidelines given in IS 456 (Bureau of Indian Standards (BIS) 2000) and IS 13920 (Bureau of Indian Standards (BIS) 1993) respectively. One-third scaled models had been developed for monolithic and precast specimens with cross-sectional dimensions 100 mm x 100 mm for both the beam and column. The clear span of the beam was 550 mm. The height of the column was 1200 mm. The cover thickness of monolithic and the two precast beam and column specimens were 10 mm.

![Figure 1: Details of the building](image)

3. Types of connections

3.1 Precast connection: Beam to column connection using J-Bolt (PC-JB)

In this connection the beam was supported on concrete corbel using J-bolt. This connection transmits vertical shear forces. J-bolt of diameter 16 mm was kept inside the corbel and cast by keeping its straight portion protruding outside. The beam was inserted on to the J-bolt and the nut tightened. Iso-resin grout was used to fill the gap between the J-bolt and the hole in
the beam. The schematic representation of the isometric view of precast concrete column with corbel and the beam connected using a J-bolt is shown in Figure 2 (a).

3.2 Precast connection: Beam to column connection using Cleat Angle (PC-CL)

In this connection two 16mm diameter bolts were used, in which one bolt connects the cleat angle with the column and the other connects the cleat angle with both the beam and the corbel. In the precast elements, sleeve holes of 21 mm diameter were cast inside the column, beam and corbel. The cleat angle used for the connection is ISA 100x100x10. The bolts used were of grade 4.6. Both the cleat angle and bolts were designed according to IS 800 (Bureau of Indian Standards (BIS) 2007). The gap between the bolts and the bolt hole was filled using iso-resin grouts. The schematic representation of the isometric view of the specimen PC-CL is shown in Figure 2 (b).

3.3 Monolithic specimen (ML)

The monolithic reinforced concrete test specimen (ML) was designed according to IS 456 (Bureau of Indian Standards (BIS) 2000) and detailed according to IS 13920 (Bureau of Indian Standards (BIS) 1993). The Flexural reinforcement for the beam consisted of four bars with one bar at each corner of the transverse reinforcement. Two numbers of 10 mm diameter bars were provided as tension reinforcement and two numbers of 10 mm diameter bars were provided as compression reinforcement. The shear reinforcement consisted of 3 mm diameter two legged stirrups spaced at 60 mm. For a distance of 100 mm from the column face the spacing of the lateral ties were decreased to 25 mm. The column reinforcement arrangement also consisted of four 10 mm diameter bars. Along the column height excluding the joint region, the lateral ties were spaced at 50 mm. At the joint region the spacing of the lateral ties were reduced to 25 mm. The schematic representation of the isometric view of the monolithic specimen is shown in Figure 2(c).

![Figure 2: Isometric view of the three specimens](image-url)
3.4 Development of finite element model

To develop a model in ANSYS it is very mandatory to know the characteristics of the elements and important part is to select the most appropriate element as per the requirement. Also the input data to simulate the behaviour of the material like strength parameters of the concrete and steel must be fed at appropriate places. The input data given to ANSYS is given below.

Following elements used for the modeling.

1. Solid65 is used for concrete element
2. Link8 for reinforcement
3. Conta174 And Targe170 for grout material
4. Solid45 for angle and loading plates

3.4.1 Sectional properties (Real constants)

The real constants considered for SOLID65 element are volume ratio and orientation angles (in X and Y direction). Discrete reinforcement is considered for the present study. Hence the smeared reinforcement capability of the Solid65 element is turned off for real constant set 1 (volume ratio and orientation angle were set to zero). The parameters considered for LINK8 element are cross sectional area and initial strain. The real constant values for LINK8 element used for modelling the models are as given in the Table 1.

<table>
<thead>
<tr>
<th>Real Constant Set</th>
<th>Element Type</th>
<th>Particulars of the Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Link 8 (Main Bars)</td>
<td>Cross sectional area (mm²)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Initial Strain</td>
</tr>
<tr>
<td>3</td>
<td>Link 8 (Stirrups)</td>
<td>Cross sectional area (mm²)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Initial Strain</td>
</tr>
</tbody>
</table>

3.4.2 Material properties

“EX” is the modulus of elasticity of the material considered and “PRXY” is the Poisson’s ratio. The characteristic compressive strength of the concrete considered is \( f_{c}^{'} \) 33.28 N/mm\(^2\) which was obtained from experiments and the Poisson’s ratio was 0.15.

\[
E_c = 57000 \sqrt{f_c^'} = 57000 \sqrt{4825.12} = 3959398.41 \text{psi} = 27308.9 \text{N/mm}^2
\]

The yield stress and tangent modulus of reinforcement bars were obtained from laboratory test. The uniaxial stress-strain relationship for concrete was developed (Desai and Krishnan.1964), which is given by following Equation, was adopted for modeling concrete.
\[ f = \frac{E_c \varepsilon}{1 + \left( \frac{\varepsilon}{\varepsilon_o} \right)^2} \]  

\[ \varepsilon_o = \frac{2f_c}{E_c} \]  

\[ E_c = \frac{f}{\varepsilon} \]  

where \( f \) - stress at any strain \( \varepsilon \), psi  
\( \varepsilon \) - strain at stress \( f \)  
\( \varepsilon_o \) - strain at the ultimate compressive strength \( f_c \)  

<table>
<thead>
<tr>
<th>Mat. Model No</th>
<th>Element Type</th>
<th>Material Properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Link-Spar8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Linear Isotropic</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EX</td>
<td>( 2 \times 10^5 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td></td>
<td>PRXY</td>
<td>0.3</td>
</tr>
<tr>
<td>2</td>
<td>Solid-Concrete65</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Linear Isotropic</td>
<td></td>
</tr>
<tr>
<td></td>
<td>EX</td>
<td>( 27308.9 \text{ N/mm}^2 )</td>
</tr>
<tr>
<td></td>
<td>PRXY</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Concrete  
Shear transfer coefficients for an open crack 0.5  
Shear transfer coefficients for a closed crack 0.9  
Uniaxial tensile cracking stress 3.83 \( \text{ N/mm}^2 \)  
Uniaxial crushing stress -1  
Biaxial crushing stress 0  
Uniaxial crushing stress under ambient hydrostatic stress state 0  
Biaxial crushing stress under ambient hydrostatic stress state 0  
Stiffness multiplier for cracked tensile condition 0.6

### 3.4.3 Modeling of beam-column joints

Three types of modeling are used to generate the model in ANSYS. They are discrete model, smeared model and embedded model. Here discrete type was adopted to generate the model. In discrete model, concrete and steel share the same nodes. To apply the pressure and displacement, steel plates are provided under free end of the beam as well as top of the beam.
column. In monolithic connection only SOLID65, SOLID45 and LINK8 elements are used to generate the model. But in precast connections contact elements are used for grout. To model the J-bolt, spider bolt modeling is adopted. The spider bolt simulation substitutes line elements for the head, nut, and stud. A series of line elements represent the head/nut in a web-like fashion. Thus, the name spider bolts. It is the most logical approach to use line elements and transferring the loads to the stud. The head/nut bending and stiffness must be simulated by the line elements. A portion of the stud line elements should be line elements with tension only capability, since no contact elements are used at the head/nut to flange connection. In beam corbel connection surface to surface contact elements are used. In column beam interface also surface to surface contact elements are used (CONTA174 and TARGE170). In surface to surface contact elements if one surface is stiffer than the other, the softer surface should be the contact surface and stiffer surface should be the target surface. Here assumed no bond slip occurs between steel and concrete. Figure 3 (a) and (b) shows the reinforcement models of monolithic and precast specimen respectively. Figure 4 (a) and (b) shows the Analytical models of the connecting elements in the precast specimens.

![Figure 3: Reinforcement models of monolithic and precast specimens](image1)

![Figure 4: Analytical models of the connecting elements in the precast specimens](image2)

4. Results and discussion

4.1 Strength

The ultimate load carrying capacity of specimen ML was 13.02 kN and 13.55 kN in the positive and negative directions respectively. For the specimen PC-JB, the ultimate load
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Carrying capacity was found to be 5.52 kN and 5.17 kN in positive and negative directions respectively which is lesser than the monolithic specimen. The load carrying capacity of specimen PC-JB was 57.61% and 59% lesser than the monolithic specimen in the positive and negative direction respectively. For the specimen PC-CL, the ultimate load carrying capacity was found to be 4.55 kN and 4.18 kN in positive and negative directions respectively. It was observed that the load carrying capacity of specimen PC-CL was 61.65% and 69.53% lesser than the monolithic specimen ML in the positive and negative direction respectively. Out of the precast specimens, the specimen with J-bolt PC-JB performed better than the specimen with cleat angle PC-CL. While comparing with the precast specimens the monolithic specimen performed better in resisting the load.

4.2 Load displacement relationship

The load-displacement hysteresis loops for the cyclic loading at each displacement excursion level of monolithic specimen ML is shown in Figure 4(a). It exhibited fat hysteresis loops with very less pinching, due to good bonding between reinforcement and joint concrete. The slight pinching was due to diagonal cracking in the joint region and flexural cracking in the beam. The areas of the hysteresis loops gradually became larger as the displacement cycle increased, which indicates good energy dissipating capacity. Figures 4(b) and 4(c), show the load- displacement hysteresis response of the precast specimens PC-JB and PC-CL. For specimens PC-JB, greater pinching was observed because of predefined gap opening at the connections, which indicates minimal energy dissipation.

![Figure 4: Load- Displacement hysteretic relationship](image)

a) Specimen ML  

b) Specimen PC-JB  

c) Specimen PC-CL
4.3 Energy dissipation

For the structure to perform satisfactorily in the inelastic range, it should exhibit good energy absorption capacity. The area enclosed by the hysteretic loop in a given cycle represents the energy dissipated by that specimen during that cycle. Figure 5 provides a comparison of the cumulative energy versus displacement levels of all the specimens. The cumulative energy dissipated was computed by summing up the energy dissipated in the consecutive cycles throughout the test. The precast connection with J-Bolt PC-JB shows 43.07% reduction in the cumulative energy dissipation when compared to monolithic specimen. The total cumulative energy dissipated by specimen PC-JB is 578.82 kNmm. It is observed that the precast specimen with cleat angle PC-CL has very less energy dissipation capacity of 410 kNmm.

![Figure 5: Comparison of energy dissipation of the three specimens](image)

4.4 Ductility

The displacement ductility is the ratio of the maximum displacement that a structure or element can undergo without significant loss of initial loading to the initial yielding deformation. The displacement ductility factor was calculated for monolithic and the two precast beam-column connections is shown in Table 3. The displacement ductility factor of the monolithic specimen ML was greater than both the precast specimens. Out of the two precast specimens, specimen with J-bolt PC-JB showed better displacement ductility when compared to specimen PC-CL. The average displacement ductility of the specimens indicated that all the three connections behaved in a ductile manner.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield displacement $\Delta_Y$ (mm)</th>
<th>Ultimate displacement $\Delta_u$ (mm)</th>
<th>Displacement Ductility factor($\mu$)</th>
<th>Average displacement ductility factor($\mu$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Positive  Negative</td>
<td>Positive  Negative</td>
<td>Positive  Negative</td>
<td>Positive  Negative</td>
</tr>
<tr>
<td>ML</td>
<td>6.7  7.1</td>
<td>30  30</td>
<td>4.478  4.225</td>
<td>4.351</td>
</tr>
<tr>
<td>PC-JB</td>
<td>6.2  9.5</td>
<td>30  30</td>
<td>4.839  3.158</td>
<td>3.998</td>
</tr>
<tr>
<td>PC-CL</td>
<td>14.8  8.8</td>
<td>30  30</td>
<td>2.027  3.409</td>
<td>2.718</td>
</tr>
</tbody>
</table>

Table 3: Ductility factor of the specimens
5. Conclusion

Precast construction is most versatile form of construction and it provides high-quality structural elements, construction efficiency, and savings in time and overall cost of investment. In the design of earthquake resistant structures that incorporate precast concrete elements the main difficulty has been to find efficient and economical methods for connecting the precast concrete members together, and create connections that give adequate strength, stiffness and ductility. But lack of sufficient experimental data affects their application in high seismic regions. In this context, monolithic and precast specimens were cast and the behavior under cyclic loading was experimentally investigated and their results were validated with analytical models. From the results it was observed that the ultimate load carrying capacity of the monolithic specimen is more than the precast specimens PC-JB. The monolithic specimen ML is more ductile and dissipates more energy compared to the precast specimen PC-JB. Precast specimens showed increased stiffness in the negative direction due to the presence of corbel. In precast connection, the column was free from damage when compared to that of monolithic connection. This behavior satisfies the fundamental requirement of strong column weak beam theory. Considering the performance of the precast connections future scope of study has been planned to improve the ductile detailing of joints.

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6. References


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