

Finite Element Analysis of Retrofitting Techniques for Reinforced Concrete Beam-Column Joint

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Abstract: This study presents an analytical investigation of three retrofitting techniques, using finite element analysis aimed at improving the behavior of reinforced concrete beam-column joints to enhance the performance and load carrying capacity of structures. The three suggested retrofitting techniques presented are; reinforced concrete jacketing, steel plate jacketing and carbon fiber reinforced polymer (CFRP) sheets wrapping in two orthogonal directions to strengthen the joint and reduce deformations. Nonlinear static finite element analysis was carried out to evaluate the performance of the original and strengthened joint models. The performance has been investigated in terms of load carrying capacity, deflection, failure pattern and displacement ductility. The study shows that using steel plates jacketing with adequate thickness is more effective in reducing the deflection than the use of concrete jacketing and CFRP sheets. On the other hand, CFRP wrapping has shown an increase in the load carrying capacity and a weak beam-strong column failure pattern.

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1. Introduction

Beam-column joint deformations and strength affect the overall performance and load carrying capacity of reinforced concrete structures making them susceptible to progressive collapse due to failure of one or more beam-column joints under gravity and earthquake loadings. Therefore retrofitting of beam-column joints is needed to maintain structural safety and reliability. The prediction of shear strength and flexibility for inadequately reinforced beam-column joints before and after retrofitting will be done using three dimensional finite element analysis to evaluate the improvement of load carrying capacity of R.C. buildings in order to compare the different methods. In this study three methods of joint strengthening will be discussed. These methods are: carbon fiber reinforced polymers (CFRP) confinement, section enlargement by concrete jacketing, and steel jacketing. The ease of application and efficiency of each method will be discussed and compared.

The joint will be under bending stress, shear stress, and axial stress. The developed stresses inside the joint will be examined before the retrofitting and after each technique by conducting static analysis. Load carrying capacity, load-deflection behavior, ductility and failure pattern will be studied to investigate the response of the joint to the retrofitting technique and if it satisfies the required capacity.

2. Joint Description and Geometry

All specimens of analytical study that have been adopted for verification consist of a cantilever portion and column portion. The specimens had reinforcement

details as per code ACI 318M-11[1] as shown in Figure 1.

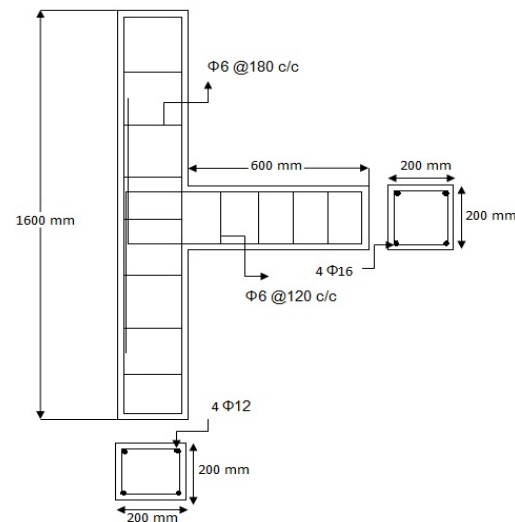


Figure 1. Reinforcement details for specimen.

The columns had a cross section of 200 mm x 200 mm with an overall length of 1600 mm. The beams had a cross section of 200 mm x 200 mm with a cantilevered portion of length 600 mm. The column portion was reinforced with 4 numbers of 12 mm diameters and the beam portion was reinforced with 2 numbers of 16 mm diameters in the tension zone and 2 numbers of 16 mm diameters in the compression zone. The main reinforcement had yield strength of 415 MPa. The lateral ties in the columns were 6 mm diameter at 180 mm center to center (c/c) spacing and the beams

had vertical stirrups of 6 mm diameter at 120 mm c/c. The lateral ties and the vertical stirrups had yield strength of 250 MPa. The concrete strength of the specimen adopted was 20 MPa.

3. Finite Element Model

Young modulus of elasticity of the concrete (E_c) and the Poisson's ratio (ν_c) were chosen as linear isotropic properties for the concrete. The modulus of elasticity of concrete based on the ACI 318M-11 equation $E_c = 4700\sqrt{f'_c}$. Poisson ratio for concrete was assumed to be 0.2 based on the compressive strength of concrete used in the beam and the column. The uniaxial crushing stress f'_c was considered to be 20 MPa and the uniaxial tensile cracking stress of concrete used in this study was considered to be 2.5 MPa.

For the non-linear isotropic behavior of concrete that the analysis needs, the stress-strain curve of concrete was built based on the equation:

$$\sigma_{cu} = \sigma'_{cu} * 2 * \left(\frac{\epsilon}{\epsilon_o} \right) - \left(\frac{\epsilon}{\epsilon_o} \right)^2$$

Where: $\sigma_{cu} = f'_c$ when $\epsilon \leq \epsilon_c \leq \epsilon_o$

ϵ is the strain and ϵ_o is the maximum strain.

σ_{cu} is the stress and σ'_{cu} is the maximum stress.

The ratio between the stress and the strain must be equal to Young's modulus at the first point of stress strain curve, and then the ratio is decreased to the last data when the compressive strength increases.

Figure 2. shows the stress, strain and the Young's modulus that describe the multi-linear isotropic behavior of the concrete.

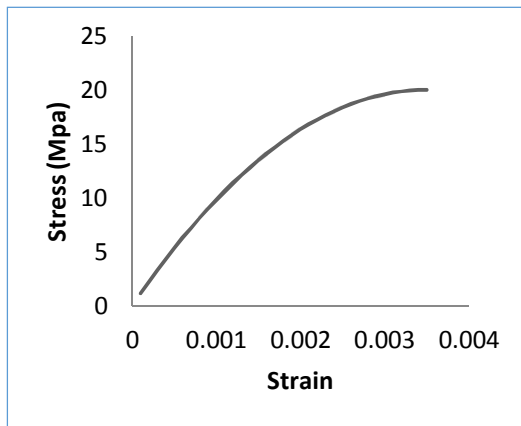


Figure 2. Stress-strain curve of concrete.

Eight-node 3D Solid 65 element type could be used as reinforced or unreinforced concrete element to verify the load carrying capacity led to an unexpected failure and crack. However, Solid 65 supports only three rebar. As a result, another element type should be used in order to model the reinforcement in tension, compression and the shear reinforcement.

The reinforcement had been modeled using element type Link180 for the main reinforcement and the ties. It was assumed to be bilinear isotropic material. The modulus of elasticity of the steel (E_s) was taken as $2 * 10^5$ MPa and the Poisson's ratio (ν_s) was 0.3. The bilinear isotropic behavior of Link180 element satisfied by Von Mises failure criterion and requires the yield stress (f_y) and the hardening modulus (tangent modulus) of steel. The yield stress for the main reinforcement was taken as 420 MPa, while for the shear reinforcement was 250 MPa and the tangent modulus was taken as zero for the both.

3.1 Control Specimen

Solid 65 was used as element type for the three-dimensional modeling of solids as concrete representation. The defined Solid 65 element type is capable of cracking in tension and crushing in compression and was defined by eight nodes having three degrees of freedom at each node: translation in the nodal x, y and z directions. The brick element had dimensions of (20 x 20 x 20) mm at each side. The most important part of Solid 65 element is the treatment of nonlinear material properties. Figure 3. shows a typical view of eight nodes solid 65 element.

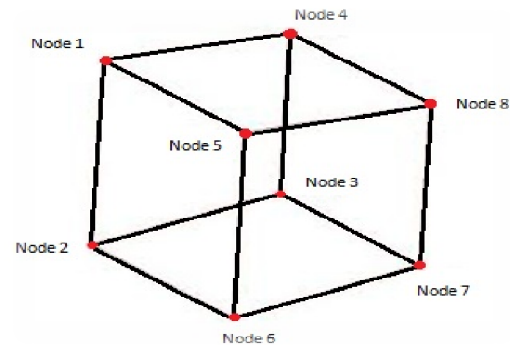


Figure 3. Typical view of eight nodes solid 65 element.

3.1.1 Modeling & Meshing

The reinforced concrete joint was modeled with dimensions (200 x 200 x 1600) mm and (200 x 200 x 600) mm which represent the column and the beam respectively. The meshing divided it into a number of small brick elements with (20 x 20 x 20) mm dimensions as shown in Figure 4.

Two methods are usually followed to model the concrete reinforcing. The first one is the smeared concrete element method. The second one is reinforcing discrete elements with geometrical properties similar to the original reinforcement which was used in this study. The reinforcement does not need any meshing because individual elements are formed in the modeling through the nodes created by concrete volumes. Figure 5. shows the reinforcement detailing for the control specimen.

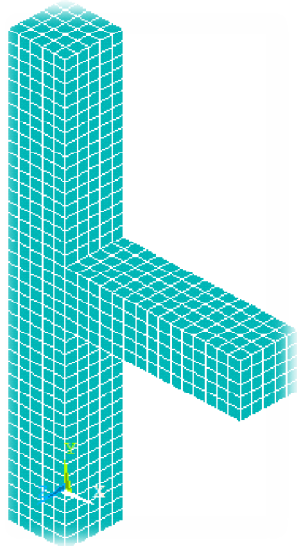


Figure 4. Mesh of the concrete volumes for the control specimen.

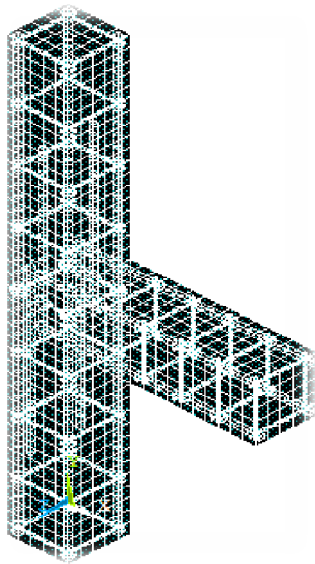


Figure 5. Reinforcement representation for the control specimen.

The model was built to be hinged at the column ends. Degrees of freedom to be constrained at the x, y and z directions with zero displacement value was applied at the nodes located at the top and bottom of the column as shown in Figure 6.

3.1.2 Loading

The applied load was performed as a static load at the free end of the cantilever beam as a small forces divided by the number of nodes at that location as shown in Figure 7. The forces at the two corners were reduced to the half value to make sure that the failure

did not happen due to concrete crushing at the edges of the beam.

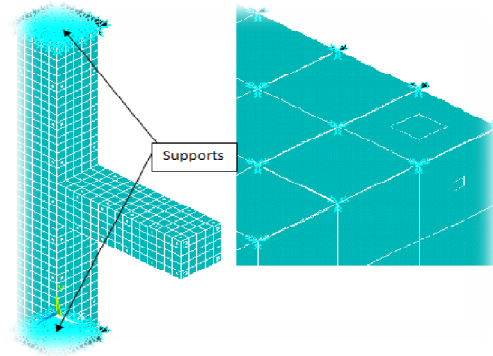


Figure 6. Restraints for control specimen.

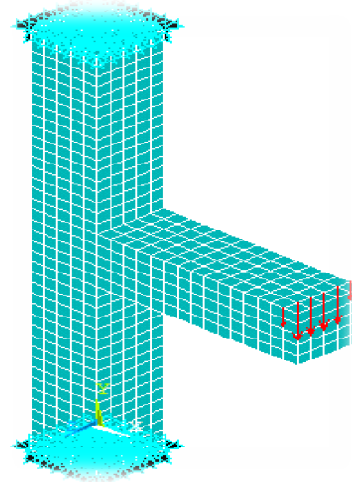


Figure 7. Static loading at the free end of the cantilever beam.

The same load was applied for all retrofitted specimen with small displacement static analysis option and time step size of 0.1 kN. The tolerance for the nonlinear solution was set to be 0.01 to make the analysis more convergence based on force analysis.

3.2 Concrete Jacketing

The first retrofitting technique that used in this study was concrete jacketing at the member level with different thicknesses at the joint region. The jacketing was done with 200 mm and 400 mm thickness at each face of the beam and the column at distance d using a concrete with compressive strength of 20 MPa and a steel mesh of 10 mm diameter with ultimate tensile strength of 250 MPa. The element types that used for modeling the concrete and the steel mesh were the same as the elements types used in modeling the concrete specimen. Solid 65 with brick dimensions (10 x 10 x 10) mm and Link 180 element types were used to model the concrete and the steel mesh respectively.

Figure 8. shows the concrete meshing and the steel mesh for the specimen jacketed by concrete with 200 mm thickness.

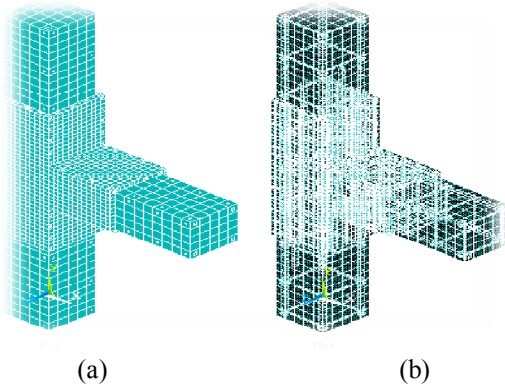


Figure 8.

(a) Meshing specimen retrofitted by 200 mm thickness concrete jacketing.

(b) Reinforcement representation for steel mesh at the joint region.

3.3 Steel Plate Jacketing

Steel plate was used as a member-level retrofitting technique in order to enhance the strength and the behavior of beam-column joints. The jacketing was done with 20 mm and 40 mm plate thickness at each face of the beam and the column at the joint region. The element type that used to model steel plate was SHELL 41 element type. The element is defined by four nodes and four thicknesses. Figure 9. shows the geometry, node locations and the coordinate system for SHELL 41 element type.

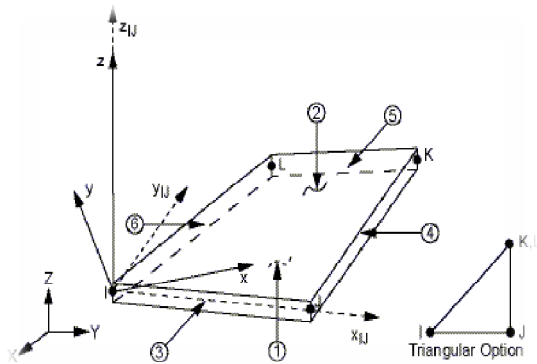


Figure 9. Typical view of SHELL 41 element type.

The steel plate was assumed to be linear isotropic material with E_s of 2×10^5 MPa and 0.3 Poisson's ratio. Figure 10. shows the steel plate representation with the main reinforcement for the jacketed specimen.

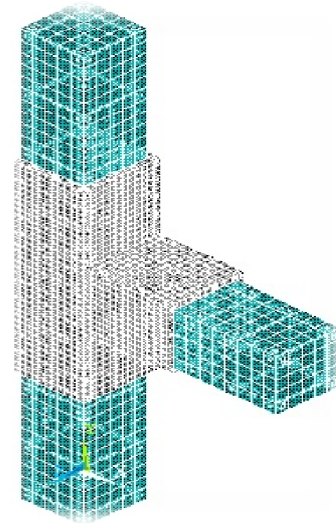


Figure 10. Steel plate representation for the jacketed specimen using SHELL 41.

3.4 CFRP Wrapping

The last retrofitting technique used in this study was Carbon Fiber Reinforced Polymer (CFRP) wrapping sheets shown in Figure 11.

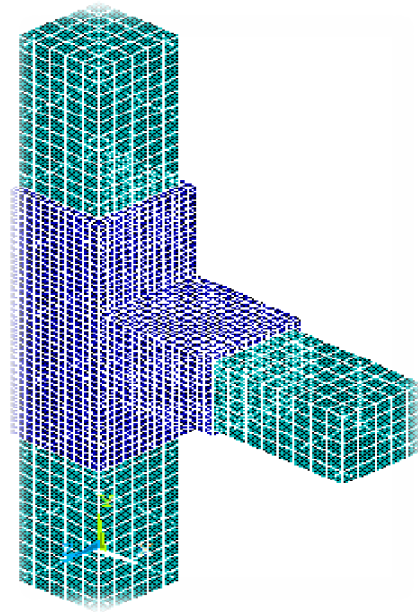


Figure 11. Representation of CFRP wrapped specimen using SHELL 41.

The material was modeled using SHELL 41 element type and assumed to be orthotropic material with thickness of 10 mm. The modulus of elasticity of CFRP was 2.3×10^5 MPa, Poisson's ratio of 0.3 and the ultimate stress (f_t) was considered to be 3400 MPa (Manufacturing properties).

4. Discussion of Results

4.1 Introduction

This section discusses the results of the original joint case and the retrofitted cases. A comparison between the carrying load capacity, ductility and the deflection results for the original and retrofitted specimens is held to identify the improvements and the effects that each retrofitting technique has accomplished.

4.2 Finite Element Analysis

The behavior of the retrofitted beam-column joints was investigated using finite element analysis method using ANSYS software. An increasing static load was applied at the free end of the cantilever beam to develop bending moment at the beam-column joint. The results from the analysis were compared between the retrofitting techniques and the control specimen to verify the effectiveness of retrofitting under loadings. The strengthen techniques is considered as member-level evaluation for the structure, and it was carried out on each original and retrofitted specimens in order to show the effect of the retrofitting technique on the strength and capacity of the joints and redistribution of the stresses along the joints.

4.2.1 Control Specimen

Non-linear analysis was done for the control specimen using the software ANSYS. An increasing static load was applied at the free end of the cantilever beam at a load interval of 1 kN up to control load of 20 kN to develop a bending moment at the joint. The maximum deflection was found to be 47.5 mm for the load of 20 kN. Ductility is measured in terms of displacement ductility; which is the ratio of the maximum deformation that the specimen can undergo without significant loss of initial yielding resistance to the initial yield deformation. The displacement results of analysis performed using finite element code ANSYS, version 14, have been used to calculate the displacement ductility. Where the displacement of the control specimen at the initial yielding was found to be 7.81 mm and the maximum deformation at the ultimate stage was 47.5 mm. The displacement ductility was found to be 6.08. The crack/crushing pattern in the specimen can be obtained using the crack/crushing plot option in finite element code ANSYS 14. In the non-linear region of the response, cracks occur as more loads are applied at the free end of the cantilever beam. The cracking pattern for the control specimen is shown in Figure 12. Once the steel reinforcement starts to yield, the displacements and the rotations of the specimen begin to increase at a higher rate as more load increments are applied, and the ability of the joint to distribute the load between the beam and the column has diminished greatly, as a result, greater deformation occurs at the joint region at the beam and columns corners. Tensile cracks were developed at the interface

between the column and the beam at the joint region. Cracks also have appeared at the cantilever beam and along the column which indicates that the failure due to the applied load was along the whole specimen with very large cracks intensity at the joint region due to lack shear reinforcement.

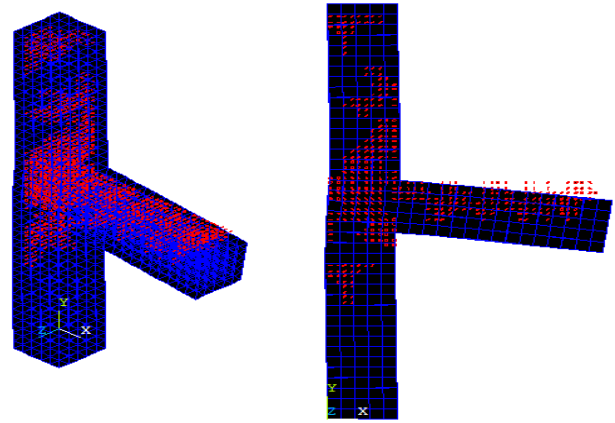


Figure 12. Cracks in the control specimen.

4.2.2 Concrete Jacketing of 200 mm

After performing the previous analysis on the beam-column joint retrofitted with 200 mm concrete jacketing. The use of this technique shows more advantages rather than the results obtained from the control specimen in some terms as shown in the following figures. Where, the load carrying capacity increased about 7.69%. However, it can be still noticed that reducing the maximum displacement of the specimen was not significantly large. Where, the maximum deflection was found to be 34.82 mm for the load of 20 kN. However, after comparing the use of concrete jacketing technique with the control specimen, the displacement ductility of this specimen has been decreased by 21.2% and the deflection has been decreased by 26.7%. It was observed that the displacement of the jacketed specimen at the initial yielding was found to be 8.92 mm, the maximum deformation at the ultimate stage was 42.69 mm and the displacement ductility was found to be 2.66.

The cracking pattern for the retrofitted specimen is shown in Figure 13. It shows that the ability of the joint to distribute the load between the beam and the column has become more efficient. As a result, greater deformation occurs at the joint region. Tensile cracks were developed at the jacketing region between the column and the beam. Cracks also have appeared with less intensity at the cantilever beam and along the column which indicates that the joint starts to take the load more effectively than the original case were the joint was not retrofitted and the failure due to the applied load was at the jacketing zone.

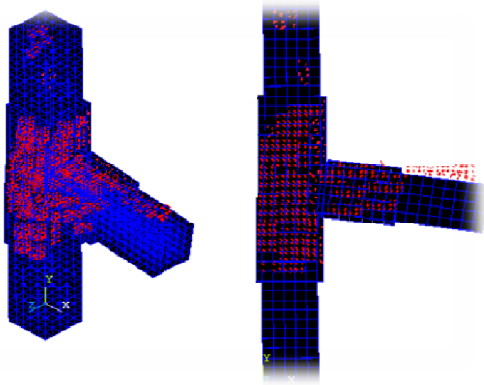


Figure 13. Cracks in the beam-column joint retrofitted with 200 mm concrete jacketing.

4.2.3 Concrete Jacketing of 400 mm

After increasing the jacketing thickness to reach a thickness of 400 mm. The same analysis was performed to see the effect of jacketing thickness in the performance of the specimen under increasing static load. The results shows that the increasing of jacketing thickness with same properties of the concrete and steel reinforcement could enhance the load carrying capacity when compared with the control specimen. Where, the load carrying capacity increased about 12.2%. The displacement at the initial yielding was found to be 8.92 mm were the maximum deformation at the ultimate stage was 23.7 mm and the displacement ductility was found to be 2.66. Decreasing in the ductility of this specimen of 56.25% has been observed. However, the reduction in deflection was found to be 59.03% which indicates that the reducing the maximum displacement of the specimen was not significant large.

The cracking pattern of the retrofitted specimen shown in Figure 14. demonstrates the ability of the joint to distribute the load between the beam and the column has become much better when compared with the control specimen and the specimen retrofitted with 200 mm concrete jacketing. Where, at the crushing load, all the cracks were developed at the jacketing region between the column and the beam. Cracks also have appeared with less intensity at the cantilever beam and along the column when compared with the specimen retrofitted with 200 mm concrete jacketing, which indicates that the joint starts to take the load more effectively and the failure due to the applied load was at the jacketing zone.

4.2.4 Steel Plate Jacketing of 20 mm

When performing the same analysis for the beam-column joint jacketed with 20 mm thickness steel plate, a proper increase in the load carrying capacity of about 13.04% more than the result obtained in the case of control specimen. This indicates that the use of steel jacketing as a retrofitting technique is better option

than the use of concrete jacketing. However, the displacement at the initial yielding was found to be 6.89 mm were the maximum deformation at the ultimate stage was 31.1 mm and the displacement ductility was found to be 4.51. The reduction in deflection that observed was 44.47% which is better than the use of concrete jacketing with the same thickness but not in the case of larger concrete jacketing thickness. In the non-linear region of the response, cracks start to appear as more loads are applied at the free end of the cantilever beam. The cracking pattern for the retrofitted specimen is shown in Figure 15.

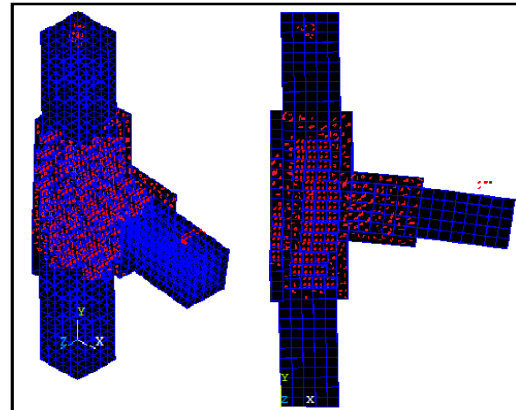


Figure 14. Cracks in the beam-column joint retrofitted with 400 mm concrete jacketing.

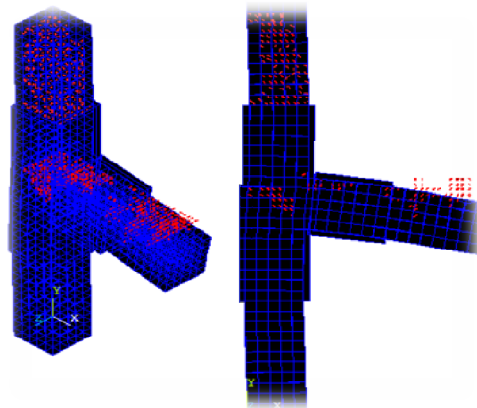


Figure 15. Cracks in the beam-column joint retrofitted with 20 mm thickness steel plates.

Once the steel plate starts to yield, the displacements and the rotations of the specimen begin to increase at a higher rate as more load increments are applied. It was observed that the cracks have started to move away from the joint region and formed with large intensity at the cantilever beam and along the column which indicates that the failure due to the applied load was not concentrated at the joint region as the case of

concrete jacketing technique. So, it can be considered as a good joint strengthen technique.

4.2.5 Steel Plate Jacketing of 40 mm

After performing the previous analysis on the beam-column joint and using a steel plate with 40 mm thickness. The analysis shows more significant improvements when increasing the steel plate thickness. The maximum deflection was found to be 10.3 mm for the load of 20 kN. The non-linear analysis shows an increase in the load carrying capacity of about 28.7% when compared with the control specimen. And a reduction in the deflection of about 78.3% was observed. These results show the effect of thickness when using steel plate jacketing technique. Where, it shows much better improvements in the behavior of the joint. The cracking pattern for the retrofitted specimen is shown in Figure 16.

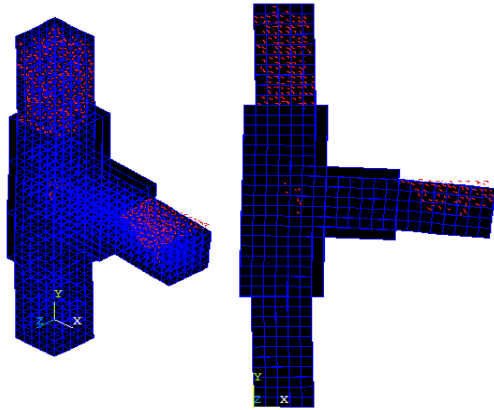


Figure 16. Cracks in the beam-column joint retrofitted with 40 mm thickness steel plate.

It shows that the main target of the joint to distribute the load between the beam and the column has reached. It was observed that the cracks are completely formed away from the joint region and formed with large intensity at the cantilever beam and along the column which indicates that the failure due to the applied load was not concentrated at the joint region as the case of concrete jacketing technique and the joint has behaved in a better way when compared with the specimen retrofitted with 20 mm steel plate thickness which again shows the effect of thickness. So, it can be considered as a good joint strengthen technique.

4.2.6 CFRP Wrapping

The final retrofitting technique that used to reach at a good beam-column joint behavior was CFRP wrapping. The results show that the use of this technique could increase the load carrying capacity when compared with the control specimen or any retrofitting techniques used. Where, the load carrying capacity has increased about 33.3% when compared

with the control specimen. However, the maximum displacement at the load of 20 kN was found to be 16.83 mm which is larger than the case of use steel plate with 40 mm thickness. It is observed that the ductility of this specimen have a decrease of 41.6%. The displacement of the specimen at the initial yielding was found to be 6.48mm were the maximum deformation at the ultimate stage was 23 mm and the displacement ductility was found to be 3.55.

The cracking pattern for the retrofitted specimen is shown in Figure 17.

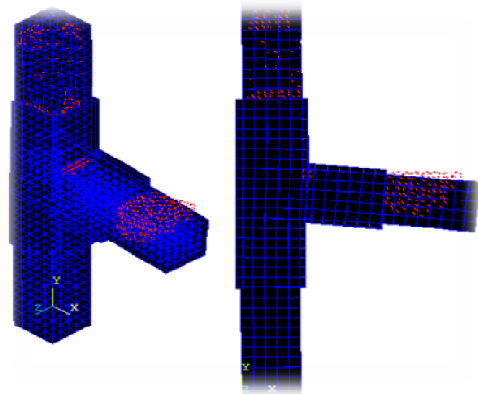


Figure 17. Cracks in the beam-column joint wrapped by CFRP.

It shows that the use of this technique makes the joint to behave like the previous case where the retrofitting technique was steel plate wrapping with 40 mm thickness. The main target of the joint to distribute the load between the beam and the column has reached. The cracks are completely formed away from the joint region and formed with large intensity at the cantilever beam which is the most preferred type of failure. This indicates that the failure due to the applied load was not concentrated at the joint region.

5. Conclusions

The following conclusions were made based on the results of this study:

1. The load carrying capacity of the beam-column joint specimen retrofitted by 200 mm and 400 mm concrete jacketing was found to be 7.69% and 12.2% respectively more than the control specimen.
2. The load carrying capacity of the specimen retrofitted by 20 mm and 40 mm steel plate jacketing was found to be 13.04% and 28.7% respectively more than the control.
3. The load carrying capacity of the beam-column joint specimen retrofitted by CFRP wrapping sheets was found to be 33.3% more than the control.
4. The reduction in deflection for the specimen retrofitted by 200 mm and 400 mm concrete jacketing

was found to be 26.7% and 59.03% respectively when compared with the control specimen.

5. The reduction in deflection for the specimen retrofitted by 20 mm and 40 mm steel plate jacketing was found to be 44.47% and 78.3% respectively when compared with the control specimen.

6. The reduction in deflection for the specimen retrofitted CFRP wrapping sheets was found to be 64.56% when compared with the control specimen.

7. When comparing the effect of thickness between the concrete jacketing and steel plate jacketing techniques, the results shows that the use of steel plate technique with smaller thickness could enhance the beam-column joint carrying load capacity more than the use of concrete jacketing with doubled thickness.

8. The results show that the use of CFRP wrapping sheets can be considered as the best retrofitting technique compared to the other two techniques when the increasing of the load carrying capacity is the target. While, the use of Steel plate jacketing with proper thickness is the best option when the decreasing of maximum deflection is the target.

9. The results show that the displacement at the initial yielding for the specimen retrofitted by concrete jacketing was large than the control specimen, while, was smaller when using steel plate jacketing and CFRP wrapping techniques.

10. The failure was along the beam and the column portion of the joint of the control specimen which is to be avoided. In the case of concrete jacketed specimens, the failure was at the jacketing zone. In the case of steel plate jacketing, the failure was away from the joint and the cracks have concentrated at the beam and the upper part of the column, while, in the case of wrapped specimen using CFRP sheets, the failure was noticed in the beam portion only and this is the most preferred type of failure.

11. More retrofitting techniques can be compared to the mentioned retrofitting techniques, such as adding stainless steel instead of steel plates, diagonal cross bracing bars and using other types of (FRP) materials such as Glass and Aluminum fiber reinforced polymer.

12. Another type of analysis can be done for this type of structures with these retrofitting techniques which is the seismic behavior instead of the behavior under static load.

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