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Seismic Performance of UHPFRC-Strengthened RC Beam–Column Joints Using Damage Plasticity Model—A Numerical Study

K. Sai Kubair and J. S. Kalyana Rama

1 Introduction

Beam–column joints are the critical sections in any structure especially when they are subjected to lateral loads. Under severe shaking, beam–column joints become vulnerable to cracking and failure. Several studies have been carried out to strengthen the beam–column joints using materials like fiber-reinforced polymer (FRPs). Ultra High-Performance Fiber Reinforced Concrete (UHPFRC) is one of the recent findings in the last decade, which can be used as a potential strengthening material of reinforced concrete structures. Tayeh et al. [1] explored the possibility of using UHPFRC as a suitable rehabilitation material for a conventional structure subjected to seismic load. The results indicated that UHPFRC is proven to be an excellent material for repair and rehabilitation because of its enhanced durability over other concretes and its low porosity characteristics. Also, the working time of UHPFRC makes it advantageous to be used as a rehabilitation material. Lampropoulos et al. [2] assessed the efficiency of strengthening a reinforced concrete (RC) beam using UHPFRC. Full-scale experimental study on the strengthened beams had been performed using three different strengthening techniques. The results obtained from the study indicated that UHPFRC was efficient in improving the load-carrying capacity of the RC beam. Prem et al. [3] investigated the flexural performance of the pre-damaged RC beam with varying cross-section using UHPFRC. UHPFRC strip was attached to the tension face of RC beam with epoxy, and the flexural testing was carried out. The results obtained after testing these beams indicated that UHPFRC was efficient in improving the strength properties of the damaged RC beam to a greater extent. Rahman et al. [4] documented several investigations on the use of UHPFRC

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for the construction of high-rise structures, retrofitting and rehabilitation of structures. Chen et al. [5] studied the structural performance of UHPFRC I-Girders using concrete damage plasticity model of a finite element software ABAQUS/CAE. From the results obtained, it was concluded that the CDP model was efficient in studying the linear and non-linear response of concrete structures. Sai Kubair et al. [6] assessed the performance of RC-framed structure strengthened using UHPFRC with varying thickness. Results indicated that UHPFRC strip with 20 mm thickness enhanced the load-carrying capacity of the structure.

From the existing studies, it can be noted that UHPFRC serves a good material in terms of both strength and durability and can be used for strengthening, retrofitting and rehabilitation of structures because of its enhanced ductility. Concrete damage plasticity model has also gained popularity in the last decade and the same is used in the present study. The influence of variation in the CDP parameters on the behavior of RC beam–column joint has been studied numerically using ABAQUS/CAE. An attempt has been made to assess the nonlinear performance of UHPFRC strip in strengthening RC beam–column joints using the CDP model. The effect of variation in the thickness of the UHPFRC strip on the load-carrying capacity of the RC beam–column joint is also addressed in the study.

2 Modeling Concrete Using the CDP Model

2.1 Compressive Behavior of Concrete

Elastic modulus of concrete: According to IS: 456:2000, the elastic modulus of a particular grade of concrete can be calculated using

$$E_{cm} = 5000(f_{ck})^{0.5} \tag{1}$$

where f_{ck} is the characteristic compressive strength of concrete.

HSU and HSU model: This model is used to generate the stress–strain curve of concrete up to a point in the descending part where the stress is equal to 0.3 times the peak stress. The yield stress is equal to half the peak stress value. The maximum strength for this model is 62 MPa. The formulations involved in this model for normal strength concrete are as given in Eqs. 2 and 3

$$\sigma_c = E_{cm}\varepsilon_c \tag{2}$$

Equation (2) is valid up to yield point

$$\sigma_c = \left(\frac{\beta\frac{\varepsilon_c}{\varepsilon_o}}{\beta - 1 + (\frac{\varepsilon_c}{\varepsilon_o})^{\beta}}\right)\sigma_{cu} \tag{3}$$

Equation (3) is used post the yield point.

Where ε_c and ε_o are the strain at any point and strain at peak stress in concrete, respectively, σ_{cu} is the peak stress in concrete in kip/in^2 and β is a parameter that decides the nature of the stress–strain curve. These parameters are obtained using Eqs. 4 and 5.

$$\varepsilon_0 = 8.9 \times 10^{-5}\sigma_{\text{cu}} + 2.114 \times 10^{-3} \tag{4}$$

$$\beta = \frac{1}{1 - \left(\frac{\sigma_{cu}}{E_{cm}\varepsilon_o}\right)} \tag{5}$$

2.2 Tensile Behavior of Concrete

The maximum tensile strength of concrete is calculated based on the formula mentioned in the EUROCODE 2 is given by

$$f_{ctm} = 0.3(f_{ck})^{2/3} \tag{6}$$

where f_{ctm} is the maximum tensile strength of concrete.

This value is given as an input in the concrete damage plasticity (CDP) model in the tension part and the maximum cracking strain value of concrete is taken as a constant equal to 0.01. These two values are given as an input in the tension part of the CDP model.

2.3 Plasticity Parameters in the CDP Model

There are some parameters used for addressing the plasticity nature of concrete in the CDP model. The values of these parameters used in the present study in modeling concrete using the CDP model are shown (Table 1).

2.4 Predicting the Damage Variables in the CDP Model

The damage variable in compression (dc) is calculated based on the damage theory as the ratio of inelastic strain in compression (crushing strain) at a particular point to that of the maximum strain allowed in concrete

Table 1 Plasticity parameters considered in the CDP model

Parameter	Value
Ψ	34^0
K_c	2/3
f_{b0}/f_{c0}	1.16
ε	0.01
μ	0

$$d_c = \frac{\varepsilon_c^{in}}{\varepsilon_c^{max}} \tag{7}$$

where, ε_{cu} is the maximum strain in compression that can be allowed in concrete calculated as per the HSU and HSU model.

The maximum value of damage variable in tension (d_t) is again taken as a constant, which is equal to 0.9 and the damage value at the yield stress will be equal to zero.

2.5 Modeling Steel Using the Plasticity Theory

Steel reinforcement inside the beam–column joint is modeled using the plastic theory of metals, which requires yield stress and plastic strain values of steel as an input. These values are taken based on direct tension test performed on steel. Fe 415 is considered for the entire analysis of the present study. The data considered for modeling the steel reinforcement are given in Table 2.

Table 2 Yield stress and plastic strain data of Fe 415 steel

Yield stress (MPa)	Plastic strain
332	0
352	0.0001
373	0.0003
394	0.001
435	0.002
435	0.003
440	0.005
435	0.01
400	0.03
370	0.06

2.6 *Modeling UHPFRC Using the CDP Model*

UHPFRC generally has higher performance in terms of strength and workability when compared with that of normal concrete. UHPFRC is modeled with a varying thickness similar to that of plain concrete and the corresponding linear and non-linear material properties are used. The data required for modeling this concrete using CDP are taken from an experiment conducted by Prem et al. [3] and the details are given in Table 3. Density: 2400 kg/m^3, elastic modulus: 40000 MPa, Poisson's ratio: 0.18.

3 Designing the RC Beam–Column Joint

The beam–column joint used in the present study for analysis is designed as per IS: 13,920:2016, the Indian Standard Code of Practice for earthquake-resistant design of structures. The grade of concrete used for analysis is M20. The cross-sectional dimensions of beam and column considered are 300 mm × 450 mm and 300 mm × 530 mm, respectively. The beam–column joint is designed in STAAD PRO V8i according to IS: 1893 for earthquake-resistant design. The parameters considered for the design purpose are as follows:

i. Earthquake zone: V.
ii. Soil type: loose soil.
iii. Damping ratio: 5%.
iv. It is considered to be a highly important structural component.

Table 3 CDP data for UHPFRC

Compression behavior of UHPFRC			Tension behavior of UHPFRC			
Compressive stress (MPa)	Inelastic strain	Damage parameter	Tensile stress (MPa)	Crack opening (mm)	Damage parameter	Displacement, (mm)
107.33	0	0.000	13.5	0	0	0
114.65	0.0032	0.032	5.5	1.235	0.564	(1.983
124.43	0.0038	0.123	0	3.786	0.988	3.786
113.32	0.0044	0.221				
93.54	0.0054	0.343				
64.76	0.0063	0.564				
41.32	0.0074	0.724				
25.92	0.0083	0.872				
14.41	0.0092	0.954				
9.76	0.012	0.972				

The occurrence of the earthquake is assumed to be in all four possible directions, i.e. + X, −X, +Z & −Z. The corresponding earthquake load definitions are assigned to the structure. The static load details used for this design are as follows:

i. *Dead loads (DL):*Self-weight = wall load = 12 KN/m.
- *Live loads (LL):*

i. Floor load = 3 KN/m^2.

Based on IS: 1893—Part-II, all the load combinations are generated depending on the nature of the general load cases that are applied to the structure.

3.1 RCC Design Details of the Beam–Column Joint

The details of reinforcement that are provided inside beams and columns of the beam–column joint are given in Tables 4 and 5.

4 Finite Element Modeling of the Beam–Column Joint

This beam–column joint designed as per IS: 13,920:2016 is modeled in ABAQUS/CAE. Beams and columns are modeled using solid element and the reinforcing bars, stirrups and links are modeled using wire elements. The concrete part is meshed using the C3D8R (eight-noded linear brick) element and the reinforcement (steel bars and stirrups) are meshed using the T3D2 (two-noded truss) element. A

Table 4 Beam reinforcement details in the beam–column joint

S. No	Grade of concrete (MPa)	Beam reinforcement details		
		Top reinforcement (mm^2)	Bottom reinforcement (mm^2)	Stirrups
1	M20	4–16 ϕ	4–22 ϕ	8 mm ϕ@200 mm

Table 5 Column reinforcement details in the beam–column joint

S. No	Grade of concrete (MPa)	Location of links	Column reinforcement details	
			Main Reinf (mm^2)	Links
1	M20	Near the joint	6–20 ϕ	8 mm ϕ@150 mm
2		Away from the joint		8 mm ϕ@80 mm

Reinf. means Reinforcement

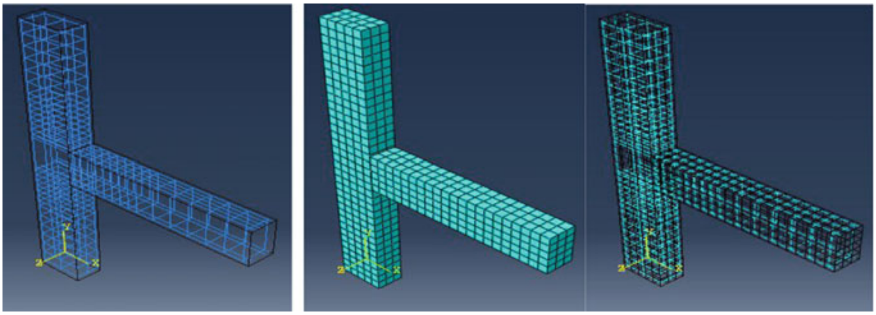

Fig. 1 Reinforcement layout and meshing of concrete and steel reinforcement

constant element size of 100 is maintained throughout the analysis for all the components of the structure. The reinforcement assembly inside the beam column, the mesh details of the beam–column joint and the reinforcement are shown in Fig. 1.

4.1 Loading Details

The response of beam–column joint is studied by subjecting it to Elcentro, 1940 earthquake ground motion at the Imperial Valley in Southern California. The load is applied as a ground acceleration at the base of the beam–column joint. First 5 s of the data is used for simulating the joint as the peak ground acceleration is maximum in the first 5 s. The acceleration time history of the Elcentro earthquake considered in the present study is shown in Fig. 2.

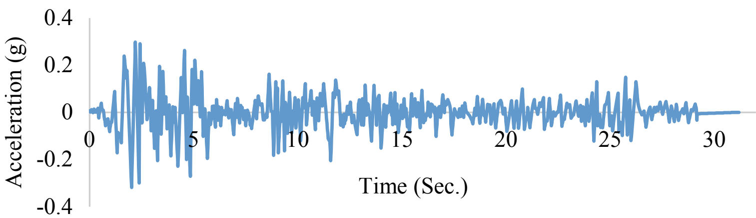

Fig. 2 Elcentro earthquake time history data

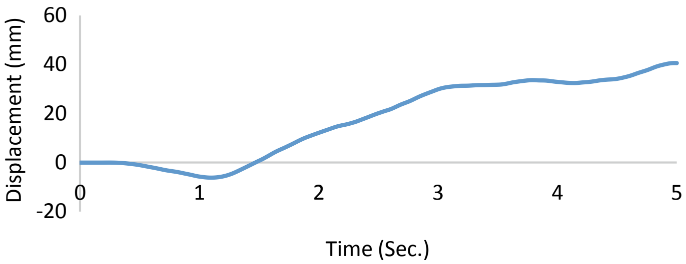

Fig. 3 Displacement—time response of the beam–column joint for 5 s Elcentro earthquake

4.2 Response of the Beam–Column Joint

The RC beam–column joint modeled based on a finite element approach using ABAQUS is assessed for its seismic response by subjecting it to Elcentro earthquake. The displacement–time response of the beam–column joint when excited with this earthquake of 5 s duration is shown in Fig. 3.

It can be observed that the beam–column joint has undergone a backward sway during the first one and half seconds of the earthquake approximately and a forward sway thereafter. The displacement of the beam–column joint is negative when the ground acceleration is negative and kept on increasing as the ground acceleration increases.

The displacement of the beam–column joint is almost constant when the ground acceleration is reduced. There are some cracks observed at the center and the base of the beam–column joint, which are the most critical parts. The reinforcement at the base of the beam–column joint also yielded under earthquake ground excitation. Cracks in the concrete of the beam–column joint appeared at the base first and later propagated to the center of the joint. This behavior of the beam–column joint obtained is as expected and it can be said that the CDP model is effective in predicting the behavior of beam–column joint subjected to seismic loads. The failure patterns in concrete and steel due to earthquake loads are as shown in Fig. 4.

5 Parametric Studies on the Behavior of the Beam–Column Joint

5.1 Dilation Angle

The influence of varying dilation angle of concrete on the behavior of the beam–column joint is also investigated. Five different dilation angles 13, 20, 34, 36 and 40° are chosen for the study. All other parameters during this analysis are kept constant

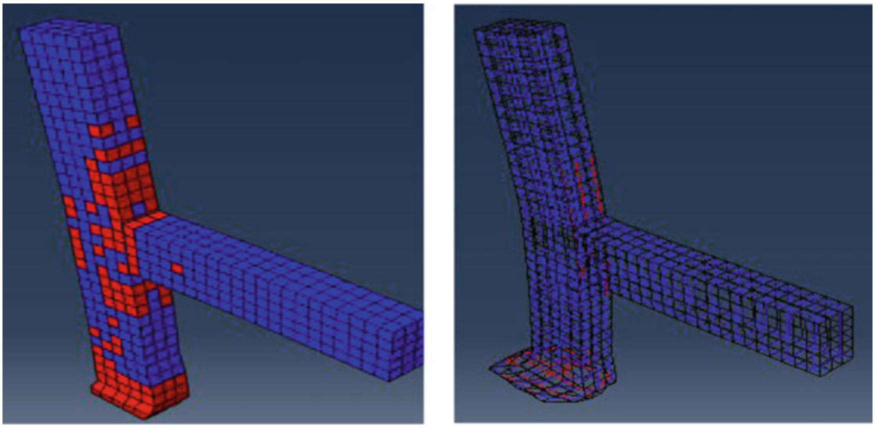

Fig. 4 Cracks in concrete and yielding of steel due to the seismic load

as shown in Table 1. The displacement–time responses of the beam–column joint due to variation in the dilation angle of concrete are shown in Fig. 5.

It is observed that the increase in dilation angle increased the displacement of beam column. The difference in the displacement values is less when the dilation angles are 13 and 20°. It can also be noted that the nature of displacement–time responses of the beam–column joint with dilation angles of 34, 36 and 40° is similar. ABAQUS User's Manual [7] suggests the dilation angles to be any value in between 34 and 40°. So, any value among 34, 36 and 40° can be considered for analyzing the beam–column joint subjected to earthquake load as these values are almost same.

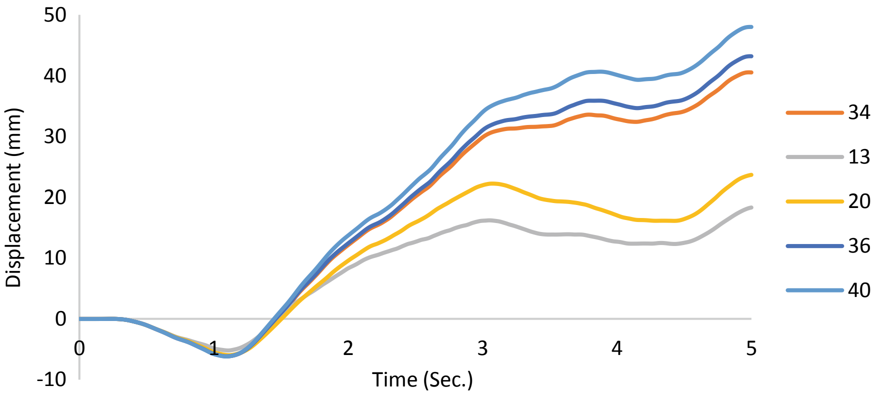

Fig. 5 Displacement—time response of the beam–column joint for various dilation angles

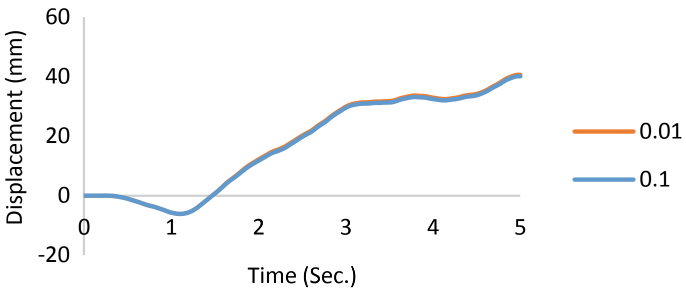

Fig. 6 Displacement—time response of the beam–column joint for two different eccentricities

5.2 Eccentricity

The effect of the varying eccentricity of concrete on the behavior of the beam–column joint is analyzed. Two eccentricity values suggested for concrete 0.01 and 0.1 are considered for the study. The displacement–time responses of the beam–column joint due to variation in the eccentricity of concrete are shown in Fig. 6. From the analysis, it is observed that there is a negligible variation in the response using two values of eccentricity.

6 Using UHPFRC for Strengthening the RC Beam–Column Joint

Based on the CDP data mentioned above, UHPFRC in the form of strip is modeled in ABAQUS and wrapped around the beam–column joint. It is wrapped up to half length of the column and half length of the beam. Generally, these types of strips are attached using epoxy adhesives, etc. But, in that case, there might be a problem of delamination of the strip. But in this study, the strip is directly tied to the beam–column joint allowing no slippage, which means that the delamination effects are neglected. The plasticity parameters for normal concrete and the UHPFRC are taken from Table 1.

Three different thicknesses of the UHPFRC strips, i.e. 5, 10 and 20 mm are considered for strengthening. The wrapped UHPFRC strip and the assembly of the beam–column joint with the strip are shown in Fig. 7. The strip is meshed using the eight-noded cubic element (C3D8R). The element size used for meshing the strip is also 100 mm so as to maintain a node-to-node connectivity. The mesh details of the strip are shown in Fig. 7. The beam–column joint strengthened with UHPFRC is tested for its seismic performance by applying a first 5 s Elcentro earthquake ground acceleration at its base. The results obtained after the analysis are shown in Fig. 8.

From Fig. 8, it can be noted that as the thickness of the UHPFRC strip increases, the displacement of the beam–column joint decreases. This means that as the thickness

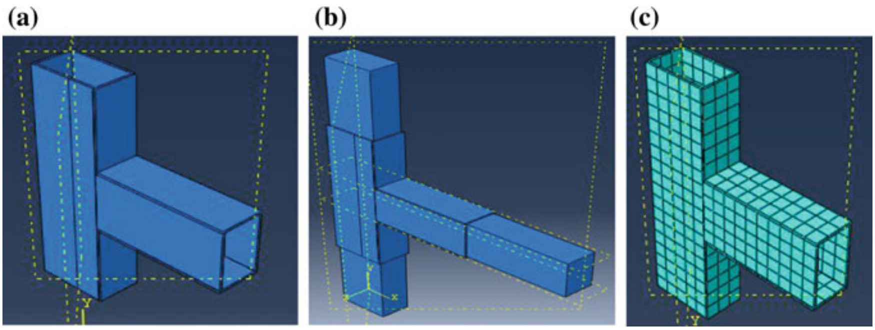

Fig. 7 **a** Wrapped HPFRC strip. **b** Assembly of the beam column joint and **c** Meshing the UHPFRC strip

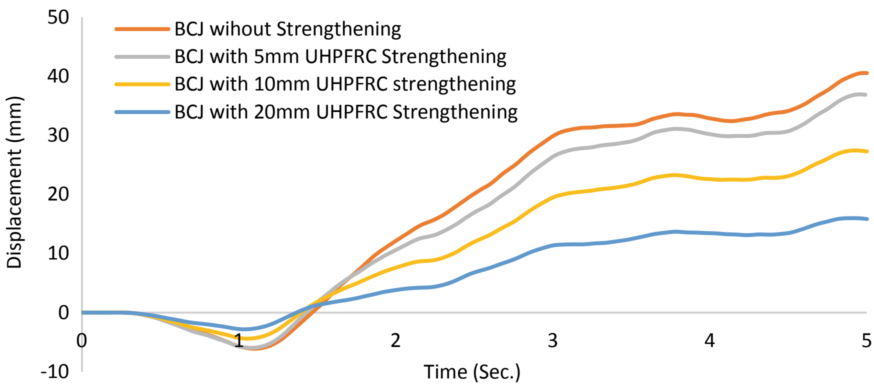

Fig. 8 Responses of the beam–column joint for various thicknesses of strengthening with UHPFRC

of the strip increases there is an improvement in the performance of the beam–column joint against earthquake loads. There is also a sudden improvement in the performance of the beam–column joint when it is strengthened using 10 and 20 mm UHPFRC strips when compared with the beam–column joint without strengthening.

The nature of the response is observed to be the same, irrespective of the thickness of strip used for strengthening the beam–column joint. Similar crack patterns as in the case of beam–column joint without strengthening are observed, i.e. cracks appeared at the base of the beam–column joint first and then they propagated to the center. From the failure patterns, it can be said that the ductility of UHPFRC material is more than that of normal concrete as the first crack in UHPFRC strip after the cracking of normal concrete at the center of the strengthened beam–column joint. The failure patterns in the strengthened beam–column joint and the UHPFRC strip are shown in Fig. 9.

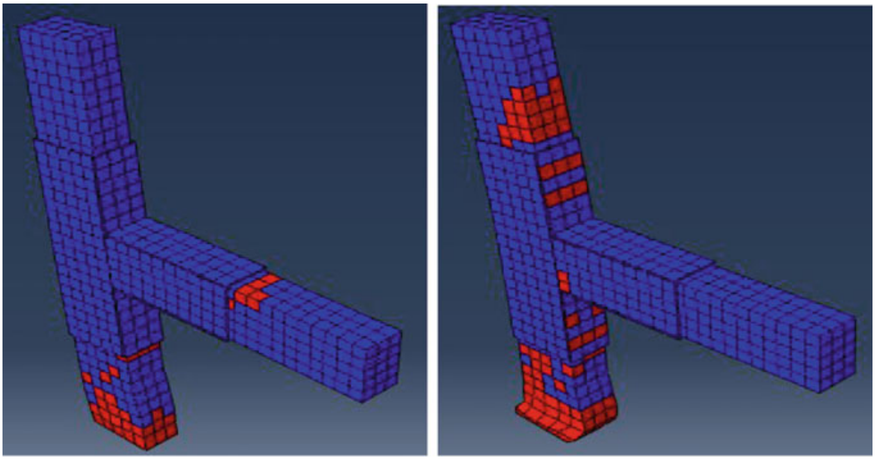

Fig. 9 Failure patterns in the strengthened beam–column joint and UHPFRC strip

7 Conclusions

1. From the results obtained, it can be concluded that the CDP model worked out well in predicting the behavior of RC beam–column joints subjected to seismic loads.
2. The variation in the dilation angle of concrete affected the performance of beam–column joint subjected to seismic loads. The displacement–time response values and nature are close to each other when the dilation angles are 34, 36 and 40°. The difference in responses of the beam–column joint is also less in the cases when the dilation angles are 13 and 20°.
3. There is no effect of variation in the eccentricity values on the performance of the beam–column joint. The displacement–time responses are almost same for eccentricity equal to 0.1 and 0.01.
4. Ultra High-Performance Fiber Reinforced Concrete can be successfully used to strengthen RC beam–column joints subjected seismic loads.
5. As the thickness of the UHPFRC strip used for strengthening the RC beam–column joint increases, the performance of the beam–column joint against seismic loads improved.
6. From the failure patterns observed in the strengthened beam–column joint, it can be concluded that this UHPFRC has higher ductility when compared with normal concrete and hence can be used to strengthen structures effectively in earthquake-prone areas.

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References

1. Tayeh BA, Bakar BA, Johari MM, Voo YL (2013) Utilization of ultra-high performance fibre concrete (UHPFC) for rehabilitation–a review. Procedia Eng 54:525–538
2. Lampropoulos AP, Paschalis SA, Tsioulou OT, Dritsos SE (2016) Strengthening of reinforced concrete beams using ultra high performance fibre reinforced concrete (UHPFRC). Eng Struct 106:370–384
3. Prem PR, Murthy AR, Ramesh G, Bharatkumar BH, Iyer NR (2015) Flexural behaviour of damaged RC beams strengthened with ultra high performance concrete. Adv Struct Eng. Springer India 2057–2069
4. Rahman S, Molyneaux T, Patnaikuni I (2005) Ultra high performance concrete: recent applications and research. Aust J Civ Eng 2(1):13–20
5. Chen L, Graybeal BA (2011) Modeling structural performance of ultrahigh performance concrete I-girders. J Bridg Eng 17(5):754–764
6. Kota SK, Rama JS, Murthy AR (2019) Strengthening RC frames subjected to lateral load with Ultra High-Performance fiber reinforced concrete using damage plasticity model. Earthq Struct 17(2):221–232
7. Dassault (2014) SIMULIA Abaqus analysis user's manual version 6.14. Dassault Systems