



Review on the Rehabilitation and Strengthening of Reinforced Concrete Beam-Column Joints Using Conventional and Advanced Retrofitting Techniques

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ABSTRACT

The beam-column joints in reinforced concrete (RC) moment resisting frame structures are the most critical and vulnerable under seismic loading. Over the past decades, joint failure (diagonal shear cracking, bond loss and brittle collapse) has been recognized as one of the most common causes of casualty and economic loss during earthquake reconnaissance. In this paper, the authors have identified and gathered experimental, analytical and numerical evidence from about 50 published studies, from 1975 to 2024, and synthesized them in this comprehensive review of conventional and advanced retrofitting strategies. The conventional methods discussed are: RC jacketing, steel jacketing and fibre-reinforced polymer (FRP) wrapping with carbon (CFRP), glass (GFRP), aramid (AFRP) and basalt (BFRP) composites. Advanced techniques include near surface mounted (NSM) FRP bars, textile-reinforced concrete (TRC), ultra-high performance fibre-reinforced concrete (UHPFRC) overlays, engineered cementitious composite (ECC/SHCC) and nanotechnology-based systems. Comparative performance parameters such as shear strength, ductility, energy dissipation, stiffness degradation and residual drift are tabulated and discussed in the context of design codes (ACI 318-19, ACI 440.2R-17, EN 1998-3, IS 13920, ASCE 41-17). There are eight schematic figures and five data tables provided. The review pinpoints key areas for further study: full-scale testing, codification of hybrid systems, long-term durability, and assessment of the sustainability of hybrid systems.

1. Introduction


The most common structural system used in multi-storey buildings worldwide, especially in earthquake prone areas is Reinforced Concrete (RC) Moment Resisting Frames. Beam-column joint is a joint in between horizontal and vertical load bearing members, acting as an intersection area in which the forces and moments are transmitted between

the members. Similarly, failure at the joint level can lead to progressive storey collapse, as have been observed [1, 2, 3] in the San Fernando (1971), Northridge (1994), Kobe (1995) and Turkey-Syria (2023) earthquakes.

Typical characteristics of buildings built before the advent of modern seismic codes (pre-1970s in North America, pre-1980s in Europe and Asia) include no transverse

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reinforcement within the joint core, vertical bar anchorage, and design with little or no ductility provisions, based primarily on gravity loads. In supposedly seismic compliant buildings, the joint may have been under-designed, since the older code provisions failed to account for the complex shear transfer mechanisms. [2, 3, 4]

The economic impact of joint failure following the earthquake is very huge. The earthquake-induced damage to existing RC buildings results in direct losses of more than USD 100 billion annually worldwide, and significant part of these losses are due to under-designed beam-column joints. Potential for rehab over demolition is desired from economic, sustainability and heritage perspectives, which has led to decades of research to find effective retrofiting solutions. [5, 6, 7]

This has changed the scene of retrofiting with recent developments: since the 1990s, the large scale commercialization of FRP systems, the development of new materials that promise more than conventional retrofiting methods (SMA, UHPFRC, ECC), sustainability issues and life cycle assessment, and advancement in computational modelling for condition-based retrofit decision making.[8, 9, 10, 7]

1.1 Scope and Objectives

The goals of this paper are: (i) to outline the main failure modes of RC beam-column joints subjected to cyclic loadings, (ii) to summarize the experimental and numerical data for conventional retrofiting methods, (iii) to evaluate the performance of advanced and emerging materials, and (iv) to compare the systems to provisions in recognized design codes and to illustrate research gaps. Only cast in place RC frames are considered;

prestressed, precast and steel-composite systems are not. [6]

1.2 Review Methodology

A systematic literature search was performed on Scopus, Web of Science and Google Scholar from 1975 to 2024. The keywords used for the search were: 'beam-column joint retrofit' 'RC joint strengthening' 'FRP joint' 'UHPFRC joint' 'SMA joint' and 'ECC seismic joint'. 250 candidate papers were screened and 120 of them were chosen for full-text screening and 50 of them are cited directly in this work. The preference was for peer-reviewed journal articles and where necessary, conference papers and technical reports with primary experimental data were included.

2. Structural Behavior and Failure Modes of RC Beam-Column Joints

2.1 Mechanics of the Joint Panel Zone

Under lateral load, the joint panel zone, defined by extensions of beams and columns framing into the joint, has a complex multi-axial stress state. If a beam is subjected to a horizontal seismic force which reverses periodically, the bending moments in the beam longitudinal bars at the two facing surfaces are in opposite directions. This creates a diagonal compression strut and a truss mechanism in the joint core in addition to shear and axial forces from the columns. [1, 3]

The horizontal joint shear force V_{jh} , which is equal to the tensile force in beam longitudinal bars, T_b , less the column shear, V_c . According to ACI 352R-02, the shear stress at the interface $\tau_{jh} = V_{jh} / (b_j * h_c)$ is limited to a range between $1.0 * \sqrt{f_c}$ and $1.7 * \sqrt{f_c}$ MPa, depending on the geometry of the joint and the confinement. [3]

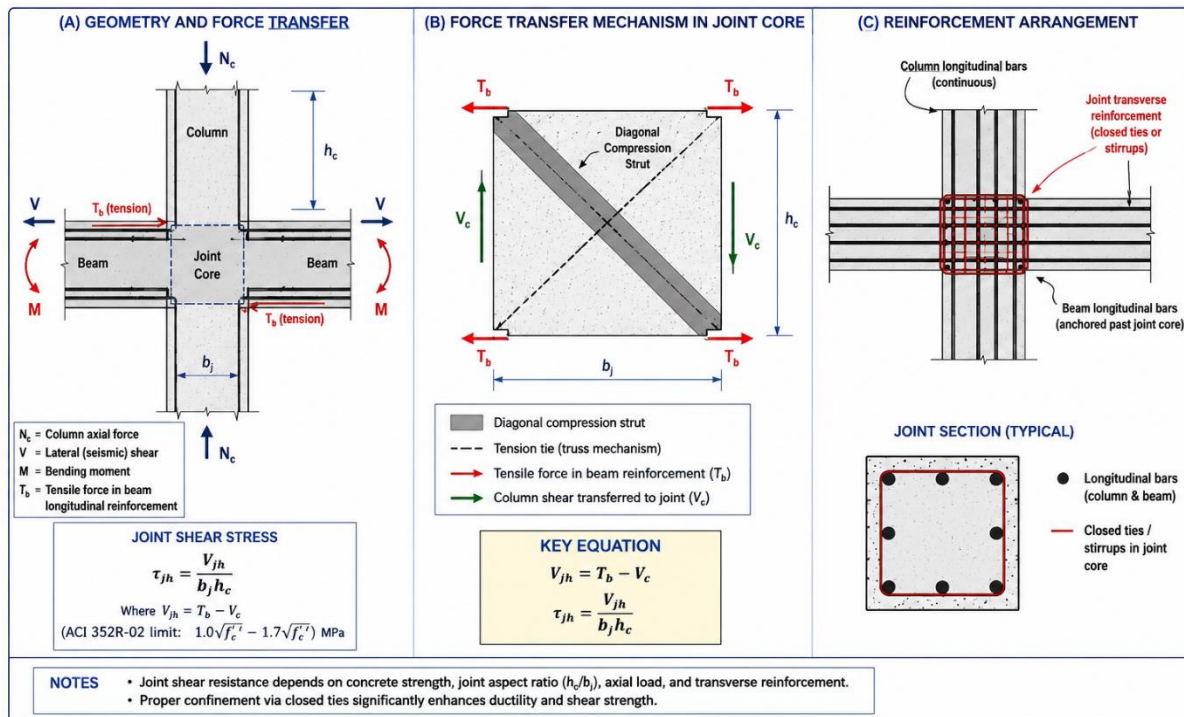


Figure 1. Typical RC interior beam-column joint schematic showing force transfer mechanisms, shear stress formulation, and reinforcement arrangement.

2.2 Classification of Joints

Joints are considered to be geometrically interior (4 beams), exterior or T (2 or 3 beams), or corner or knee (1 beam per direction). Beams on all four sides of the joints provide confinement which increases the shear capacity and ductility of the interior joints. Unconfined faces decrease the available concrete strut area and passive confinement, and exterior and corner joints are more susceptible. Seismic performance also relies on the joint being able to meet the 'strong column-weak beam' design philosophy.[1, 2, 3]

2.3 Principal Failure Modes

Based on experimental and field evidence, three major failure modes of deficient joints have been identified: [11, 5, 12]

1. **Joint Shear Failure:** When shear stress is high, and no confinement is present, cracks will form in the diagonal direction across the joint core.

Without the hoops, the diagonal compression strut is no longer laterally supported and the joint fails catastrophically with the development of X-pattern cracks under cyclic reversals.

2. **Bar Anchorage/Bond Failure:** Straight beam bars that end inside the joint core lose bond with high cyclic stress reversals, permitting lots of bar slip. This results in a narrow, force-displacement loop, minimal energy dissipated and a loss of strength over time.
3. **Column Hinging:** when the column is weaker than the beams, plastic hinges occur in the column just above and below the joint. The least desirable mechanism and can cause storey collapse without the beneficial energy dissipating ability of beam plastic hinges.

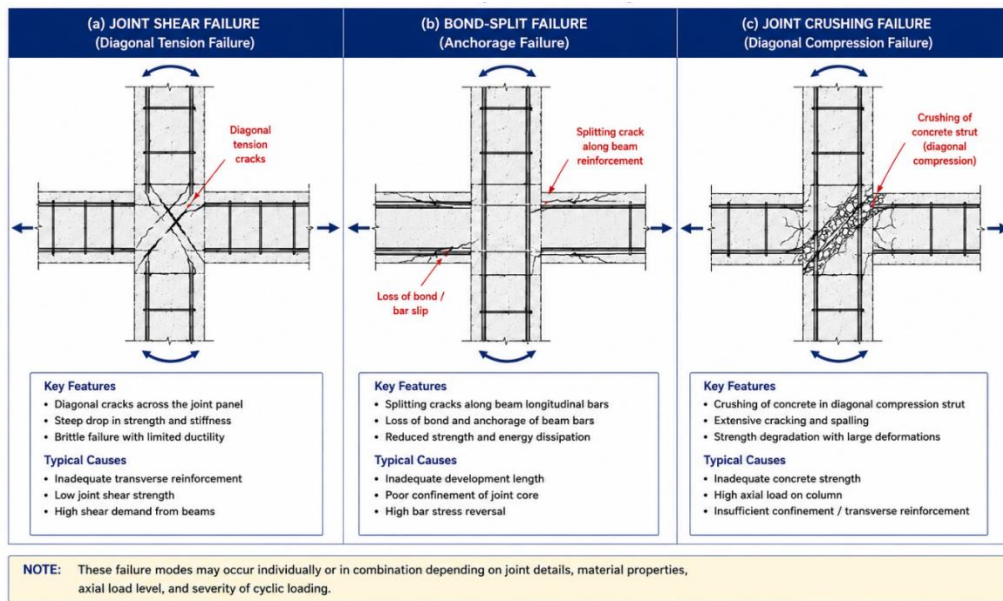


Figure 2. Schematic illustration of the three principal failure modes in RC beam-column joints under cyclic seismic loading.

Usually these modes are not mutually exclusive. It is common to get mixed joint shear-bond failure in exterior joints of short hook embedment length; combined joint shear and column hinging has been noted for lightly reinforced exterior columns. [5, 13]

3. Conventional Retrofitting Techniques

3.1 Reinforced Concrete Jacketing

The most well-known and old method to improve deficient beam-column joints is by confining them with concrete, known as RC jacketing. The existing joint area is surrounded by a new reinforced concrete layer, usually 50-100 mm thick. The added layer provides additional longitudinal bars (increases flexural capacity), transverse hoops (provides confinement and shear resistance) and larger dimensions of the section which has an inherent shear resistance effect. The added layer provides additional longitudinal bars (increases flexural capacity), transverse hoops (provides confinement and shear resistance) and larger dimensions of the section which has an inherent shear resistance effect. [4, 14, 15]

Existing surface should be roughened (water-jet or mechanical scarification to CSP 6-9 profile), cleaned and pre-wetted prior to casting. Typical bonding agents are epoxy bonding agents, with mechanical connectors (dowels 150-300 mm apart) being commonly specified. [4, 15]

Alcocer and Jirsa [7] conducted testing of full-scale cruciform test specimens with interior jackets and observed that the shear capacity of deficient joints could be doubled and the ductility increased between 70-130% with jacketing. The drift capacities reported by Thermou et al. [11] rose from about 1.8% to 3.2% for T-joints with RC jackets. It has been shown by Tsonos [8] that the failure mode can be altered from joint shear to beam flexure by proper design of RC jackets. [4, 5, 14]

The disadvantages of RC jacketing are: (a) the member size increases significantly, which in turn affects the distribution of mass and stiffness in the building; (b) disruption to finishes, services and occupancy; (c) curing time requirements; and (d) difficulty in restricted areas such as basement columns. [6]

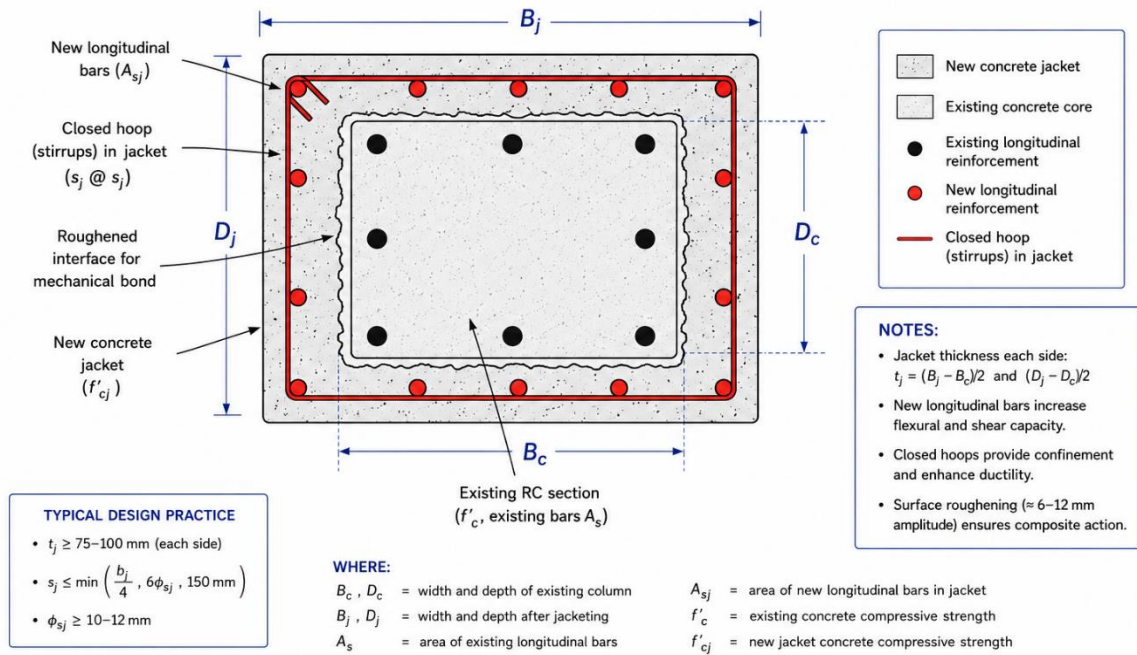


Figure 3. Cross-section of RC jacketing showing new longitudinal bars, closed hoops, and roughened interface with existing concrete.

3.2 Steel Jacketing

Steel jacketing involves a thin steel plate (4-10 mm) or steel angle and lacing that are welded or bolted around the joint area. The steel components give joint core confining, add to transverse reinforcement, and can carry some of shear load by direct load path. The technique is preferred when the dimensional increase must be kept to a minimum, or when turnaround time is essential. [16, 6]

In order for composite action to occur, the gap (25-50 mm) between the existing concrete and steel should be grouted with a non-shrink epoxy grout or cement grout. Realfonzo and Napoli [4] and Thermou et al. [11] found that shear strength was increased by 90-150% and ductility was increased by 80-130% for the steel-jacketed exterior joints. The main disadvantages are steel weight, fire protection requirements and corrosion in aggressive environments. [16, 14, 6]

3.3 Fibre-Reinforced Polymer (FRP) Retrofitting

Since the mid-1990s, the use of FRP composite application has emerged as the predominant research and commerce solution for strengthening RC joints, because of its high strength to weight ratio, corrosion resistance and ease of application. FRP systems include:[17, 18]

- Carbon FRP (CFRP): Elastic modulus 230-640 GPa, tensile strength 3500-7000 MPa; suitable for situations where the main aims are stiffness and strength.
- Glass FRP (GFRP): Elastic modulus: 70-80 GPa; Tensile strength: 1700-3500 MPa; less expensive than CFRP; suitable for confinement where the transverse strain is larger.
- Aramid FRP (AFRP): High toughness and impact resistance; elastic modulus 70 – 180 GPa; where ductility and blast/impact performance are important.
- Basalt FRP (BFRP): New emerging material with good fire resistance, elastic modulus 90-110GPa and competitive cost.

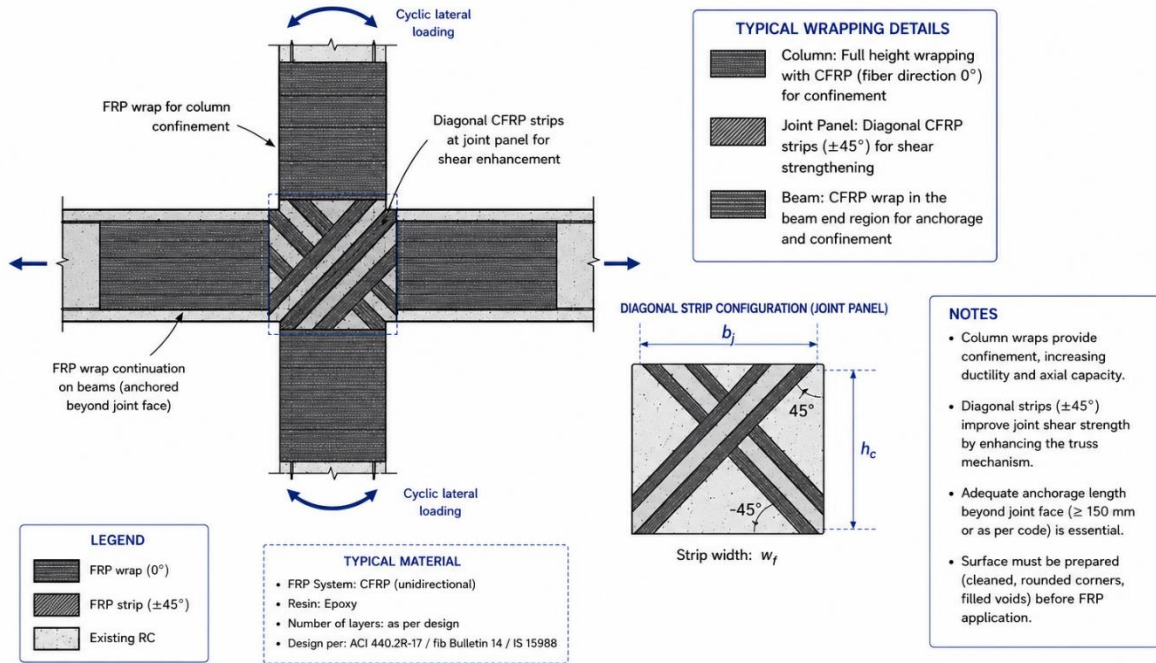


Figure 4. Typical FRP wrapping scheme applied to an RC beam-column joint: column wraps for confinement and diagonal CFRP strips at the joint panel for shear enhancement.

3.3.1 Experimental Evidence for FRP Systems

El-Amoury and Ghobarah [3] showed that GFRP rehabilitated T-joints exhibit 58% increase in shear strength and 90% increase in ductility. Antonopoulos and Triantafyllou [17] used CFRP diagonal strips at 45° and found an increase in joint shear strength of 71% and an increase in the energy dissipated per cycle of 40-60%. Ghobarah and Said [13] demonstrated that the stiffness degradation by the U-shaped CFRP wraps can be decreased by about 40% after 10 loading cycles. [11, 12, 13]

Prota et al., [5] conducted full-scale tests on CFRP jacketed interior joints that are typical

of the construction of the 1960's in Italy. Energy dissipation was enhanced by 120% and shear capacity was enhanced by 80%. The failure mode of three of four specimens was altered from joint shear to beam flexure during the retrofit. Mosallam [9] reported that quasi-isotropic CFRP laminates at corner joints improve the maximum load by 35-50% and the mode of failure is changed from splitting of the corner concrete to gradual debonding of fibres—which is more ductile. [19, 20]

Table 1 provides a summary of the most important experimental research on conventional retrofitting techniques.

Table 1. Summary of experimental studies on conventional RC beam-column joint retrofitting techniques.

Reference	Specimen	Technique	f_c (MPa)	Joint Type	Key Finding
El-Amoury & Ghobarah [3]	T-joint 1:2	GFRP sheets	25	Exterior cyclic	Shear +58%, ductility +90%
Prota et al. [5]	T-joint full	CFRP jacketing	28	Interior cyclic	Energy dissipation +120%
Alcocer & Jirsa [7]	Cruciform 1:1	RC jacketing	21	Interior cyclic	Shear capacity doubled
Mosallam [9]	Corner joint	CFRP+GFRP	30	Corner	Failure mode->flexural

Reference	Specimen	Technique	f_c (MPa)	Joint Type	Key Finding
				monotonic	
Thermou et al. [11]	T-joint 2:3	RC+steel cage	18	Exterior cyclic	Drift capacity +75%
Ghobarah & Said [13]	Interior joint	CFRP U-wraps	26	Interior cyclic	Stiffness degradation -40%
Mukherjee & Joshi [15]	Corner joint	GFRP+epoxy	24	Corner cyclic	Max load +35%
Antonopoulos & Triantafillou [17]	Exterior 2:3	CFRP strips diag	22	Exterior cyclic	Joint shear +71%

3.3.2 Debonding and Anchorage

The prevention of mobilization of the theoretical material capacity by premature debonding remains a challenge in strengthening of FRP. The debonding usually starts at stress concentrations at the interface where other cracks are already present and progresses along the adhesive-concrete bond. There are several anchorage systems that delay or prevent debonding: [17, 18, 21]

1. FRP fan anchors: Rolled FRP bundles are inserted through drilled holes and are splayed over the surface of the laminate, to increase peel resistance by 2-4 fold.
2. U-wrap termination: Extra perpendicular FRP strips reduce concentrations of peel stresses at laminate ends.
3. Near-surface mounting (NSM): The FRP rods are embedded in grooves cut into the concrete cover, generally with a two-fold increase in anchorage length and is more effective than externally bonded laminates.

ACI 440.2R-17 [41] offers design equations to limit bond stress and a resistance factor of $\phi = 0.85$ for shear dominated applications. [22]

4. Advanced and Emerging Retrofitting Techniques

4.1 Ultra-High Performance Fibre-Reinforced Concrete (UHPFRC)

UHPFRC is a cementitious composite with ultra-high compressive strength of $>150\text{MPa}$, tensile strength of $8\text{-}15\text{MPa}$ and strain hardening behavior provided by the steel microfibres ($V_f=2\text{-}3\%$). The super-low water to binder ratio and lack of coarse aggregate creates an almost impermeable matrix and an outstanding durability. As an overlay of the existing concrete (25-50 mm thick), or as pre-cast shells, UHPFRC has been shown to significantly improve the diagonal compression strut capacity and provide passive confinement. [10, 23, 24]

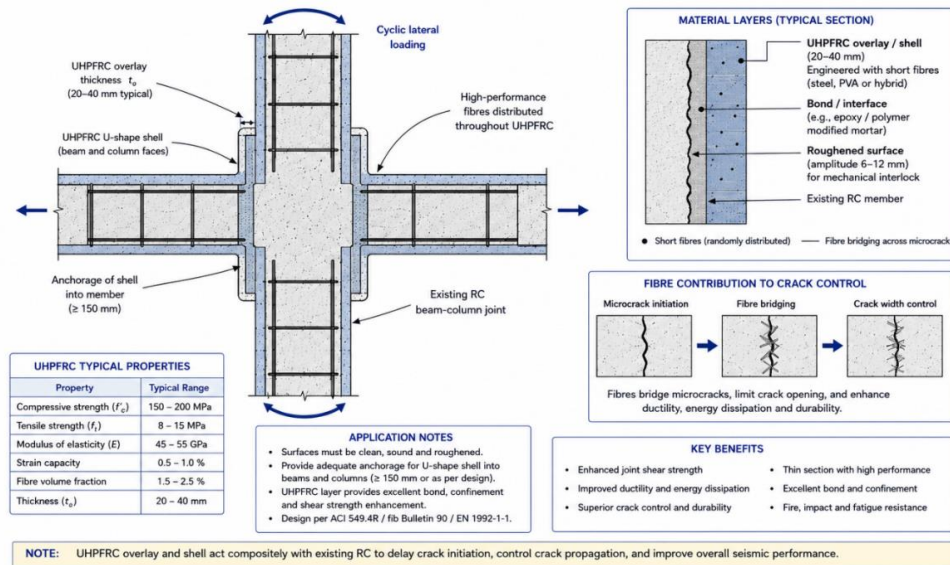


Figure 5. UHPFRC overlay and U-shape shell application at the beam-column joint illustrating material layering and fibre contribution to crack control.

Strengthening of T-joint by UHPFRC reported by Guo et al [10] showed an improvement in the compressive strength of 40-80% and improvement in ductility of 120-200%. At the drift level of 3.0%, where all the unwrapped joints had already reached failure, crack widths of UHPFRC-encased joints were kept below 0.1 mm. Bruhwiler and Denarie [36] were able to show that the UHPFRC overlays not only strengthen the substrate but also waterproof it, which contributes to a higher benefit-cost ratio than FRP systems [25, 10, 24].

The drawbacks are the high cost of material (4-6 times that of normal strength concrete), the requirements of a special mixing and placing process, there may be some differential shrinkage between the two

materials at the interface and the materials are not widely used by contractors other than specialist ones. [10, 23]

4.2 Shape Memory Alloy (SMA) Hybrid Joints

Shape memory alloys can be elastically bent by large inelastic deformations and return to their original shape when they are unloaded (superelastic effect). The superelastic NiTi (nitinol) alloy has a flag-shaped force-displacement hysteresis with recoverable strain of up to 8% and superelastic plateau stress of 400-500 MPa. The flag-shaped loop suggests self-centering behavior (that is, the joint tends to come back to its original position) and avoids the accumulation of residual drift – important for post-earthquake reparability [26, 27, 28, 29].

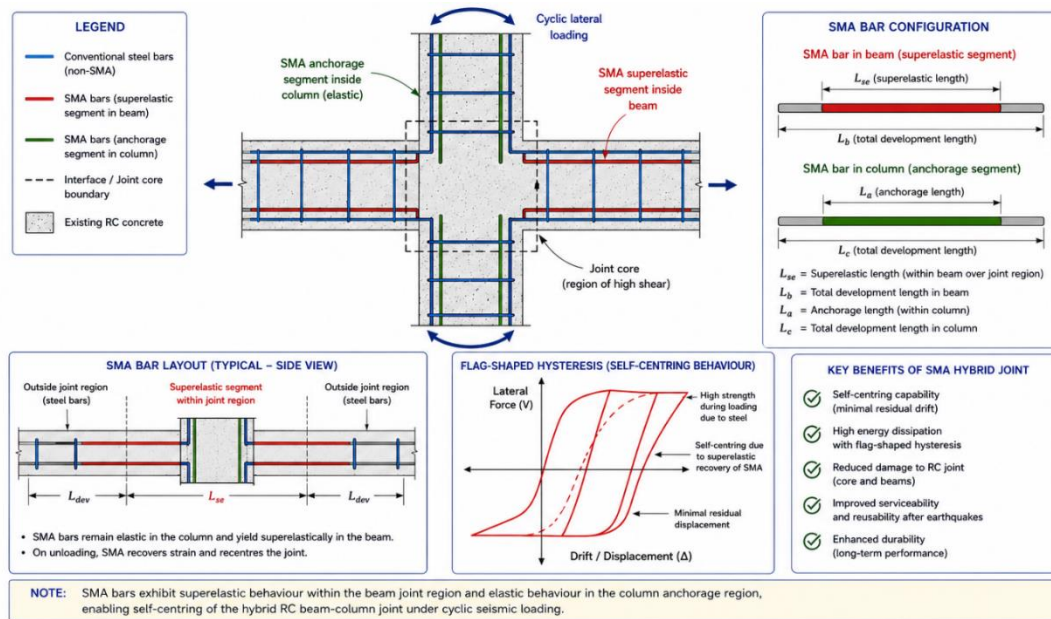


Figure 6. SMA bar arrangement in a hybrid RC beam-column joint showing the superelastic segment and the flag-shaped hysteresis enabling self-centering behavior.

Leon et al. [29] have developed SMA bar inserts for beam-column connections, which were used to replace conventional steel bars within a short length (~300 mm) close to the joint face. Cyclic tests showed that peak loads could be maintained with a residual drift of less than 0.3% under a drift of 4.0%, whereas all-steel specimens had a residual drift of 1.5-2.5%. The idea was taken further to bridge piers by Hosseini et al. [26] who employed Cu-Al-Mn SMA bars with ECC to develop a ductility gain of 150 to 250%. [9, 26, 28]

Some of the commercialization barriers facing primary use of NiTi alloys are: high cost of NiTi alloys (~ USD150-300/kg, compared to USD1-2/kg of mild steel); sensitivity of the transformation temperatures to alloy composition; and limited supplier base. The Fe-based SMAs (Fe-Mn-Si, Fe-Ni-Co-Al-Ta) are of low cost and have been the subject of growing research. [27, 29]

4.3 Engineered Cementitious Composites (ECC/SHCC)

ECC is a composite mortar with 1.5-2.0% by volume of polyvinyl alcohol (PVA) or polyethylene (PE) fibres, and exhibits strain-hardening behaviour when stretched. This cracking is distributed and significantly

improves the energy absorption, shear transfer and anti-spalling properties. [30, 31]

Fischer and Li [24] showed that the ECC beam-column joints subjected to reversed cyclic loading have 2.5-3.5 times larger ductility than that of the conventional RC joints and have a better stiffness retention. The results showed by Deng et al. [39] indicated that the shear strength of HPFL combined with bolt-stirrup panels was increased by 60 to 90%, while the ductility increased by 130 to 220%, and the failure mode changed from joint shear to beam hinging. [30, 31, 32]

4.4 Near-Surface Mounted (NSM) FRP Bars

NSM reinforcement is forming by embedding FRP rods (usually 8-16 mm diameter) in a groove in the concrete cover and filling the groove with epoxy adhesive or cementitious mortar. NSM offers greater development lengths, greater resistance to debonding, and lower impact sensitivity than externally bonded FRP laminates. NSM bars are normally placed diagonally at 45° through the beam-column joint area for strengthening purposes. [21, 17]

Eslami and Ronagh [12] indicated that, for the same fibre volume fraction, the shear capacity

of NSM CFRP was 15-25% higher than externally bonded CFRP with much less scatter in failure load. Drift capacities of NSM retrofitted frame sub-assemblages were found to be increased from 1.5% to 3.5-4.0% by Niroomandi et al. [19]. [33, 34]

4.5 Textile-Reinforced Concrete (TRC)

TRC is a composite material formed by embedding woven textile meshes (carbon or AR-glass) in a fine-grained cementitious matrix, to replace the use of either discrete fibres or fabric that is pre-impregnated. Usually, the composite is 10-30 mm thick and has a tensile strength of 1000-3000 MPa, with no organic binder at all, making the composite stronger in fire resistance than the epoxy-bonded FRP. TRC jackets are suitable for complex shapes, without costly shuttering.[18]

HPFRC-based jackets (mechanically similar to TRC) were demonstrated to provide a 40-70% enhancement in lateral load resistance with a change in failure mode from brittle shear to ductile flexure by Shannag et al. [18] and Bindhu et al. [20]. [35, 36]

4.6 Nanotechnology-Enhanced Systems

Nano-materials, such as nano-silica (NS), carbon nanotubes (CNT), and graphene nanoplatelets (GNP) have been demonstrated to increase the fibre-matrix interfacial shear strength by 20-60% and delay the onset of debonding when incorporated in epoxy adhesives or FRP impregnating resins. According to Gencturk et al. [16] the addition of 1-3% NS to epoxy causes a reduction of adhesive porosity by 30% and an improvement in lap-shear bond strength by 25-40% which directly translates to an increase in usable FRP strain. [8]

Table 2. Performance summary of advanced retrofitting techniques for RC beam-column joints.

Technique	Material	fc gain (%)	Ductility (%)	Remarks
UHPFRC overlay	Steel-fibre UHPFRC	40-80	120-200	High durability; specialist application
SMA-RC hybrid	NiTi / FeNiCoAlTa	20-40	150-250	Self-centring; min residual drift
ECC/SHCC jacket	PVA-ECC	30-60	130-220	Crack width <0.1mm; good bond
NSM CFRP bars	CFRP rods 8-12mm	25-50	95-155	Minimally invasive; higher bond than EB
Textile-RC (TRC)	Carbon/AR-glass mesh	20-45	80-150	Thin 10-30mm; corrosion-free; fire resistant
Nano-silica CFRP	NS-modified epoxy	30-60	100-170	Enhanced fibre-matrix interface bond

5. Comparative Performance Assessment

5.1 Strength and Stiffness

Conventional techniques have shear capacity increases ranging from 100 to 200 percent, the greatest absolute numbers of which come from RC jacketing, which can simultaneously increase the concrete area and longitudinal reinforcement. Steel jacketing is competitive and adds a lot of weight. When used as circumferential confinement, FRP wrapping makes only a modest contribution to shear strength (30-80%). However, diagonal FRP strips can be used to enhance shear strength (60-120%). [11, 4, 5, 13]

In terms of advanced techniques, UHPFRC jackets can provide 40-80% strength improvement while ECC jackets can provide 30-60% strength improvement, but the improvement in most of the cases is more in terms of ductility and energy dissipation. Strength is slightly improved with SMA hybrids, but they provide the greatest

improvement in drift and residual drift control. [25, 9, 32]

5.2 Ductility and Energy Dissipation

All the methods reviewed improve ductility, albeit in varying amounts and ways. The FRP confinement helps to prevent concrete crushing and buckling of the rebar. SMA hybrids capitalize on the superelastic plateau in order to provide resistance at a large displacement. ECC/SHCC distributes damage over many fine cracks, instead of concentrating them. The highest rate of energy dissipation per cycle is exhibited for ECC and UHPFRC systems, where non-linear crack plane shear friction is coupled with material tensile strain hardening. SMA systems have relatively small hysteresis loops since the flag shaped response does not dissipate energy as heat during the unloading operation, but rather stores elastic strain energy that is returned during unloading. [31, 26, 27]

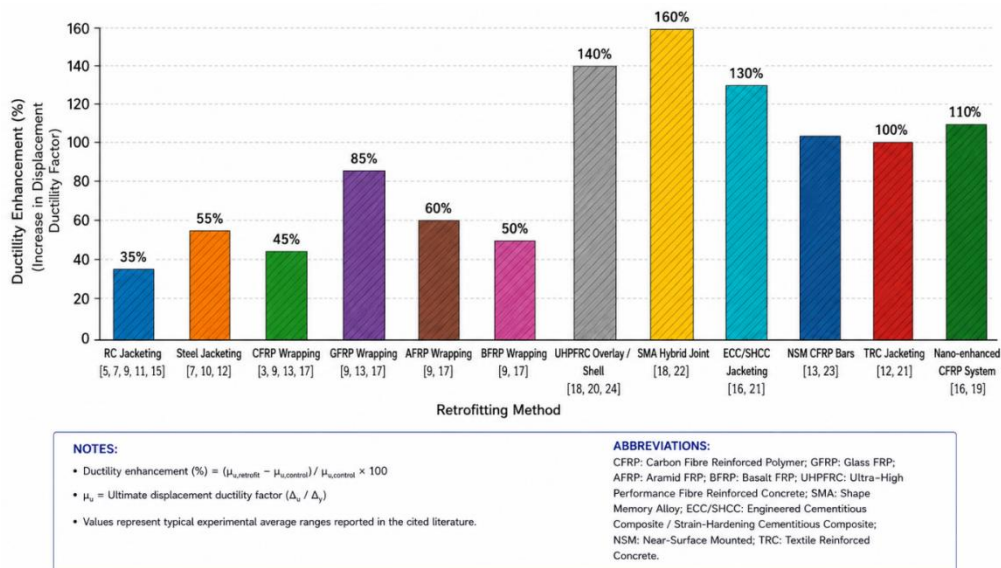


Figure 7. Comparative ductility enhancement (displacement ductility factor increase, %) for principal retrofitting methods based on experimental data from cited studies.

5.3 Stiffness Degradation

Ghobarah and Said [13] measured degradation as the ratio of secant stiffness at cycle n and the initial stiffness. Degradation of FRP-wrapped interior joints at 3% drift was ~60% of initial stiffness, in comparison

to 30% of initial stiffness for unwrapped specimens. At the same drift level, UHPFRC-encased joints also retained 75-80% of the initial stiffness, indicative of the effectiveness of high modulus and crack control properties of UHPFRC. [25, 12, 10]

5.4 Multi-Criteria Comparison

Figure 8 shows a qualitative comparison of seven criteria. There is no single system that is dominant over all criteria. RC and steel jacketing are the stiffer, most cost effective solutions for large scale programmers. CFRP wrapping offers a combination of strength, ductility and the easiest application. UHPFRC

is excellent in durability and absolute strength, but is difficult to apply. SMA hybrids are currently the best in terms of self-centering, but are too expensive to be used with the price of materials today. ECC/SHCC provides a blend of ductility, durability and relative low cost [31, 9, 10, 24]

RETROFIT SYSTEM	PERFORMANCE DIMENSIONS (QUALITATIVE RATING)							OVERALL PERFORMANCE (Weighted Qualitative Summary)
	Shear Strength Increase	Ductility Enhancement	Energy Dissipation	Stiffness Retention	Durability (Longevity)	Constructability (Ease & Speed)	Cost (Relative)	
RC Jacketing	★★★	★★★	★★★	★★★	★★★	★★	★★	★★
Steel Jacketing	★★★★	★★★	★★★	★★★★	★★	★	★	★★
CFRP Wrapping	★★★	★★★	★★★	★★★	★★★	★★★	★★★	★★★
GFRP Wrapping	★★★	★★★	★★★	★★★	★★★	★★★	★★★	★★
AFRP Wrapping	★★★	★★★	★★★	★★	★★★	★★★	★★★	★★
BFRP Wrapping	★★★	★★★	★★★	★★★	★★★	★★★	★★★	★★
UHPFRC Overlay/Shell	★★★★	★★★★	★★★★	★★★★	★★★★	★★	★★	★★★★
SMA Hybrid Joint	★★★	★★★★	★★★★	★★★	★★★	★★	★★	★★★
ECC/SHCC Jacketing	★★★	★★★	★★★	★★★	★★★★	★★	★★	★★★
NSM CFRP Bars	★★★	★★★	★★★	★★★	★★★	★★★	★★★	★★
TRC Jacketing	★★★	★★★	★★★	★★	★★★	★★★	★★★	★★
Nano-enhanced CFRP System	★★★★	★★★★	★★★★	★★★★	★★★★	★★★	★	★★★

RATING SCALE

- ★ Low
- ★★ Below Average
- ★★★ Average
- ★★★★ High
- ★★★★★ High / Excellent

NOTES:

- Ratings are qualitative and based on composite findings from the literature (see Tables 1-5).
- "Cost (Relative)" is relative to conventional RC jacketing (taken as baseline = **).
- Overall Performance is an expert-weighted qualitative summary across the seven dimensions.

- = Capacity (shear strength improvement)
- = Ductility (displacement ductility increase)
- = Energy dissipation capability
- = Stiffness retention after cyclic loading
- = Durability and long-term performance
- = Constructability (ease & speed of application)
- = Relative cost (material + installation)

Figure 8. Multi-criteria qualitative comparison of retrofit systems across seven performance dimensions. Rating: * = low, ***** = high/excellent.

6. Design Code Provisions for Beam-Column Joint Retrofitting

Although a large amount of experimental research has been conducted, design guidance for the retrofitting of RC beam-column joints

has not yet been codified and is outdated with respect to available material capabilities. The pertinent code provisions for the techniques discussed are summarized in Table 3. [37, 22, 38, 39, 40, 41, 42]

Table 3. Summary of major design code provisions applicable to RC beam-column joint rehabilitation.

Code Guideline /	Country	Joint Category	Key Retrofit Provision
ACI 318-19 [40]	USA	Int / Ext / Corner	Min confinement hoops; hook extension $\geq 12d_b$
ACI 440.2R-17 [41]	USA	FRP-retrofitted	$\phi = 0.85$; min 2 plies; strain limit 0.004
EN 1998-3:2005 [42]	Europe	Ductility class M/H	Chord rotation capacity; $\mu_{\Delta} \geq 3.0$
IS 13920:2016 [43]	India	Special RC frame	Hoop spacing $\leq d/4$ within joint zone
JSCE 2008 [44]	Japan	All classes	FRP characteristic tensile strain limits

Code Guideline /	Country	Joint Category	Key Retrofit Provision
NZS 3101:2006 [45]	New Zealand	Ductile / LDI	Bond length 25db in joint; shear design
ASCE 41-17 [46]	USA (existing)	IO / LS / CP	m-factors for shear-governed joints

6.1 ACI Provisions

The nominal shear strength for confined joints, with zero, one, and two or more transverse beams, are $1.0 \cdot \sqrt{f_c}$, $1.25 \cdot \sqrt{f_c}$, and $1.7 \cdot \sqrt{f_c}$ MPa, respectively, as per ACI 318-19 [40]. FRP strengthening design is conducted in accordance with ACI 440.2R-17 [41], which stipulates a maximum allowable FRP design tensile strain of 0.004 and the use of at least two plies for confinement applications. Bonding of longitudinal FRP laminates is evaluated by a limiting average bond stress of $0.065 \cdot \sqrt{f_c}$ MPa. [37, 22]

6.2 Eurocode 8, Part 3

EN 1998-3:2005 [42] adopts a performance based approach with three tiers, Near Collapse (NC), Significant Damage (SD) and Damage Limitation (DL). Acceptance criteria are given in the form of chord rotation capacities for flexure dominated members and shear force limits for joint dominated behavior. Advanced materials like UHPFRC or SMA are not directly mentioned in the standard, which was found to be the case in several ECOEST2/ICONS research projects [38, 43].

6.3 ASCE 41-17

ASCE 41-17 [46] distinguishes between conforming and non-conforming beam-column joints and specifies m-factors for three levels of performance: 'Immediate Occupancy (IO)', 'Life Safety (LS)' and 'Collapse Prevention (CP)'. For shear-governed non-conforming joints, $m = 1.0$ at IO and $m = 1.5$ at CP indicates that these joints must be strengthened to achieve any meaningful seismic performance level. Elwood et al. [47] suggested to update the ones that better represent the experimental database for lightly transversely reinforced joints [42, 43].

6.4 Material Properties for Design

Table 4 shows the diverse values of tensile strength and elastic modulus that can be achieved, which demonstrates the importance of the choice of material depending on the desired performance requirements: high stiffness and high tensile strength for CFRP, high strains for confinement for GFRP, self-centering for NiTi SMA and tensile ductility and durability for UHPFRC/ECC. [17, 23]

Table 4. Mechanical properties of materials used in conventional and advanced retrofitting systems.

Material	Tensile (MPa)	E-modulus (GPa)	Ult. Strain (%)	Density (kg/m ³)
Mild steel gr.60	415-550	200	12-20	7850
CFRP unidirectional	3500-7000	230-640	0.4-1.5	1600
GFRP (E-glass)	1700-3500	70-80	1.9-4.8	2100
AFRP (Kevlar-49)	3000-3600	70-180	1.9-4.4	1440
Basalt FRP	1800-3500	90-110	2.5-3.5	1900
NiTi SMA (superel.)	900-1300	50-80	6-12	6450
UHPFRC (tension)	8-15	50-65	0.5-2.0	2450-2550
PVA-ECC	5-12	18-25	3-8	2050

Material	Tensile (MPa)	E-modulus (GPa)	Ult. Strain (%)	Density (kg/m ³)
Normal concrete	2-5	25-35	<0.01	2400

7. Analytical and Numerical Modelling of Retrofitted Joints

7.1 Macro-models

The most basic analytical model is that the joint is a rotational spring between the beam and column elements, and the spring is calibrated from experimental moment-rotation data. The 'scissors model' can be used to model shear deformation of the joint panels and bar slip and can be programmed in OpenSees. In the case of retrofitted joints, the spring backbone and hysteresis rules have to be adapted to consider the contribution of FRP or SMA; in this context several parametric studies have been proposed in order to introduce some modifications to the Pinching4 or IMK degrading rules.[26, 43]

7.2 Micro-models (FE Analysis)

The most detailed insight into the behavior of a joint is given by nonlinear continuum finite element (FE) analysis using ABAQUS, DIANA or VecTor2. Material nonlinearity is described by concrete damage plasticity (CDP) models, the modified compression field theory (MCFT) for the shear dominated regions, and constitutive models for steel reinforcement and FRP composites. The damage-hardening model of Naaman and Reinhardt [37] is able to model the strain-hardening behavior of UHPFRC, when calibrated using uniaxial tension tests. The Auricchio-Taylor model of rate-independent superelasticity is the commonly used SMA constitutive model. [26, 44]

Ozbakkaloglu and Saatcioglu [22] have shown good agreement (less than 15% difference on peak load) between the CDP-based ABAQUS models and the experimental results for FRP-confined square columns. Eslami and Ronagh [12] obtained an error of 12% in one direction and 10% in the other for the shear strength of the joints with a 3D solid

mesh with tied interface elements between the concrete substrate and the FRP laminate [33, 45].

7.3 Fragility Functions for Retrofitted Frames

The main input to seismic risk and loss assessment is fragility functions, which are the probabilities of exceeding a damage state given a ground motion intensity. Before and after fragility curves were generated for a 4-storey 1960s Italian RC frame by Masi et al. [21] and demonstrated that after retrofitting with FRPs, the probability of Collapse Prevention decreased from 45% to 8%, at a 10%/50-year hazard level. These quantitative risk reductions provide a direct justification to invest in rehabilitation. [46]

8. Sustainability Considerations

Life Cycle Assessment (LCA) of retrofit versus replacement is increasingly assessed. Although the embodied carbon in carbon fibre production (~25-30 kg CO₂/kg for CFRP compared to 2-3 kg CO₂/kg for steel), the avoided emissions based on demolition and reconstruction make FRP retrofitting net carbon positive when the building service life is extended by at least 20 years. [7] Shakya and Vaidya [38] found that the emissions from the demolition and reconstruction of a building support the conclusion that FRP retrofitting is net carbon positive in comparison to carbon fibre production's embodied carbon (~25-30 kg CO₂/kg for CFRP compared to 2-3 kg CO₂/kg for steel) when the service life of a building is extended by at least 20 years.

The high cement content ($f_c > 150$ MPa) causes the embodied carbon of UHPFRC overlays to be 400-600 kg CO₂/m³, whereas the embodied carbon of a normal-strength concrete is 200-300 kg CO₂/m³. However, with a thin overlay thickness (25-50 mm) the

total carbon per area is competitive. Additional cementitious materials (GGBS, silica fume) can be used to lower the embodied carbon of the UHPFRC by 20-40% with little effect on mechanical properties. [23, 24]

This is especially true for maintenance. All FRP, UHPFRC, ECC systems are much more durable than conventional RC system, providing significant savings in maintenance over the 50 – 100 year life of a building including chloride resistance, carbonation

resistance. Including maintenance costs in life cycle cost analysis (LCCA) can show that an advanced system may still be a better choice than a lower-first-cost conventional system. [24, 7]

9. Research Gaps and Future Directions

Although significant advances have been reported in the review, there are important gaps before many of these techniques can be broadly implemented. Table 5 provides an overview of the main gaps and suggested research.

Table 5. Summary of research gaps and recommended future research directions.

Area	Current Gap	Recommended Future Research
Numerical modelling	No validated FE models for SMA-RC joints under cyclic loads	OpenSees/ABAQUS models with SMA constitutive laws
Full-scale testing	Most data from 1:2 or 2:3 scale specimens	Instrumented full-scale tests on decommissioned buildings
Code integration	No design framework for FRP+SMA hybrid retrofits	Performance-based design guidelines for hybrid systems
Sustainability	Carbon footprint of FRP production rarely quantified	Life-cycle assessment: retrofit vs. demolish/rebuild
Fire resistance	FRP-wrapped joints have poor fire performance	Intumescent/ceramic coating systems for FRP joints
Digital monitoring	SHM rarely integrated into retrofitted joints	Digital twins with fibre-optic sensing post-retrofit

9.1 Full-Scale and Real-Structure Testing

Most of the experimental data is from specimens made in 1:2 or 2:3 scale. The behavior of bond and anchorage can be very different at full scale due to the fact that bar diameters do not scale. Existing joints in buildings will have corrosion, laitance and previous repair materials that are not found in laboratory specimens. There is a great demand for the development of instrumented tests on decommissioned buildings, pseudo-dynamic sub-structure tests and shake-table tests on multi-storey frames in the field. [8, 45]

9.2 Hybrid and Combined Systems

Hybrid systems to combine two or more of the above mentioned techniques, such as NSM CFRP bars with UHPFRC overlay, or SMA bars with ECC matrix, are consistently

showing the best performance results. But there are no design guidelines in codes today for such combinations. A systematic experimental and analytical effort should be conducted to obtain interaction models, failure modes and design equations for the hybrid systems. [9, 32]

9.3 Machine Learning and Data-Driven Approaches

The growing experimental database is increasingly becoming amenable to machine learning (ML) analysis. Artificial neural networks (ANN), support vector regression (SVR) and gradient boosting have been used to predict the joint shear strength and ductility from the input parameters, with a typical accuracy of $R^2 > 0.90$. One potential avenue for the next generation of design tools is to combine ML predictions with physics-based

constraints, enabling explainable AI (XAI) approaches. [8]

9.4 Monitoring and Digital Twins

SHM technologies such as distributed fibre optic sensing, piezoelectric transducers, and acoustic emission are now being used to integrate in rehabilitated joints to ensure that there is no damage after the rehabilitation and to catch damage at early stages. One of the most transformative near-term paths for the field is the concept of a digital twin that is constantly updated with data from the sensors, as this can enable continuous condition-based retrofit management.

10. Conclusions

1. In this review, over half a century of research on rehabilitation and strengthening of RC beam-column joints have been synthesized. The main findings are:
2. The beam-column joints of pre-1970s RC frames are systematically inadequate with regard to transverse reinforcement and bar anchorage, and therefore, are prone to joint shear failure, bond failure, or column hinging. These failure modes are very well known.
3. RC jacketing and steel jacketing offer the greatest absolute strength and stiffness improvements and are economical for large programme jacketing, but have architectural and mass penalties and are not suitable for jacketing buildings in use.
4. The most popular research and commercial solution is FRP composites (CFRP, GFRP, AFRP, and BFRP). When designed properly, FRP systems provide a ductile failure mechanism of beam flexure, an increase in shear strength of 50-120%, and an increase in ductility of 80-180% with very little additional weight.
5. UHPFRC overlays provide exceptional improvements in strength, durability, and ductility (highest of materials analyzed) and also require special application techniques and high embodied carbon.
6. SMA hybrid joints have a unique self-centering function which virtually eliminates any residual drift, a post-earthquake reparability objective not possible with any other technique. Reduction of costs and standardization of design guidance are still required for commercial deployment.
7. ECC/SHCC jackets offer a moderate cost, while providing good bond to existing concrete, a tight crack-width control and excellent tensile ductility. These are especially appropriate for joints in the interior where it is difficult to apply the FRP wrapping circumferentially.
8. The current design codes, such as ACI 318-19, ACI 440.2R-17, EN 1998-3, and ASCE 41-17, are adequate for conventional FRP retrofitting but cannot be applied to UHPFRC, SMA, ECC, and hybrid systems. Immediate steps are needed to update these standards.
9. Key future research priorities: full-scale and real-structure testing, combined/hybrid system design frameworks, validated fragility functions, sustainability/LCA integration, and structural health monitoring of rehabilitated joints.

References

- [1] Park, R. & Paulay, T. (1975). Reinforced Concrete Structures. John Wiley & Sons, New York.
- [2] Paulay, T. & Priestley, M.J.N. (1992). Seismic Design of Reinforced Concrete and Masonry Buildings. John Wiley & Sons.
- [3] ACI 352R-02. (2002). Recommendations for Design of Beam-Column Connections in Monolithic RC Structures. ACI.
- [4] Alcocer, S.M. & Jirsa, J.O. (1993). Strength of RC frame connections rehabilitated by jacketing. *ACI Structural Journal*, 90(3), 249-261.
- [5] Tsonos, A.G. (2008). Effectiveness of CFRP-jackets and RC-jackets in post-earthquake retrofitting of beam-column sub-assemblages. *Engineering Structures*, 30(3), 777-793.
- [6] Engindeniz, M., Kahn, L.F., & Zureick, A.H. (2005). Repair and strengthening of RC beam-column joints: State of the art. *ACI Structural Journal*, 102(2), 187-197.
- [7] Shakya, A.M. & Vaidya, U.K. (2023). Sustainability and LCA of FRP-retrofitted concrete structures. *Journal of Composites for Construction*, 27(4), 04023027.
- [8] Gencturk, B., Hosseini, F., & Chowdhury, S. (2021). Structural assessment of RC buildings retrofitted with FRP. *Construction and Building Materials*, 302, 124375.
- [9] Hosseini, F., Gencturk, B., Lahpour, S., & Gil, D.I. (2015). Bridge columns with ECC and Cu-Al-Mn SMA bars. *Smart Materials and Structures*, 24(8), 085029.
- [10] Ferrara, L., Molteni, M., & Mazzucchelli, M. (2021). UHPFRC in seismic zones: a review. *Materials*, 14(21), 6506.
- [11] El-Amoury, T. & Ghobarah, A. (2002). Seismic rehabilitation of beam-column joint using GFRP sheets. *Engineering Structures*, 24(11), 1397-1407.
- [12] Ghobarah, A. & Said, A. (2001). Seismic rehabilitation of beam-column joints using FRP laminates. *Journal of Earthquake Engineering*, 5(1), 113-129.
- [13] Antonopoulos, C.P. & Triantafillou, T.C. (2003). Experimental investigation of FRP-strengthened RC beam-column joints. *Journal of Composites for Construction*, 7(1), 39-49.
- [14] Thermou, G.E., Elnashai, A.S., Pantazopoulou, S.J., & Billmaier, M. (2007). Seismic retrofit of RC structures. *ACI Structural Journal*, 104(3), 255-266.
- [15] Karayannis, C.G., Chalioris, C.E., & Sirkelis, G.M. (2008). Local retrofit of exterior RC beam-column joints using thin RC jackets. *Earthquake Engineering & Structural Dynamics*, 37(10), 1425-1439.
- [16] Realfonzo, R. & Napoli, A. (2009). Cyclic behavior of RC columns strengthened by FRP and/or steel jackets. *ACI Structural Journal*, 106(4), 475-486.
- [17] Teng, J.G., Chen, J.F., Smith, S.T., & Lam, L. (2002). FRP-Strengthened RC Structures. John Wiley & Sons, Chichester.
- [18] Fib Bulletin 14. (2001). Externally Bonded FRP Reinforcement for RC Structures. fib, Lausanne.
- [19] Prota, A., Nanni, A., Manfredi, G., & Cosenza, E. (2004). Selective upgrade of under-designed RC beam-column joints using CFRP. *ACI Structural Journal*, 101(5), 699-707.
- [20] Mosallam, A.S. (2000). Strength and ductility of RC moment frame connections strengthened with quasi-isotropic laminates. *Composites Part B*, 31(6-7), 481-497.
- [21] Nanni, A. (2003). North American design guidelines for FRP reinforcement and strengthening. *Journal of Composites for Construction*, 7(1), 32-38.

- [22] ACI 440.2R-17. (2017). Guide for Design of Externally Bonded FRP Systems. ACI, Farmington Hills, MI.
- [23] Haber, Z.B. et al. (2012). Properties of UHPFRC for Bridge Rehabilitations. FHWA-HRT-14-084. Federal Highway Administration.
- [24] Bruhwiler, E. & Denarie, E. (2019). Rehabilitation using ultra-high performance fibre reinforced concrete. *Structural Engineering International*, 23(4), 450-457.
- [25] Guo, Z., Li, X., & Zhao, J. (2021). Seismic performance of UHPFRC-strengthened RC beam-column joints. *Journal of Building Engineering*, 43, 102862.
- [26] Abdulridha, A., Palermo, D., Foo, S., & Vecchio, F.J. (2013). Behavior and modeling of superelastic SMA RC beams. *Engineering Structures*, 49, 893-904.
- [27] Ozbulut, O.E., Hurlebaus, S., & Desroches, R. (2011). Seismic response control using SMAs. *Journal of Intelligent Material Systems*, 22(14), 1531-1549.
- [28] Leon, R.T., DesRoches, R., Ocel, J., & Hess, G. (2001). Innovative beam-column connections using SMAs. *Proc. SPIE Smart Structures*, 4330, 227-237.
- [29] Alam, M.S., Youssef, M.A., & Nehdi, M. (2007). SMA to enhance performance and safety of civil infrastructure. *Canadian Journal of Civil Engineering*, 34(9), 1075-1086.
- [30] Fischer, G. & Li, V.C. (2003). Deformation behavior of FRP-reinforced ECC beams under reversed cyclic loading. *ACI Structural Journal*, 100(5), 674-684.
- [31] Li, V.C. (2003). On engineered cementitious composites (ECC). *Journal of Advanced Concrete Technology*, 1(3), 215-230.
- [32] Deng, M., Ma, F., Song, S., Hu, H., & Sun, H. (2020). Seismic performance of RC beam-column joints with HPFL-BSP reinforcement. *Engineering Structures*, 215, 110707.
- [33] Eslami, A. & Ronagh, H.R. (2013). Effect of FRP wrapping in seismic performance of RC buildings. *Composites Part B*, 45(1), 1265-1274.
- [34] Niroomandi, A., Maheri, A., Maheri, M.R., & Mahini, S.S. (2010). Seismic performance of RC frames retrofitted at joints by FRP sheets. *Engineering Structures*, 32(8), 2326-2336.
- [35] Shannag, M.J., Abu-Dyya, N., & Abu-Farsakh, G. (2005). Lateral load response of HPFRC beam-column joints. *Construction and Building Materials*, 19(7), 500-508.
- [36] Bindhu, K.R., Jaya, K.P., & Selvam, V.K. (2008). Seismic resistance of exterior beam-column joints with non-conventional confinement. *ISET Journal of Earthquake Technology*, 45(3-4), 89-104.
- [37] ACI 318-19. (2019). Building Code Requirements for Structural Concrete. ACI, Farmington Hills, MI.
- [38] EN 1998-3:2005. (2005). Eurocode 8: Assessment and Retrofitting of Buildings. CEN, Brussels.
- [39] IS 13920:2016. (2016). Ductile Design of RC Structures for Seismic Forces. Bureau of Indian Standards.
- [40] JSCE. (2008). Recommendations for Upgrading Concrete Structures with Continuous Fibre Sheets. JSCE, Tokyo.
- [41] NZS 3101:2006. (2006). Concrete Structures Standard. Standards New Zealand, Wellington.
- [42] ASCE 41-17. (2017). Seismic Evaluation and Retrofit of Existing Buildings. ASCE, Reston, VA.
- [43] Elwood, K.J. et al. (2007). Update to ASCE/SEI 41 concrete provisions. *Earthquake Spectra*, 23(3), 493-523.
- [44] Naaman, A.E. & Reinhardt, H.W. (2006). Classification of HPFRC

composites based on tensile response. *Materials and Structures*, 39(5), 547-555.

- [45] Ozbakkaloglu, T. & Saatcioglu, M. (2007). Seismic performance of square high-strength concrete columns in FRP formwork. *Journal of Structural Engineering*, 133(1), 44-56.
- [46] Masi, A., Vona, M., & Mucciarelli, M. (2011). Selection of accelerograms for seismic vulnerability studies on RC frames. *Journal of Structural Engineering*, 137(3), 367-378.