Građevinski materijali i konstrukcije
Building Materials and Structures
journal homepage: www.buildingmaterialsstructures.com

https://doi.org/10.5937/GRMK2600016V

Review paper

Demountable connections for structural concrete reusability - State of the art and future directions for reinforced concrete, Part II: precast frames

Vidović Milica*[1)], Milićević Ivan[1)], Carević Jelena[1)]

[1)] *University of Belgrade, Faculty of Civil Engineering, Bulevar kralja Aleksandra 73, 11000 Belgrade, Serbia*

Article history

Received: 20 May 2026
Received in revised form: 28 May 2026
Accepted: 29 May 2026
Available online: 12 June 2026

Keywords

precast structures,
dry and semi-dry joints,
demountable connections,
reusability,
frame structures,
sustainable structures

ABSTRACT

This paper presents a comprehensive review of demountable connections in precast concrete frame structures, as a key strategy for the potential reuse of structural elements. The focus is placed on dry and semi-dry joints, which allow relatively simple assembly and disassembly. The paper systematically examines connection solutions for beam-to-column joints, column-to-foundation joints, and column-to-column joints, describing their mechanical devices, force-transfer mechanisms, and typical failure modes. The proposed solutions differ in complexity, construction time, and the amount of steel components used. The review shows that, in certain cases, fully demountable precast joints can achieve adequate resistance and performance comparable to monolithic joints. The most connection solutions demonstrated limited reuse possibilities of precast elements. Only a few studies have experimentally investigated the behaviour of joints in the second life cycle of fully reused precast elements, comparable to that of original joints. Although significant effort was made to identify optimal solutions, their widespread implementation in construction practice will require comprehensive experimental and numerical research focusing on repeated assembly–disassembly cycles and system-level behaviour, as well as their integration into design standards and practical guidelines.

1 Introduction

Reinforced concrete (RC) frames play a major role in RC building structures, forming part of both the gravity-load-resisting and lateral-load-resisting systems. They comprise beams (with or without slabs), columns and beam-to-column joints. The RC frames are most commonly designed and constructed as cast-in-situ (monolithic) moment-resisting frames, capable of resisting the combinations of bending moments, shear forces and axial forces. In this regard, the reliable behaviour of columns and beams at the joint locations, as well as the behaviour of the beam-to-column joints themselves, is of paramount importance for the adequate performance of RC frames under gravity and lateral loads, such as wind and design earthquake loads. For the earthquake events, RC frames acting as a main seismic-load-resisting system are designed as "Strong-Column/Weak-Beam" systems, i.e., with the RC columns that have bending resistance (with axial forces) significantly higher than adjacent RC beams (20-30%, depending on the seismic design code). In this manner, the reliable beam-sway mechanism is provided by forming plastic hinges at the base of the columns and at ends of beams. The beam-to-column joints are often subjected to high stresses under seismic action, and are primarily designed to remain elastic, with sufficient strength and stiffness to ensure the required degree of restraint in the RC frames.

The construction industry is increasingly focused on reducing its environmental impact through circular economy principles – reduce, recycle and reuse, including design for deconstruction of reinforced concrete structures [1-5]. At the end of service life, or when irreparable damage occurs to the monolithic RC frames after the main earthquake event, most columns and beams in RC frame buildings are demolished and the usual sustainable solution is to recycle them. Although reuse of existing monolithic RC elements has the potential to reduce waste generation, greenhouse gas emissions, and consumption of raw materials, its practical implementation remains limited due to technical, regulatory, and market-related barriers [6], [7].

The reinforced concrete frames can also be designed and constructed as precast concrete elements to primarily reduce the construction time. In terms of sustainability, apart from recycling, precast RC frames also have great potential for disassembly, replacement, and even the reuse of columns and beams at the end of their service life.

Based on the broader context of circular construction and reusable precast concrete systems discussed in Part I of this study [8], this paper focuses on demountable connections in precast concrete frame structures. The design of slab-to-slab

* Corresponding author:
E-mail address: milica@imk.grf.bg.ac.rs

and slab-to-beam connections is primarily governed by serviceability demands, gravity load transfer and rigid diaphragm requirements [8]. On the other hand, beam-to-column, column-to-column, and column-to-foundation joints represent the critical load-bearing regions of precast frame systems. These connections must ensure reliable transfer of axial forces, bending moments, and shear forces while simultaneously providing adequate stiffness, strength, and ductility. In seismic regions, their role becomes even more significant, as the connection detailing directly influences energy dissipation, plastic hinge formation, residual deformations, and the overall structural response. Therefore, recent research is focused on the development of demountable dry and semi-dry connections that enable easier assembly, disassembly, and reuse of precast elements while maintaining adequate structural performance [9].

The aim of this study is to critically review demountable dry or semi-dry beam-to-column, column-to-foundation, and column-to-column connections from the available literature. The discussion includes not only the load-bearing capacity, stiffness, deformation capacity and energy dissipation of connections and joints but also their assembly and disassembly characteristics, as well as their potential for replacement and reuse beams, columns, foundations and connections.

2 Beam-to-column connections

According to the Fédération Internationale du Béton [10], precast RC frame systems can generally be divided into two major categories, depending on the type and design of the connections between beams and columns: "equivalent monolithic" systems and "jointed" systems. In the former, wet joints are commonly used and are designed and constructed to be equivalent to those in monolithic structures (cast-in-place emulation). Regarding disassembly, replacement and reuse, this type of precast RC frame system has similar drawbacks to actual cast-in-place RC frame structures. In "jointed" systems, dry or semi-dry connections are used. In general, the connections can be formed in two ways: (1) by welding or bolting reinforcing bars, using plates or steel embedments, with dry-packing or local grouting, or (2) utilising unbonded post-tensioned tendons or rods. However, jointed systems are generally considered not to fully emulate the performance of cast-in-place RC frames, although the ductility of the connections can be achieved to some extent.

According to Eurocode 8 [11], the connections in precast concrete structures under earthquake action can be divided into three major categories, regarding their resistance and ductility:

- Connections located well outside critical regions, which do not affect the energy dissipation capacity of the structure;
- Overdesigned connections located within critical regions, but adequately designed for strength based on capacity design rules with respect to the rest of the structure, so that they remain elastic while plastic hinging is shifted outside the connection;
- Connections located within critical regions with substantial ductility.

The choice of the connection category can significantly impact primarily the overall behaviour of the joint under seismic action, and its potential for disassembly, replacement and reuse. In general, the similar categorisation of connections can be applied for gravity loads and wind loads as well, depending on their strength and the location along the beam's length.

Over the past decades, many different configurations of dry and semi-dry beam-to-column connections with anchor bolts, rods and couplers, were experimentally and numerically studied, either as pinned connections with dowels [12], [13], or as moment resisting (semi-rigid or rigid/fixed) connections [14-16]. The main objective was to gain insight into the cyclic performance of the beam-to-column joints and, in case of moment resisting connections, to achieve the emulation of monolithic beam-to-column joints. However, in general, a little to none consideration on the disassembly of the precast elements was paid.

In the following subsections, the review of the latest research on dry and semi-dry precast beam-to-column joints is presented, which mainly employ mechanical fasteners and connectors, with or without steel plates. However, the joints which include welding of reinforcement with partial grouting, different kinds of dissipative or replaceable devices, as well as innovative post-tensioned connections are also covered. The focus is placed on moment resisting joints.

2.1 Connections with RC corbels

Precast RC columns can be constructed with RC corbels, which are used for supporting precast RC beams (resist shear forces) and to simplify the assembly process during the construction. Traditionally, the beams are simply supported via corbels, i.e., hinged/pinned connections with dowels are usually employed. However, this type of connection can also be designed as moment resistant connection, if appropriate mechanical connectors for continuity of longitudinal reinforcement or anchor bolts are used.

Bournas et al. [17] and Negro et al. [18] have investigated the behaviour of a full-scale three-storey precast RC structure consisting of dapped-end beams and columns with RC corbels, by subjecting the structure to a series of pseudo-dynamic (PsD) experimental tests. As stated by the authors, the dry connections were used because of quick erection, easy maintenance and possible reuse. Two types of dry beam-to-column connections were examined – nominally pinned and moment connections, for two different prototypes of the building. The layout of the connections is presented in Figure 1. The dimensions of wide RC beams were b_b/h_b = 2250/400 mm, while the dimensions of RC columns were b_c/h_c = 500/500 mm, with 2250 mm wide corbels ("capitals").

For the first prototype of the building, nominally pinned connections were used and, therefore, the columns were expected to work as cantilevers. The connections were realized via two dowels per connection with variable diameter along their length which were protruding from the corbels, as shown in Figure 1 a). The dowels were screwed into anchors at the bottom of steel sleeves in the column corbels. Steel pads were placed between the column corbels and the dapped-end of beams in order to enable relative rotations between the elements. After the seating of the dapped-end beams on the corbels was ensured, the sleeves were filled with a fine non-shrinking grout, while the connection was completed by using top anchoring plates, washers and nuts. The increased diameter of dowels was used to resist high shear demands at the beam-to-column connection (top of the corbel) as a result of their design as "strong" (overdesigned) connections for seismic loads, while the steel anchoring plates were used to resist uplifting forces due to possible overturning. After the testing of the first prototype, it was concluded that the dowels had sufficient strength to resist high shear forces with small damage, albeit high overall flexibility of structure with cantilever columns and corresponding large displacements. Compared to the reduced shear forces divided by the behaviour factor, the increase of shear forces acting on the dowels can be substantial.

The second prototype of the building was formed from the first prototype, with the activation of the previously embedded, dry mechanical connections, which were supposed to emulate moment (fixed) beam-to-column joints, as shown in Figure 1 b). These connections were provided in order to ensure the continuity of the longitudinal reinforcement crossing the joint, and were considered as ductile connections. Two thick steel plates, used to anchor the two longitudinal rebars with enlarged ends (rivets), were embedded at the top and bottom of ends of the dapped-end beams and corbels. The connection was realized with demountable bolts. In total, four longitudinal rebars at the bottom and four rebars at the top of the connections were continued through the joint. At the joint interfaces between corbel and the beam, the small gaps (approximately 10–15 mm) were filled by a mortar, albeit the control of grouting execution was not possible (grouting was not identical at all joints). It should be emphasized that the holes used for the connecting bolts were not grouted (see Figure 1 b)). During the testing, it was noted by the authors that the emulative joints experienced inelastic behaviour and concrete cracking; however, the strength of the connection was not reached since the plastic hinging was formed at the base of the columns well before the maximum capacity of the connections was reached. Although the beam-to-column joint slip was reduced dramatically in the case of moment resisting joints compared to the pinned joints, the emulative beam-to-column joint response was quite different from a rigid joint and was classified as semi-rigid beam-to-column joint.

After the testing, it seems that the structural elements were disassembled without any major destruction methods, although Bournas et al. [17] have used the term “demolition” in the presented research. The possibility of replacement or reuse was not discussed in the paper. However, since the significant damage of beam-to-column joints was not reached (except for cracking of the beams) even for high peak ground acceleration (0.3g and 0.4g), there was a considerable potential for replacement and possible reuse of beams and columns with this type of connection, especially in low and moderate seismic regions.

Zhong et al. [19] have proposed similar dry bolted connection for moment beam-to-column joints, subjected to cyclic load. The cross-sectional dimensions of the beam were b_b/h_b = 250/400 mm, while the dimensions of the column were b_c/h_c = 450/450 mm. The proposed connection consists of anchoring steel plates with welded longitudinal reinforcement, embedded in dapped-end precast beam and longer corbel (protruding beam) from RC column, with the length smaller than the depth of the beam. Steel plates are connected with demountable bolts. The gaps between steel plates, as well as at beam and column corbel, were filled with rubber layers (cushions). In addition, between the plates and bolts, flexible gaskets (i.e., rubber washers) were provided. The authors have stated that rubber layers were used in order to make a well-distributed contact and prevent any leakage of moisture from the external environment. The rubber washers were used in order to adjust (decrease) the stiffness of RC beams, i.e., to achieve “strong column-weak beam” design concept. Although the authors have indicated that this type of connection can be used for beam-to-column joint with precast slab, only the specimens which simulate the monolithic RC floor slab system with the upper concrete layer cast in-situ were experimentally tested. Therefore, the bolted connectors were only installed at the bottom of the joint, and the vertical dowel protruding from the corbel was not installed.

The cyclic test results have shown that the presented precast solution has similar strength and deformation capacity as monolithic specimen, with lower dissipation energy and approximately 60% lower stiffness. Thus, the joints were classified as semi-rigid. The major damage was concentrated at the connection, while significant cracking of the beam appeared both at sections between the connection and column face and outside the connection (away from the column).

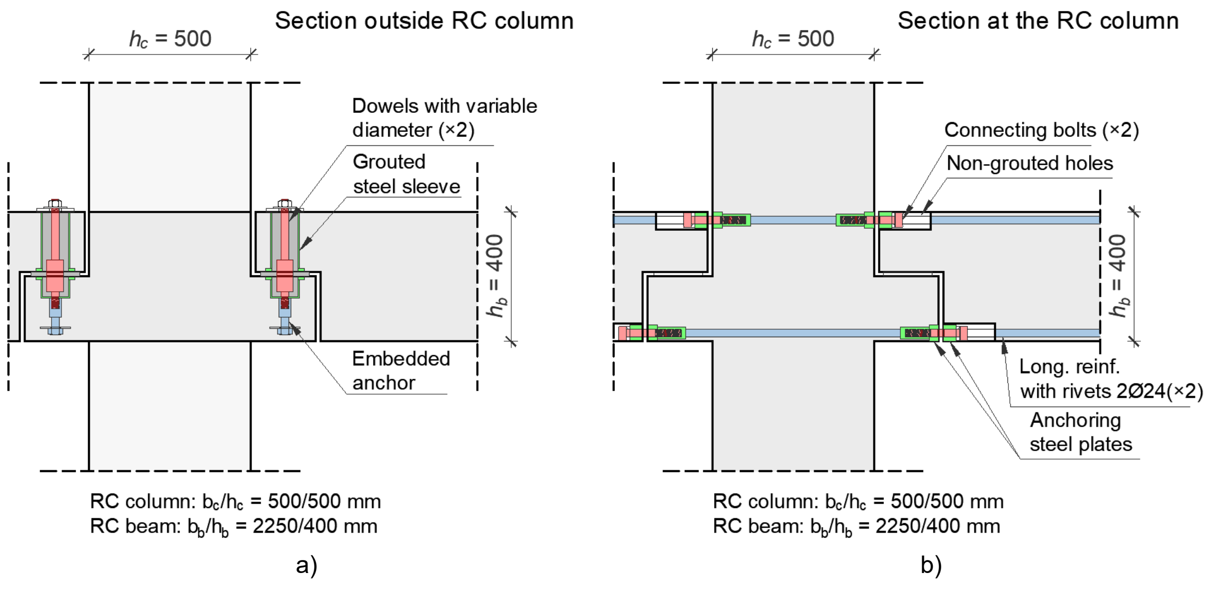


Figure 1. Beam-to-column joints with dapped-end beam: a) pinned connection with dowels, b) moment connection with thick steel plates, longitudinal reinforcement with enlarged ends and demountable bolts (adapted from [17] and [18])

Regarding the disassembly possibilities of the proposed connection, Zhong et al. [19] have concluded that the connectors, such as bolts and flexible layers, were easy to remove after testing. In other words, tested precast specimens showed the advantages of detachable precast specimens after the structure was damaged by an earthquake. However, no discussion of the possibilities of replacement nor reuse was provided in the presented research, and can be considered as low since the major damage occurred in the connection, with significant cracking in beam and protruding corbel.

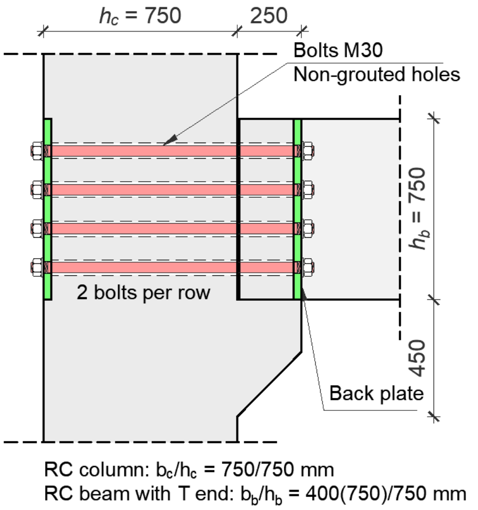
Ding et al. [20] have conducted experimental cyclic tests on the precast beam-to-column joint with dry connection realized with through anchor bolts presented in Figure 2 a). The joint comprised RC column with dimensions b_c/h_c = 750/750 mm with RC corbel, and T-end RC beam with general dimensions b_b/h_b = 400/750 mm and dimensions $b_{b,T}/h_{b,T}$ = 750/750 mm at the end of the beam. At the back of the T-end beam, the steel pressure plate was provided. Similarly, the pressure plate was embedded in the back of the RC column. After the seating of the beam on the RC corbel was ensured, the connection was realized by eight through bolts M30, inserted in the non-grouted holes, which ensured that anchor bolts were unbonded. However, the 25 mm gap between RC beam and column was grouted. The through anchor bolts were ultimately pretensioned, while the grade of the bolts was varied.

The cyclic test results have shown that failure was attained with severe damage of T-end of the precast beam, with cracks on the steel end plate and the rupture of through anchor bolts at the upper side. The column was not damaged. The joint was classified as semi-rigid, although no comparison with monolithic companion was provided.

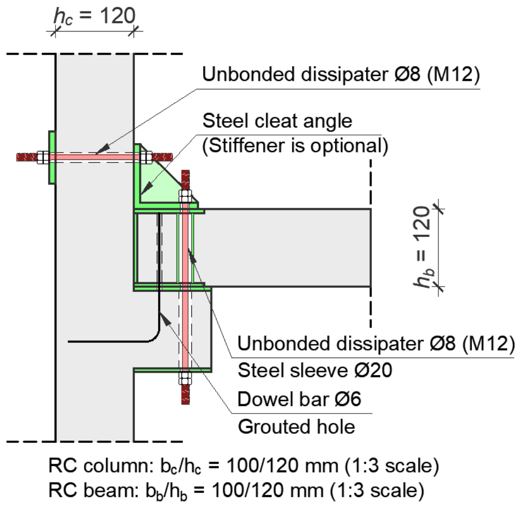
Ding et al. [20] have stated that the proposed solution of the dry connection for beam-to-column joint is suitable for post-disaster repair works, by replacing the through anchor bolts as the main load-bearing components (ductile connection), which greatly reduces the repair time and work cost of the joint components. Although it seems from the presented research that the RC column has strong potential for reuse, the repeated testing of the connection with the replaced through bolts should be conducted in order to confirm the adequacy of the RC beam for reuse.

Liu et al. [21] have conducted experimental cyclic tests on the precast beam-to-column joint, which was almost the same as it was analysed by Ding et al. [20], as shown in Figure 2 a). The main difference was that instead of the back plates without stiffeners, the haunched steel end plate (steel angles with stiffeners) was used. In addition, pressure steel plates were placed at the beam's end and RC column, at their interface. Although overall dimensions were the same as tested by Ding et al. [20], different diameter of reinforcement as well as diameters and grade of through bolts were used. Albeit the failure modes were the quite similar, the aforementioned differences led to the increase of the joint strength. The same conclusions regarding the post-earthquake repairment possibilities were drawn as highlighted by Ding et al. [20].

Krishnan and Purushothaman [22] have conducted cyclic test on one-third scaled model of semi-dry connection for beam-to-column joints, as presented in Figure 2 b). The proposed connection consists of precast RC beam (b_b/h_b = 100/120 mm) and column (b_c/h_c = 100/120 mm), steel cleat angle, unbonded vertical and horizontal dissipaters (through anchor rods) and dowel bar protruding from RC corbel. A channel section with steel duct was placed at the beam end. After the seating of the RC beam on the bearing plate at the top of the corbel was ensured, the hole for dowel bar was grouted. Afterwards, the cleat angle was placed, the unbonded anchor rods were inserted and clamped with nuts. The aim of this connection is that the anchor rods are used as dissipaters, i.e., for interaction and dissipation of energy in the connection as a sacrificial fuse (ductile connection). Therefore, their diameter was reduced along the major part of their length. The dowel bar was used for additional anchorage and prevention of the unseating of the beam in case of vertical dissipator failure. A channel section at beam's end was used to prevent concrete damage at the end of the beam. Krishnan and Purushothaman [22] adopted this concept in order to improve previous similar solutions [23], without dowel bar and channel section, which exhibited severe damage of beam's end and corbel. Furthermore, geopolymer concrete with recycled concrete aggregate (GCRCA) was used for RC elements, in order to supplement sustainable precast construction.



a)



b)

Figure 2. Beam-to-column joints with RC corbels and through bolts: a) adapted from Ding et al. [20], b) adapted from Krishnan and Purushothaman [22]

The specimens with and without stiffeners on the cleat angles were tested under cyclic loads and compared to the equivalent, fully cast-in-situ (monolithic) specimen. Only the connections with stiffeners exhibited dissipater failure (shear/tension) with mainly flexural cracks at the beam end. The RC columns remained undamaged. In all cases, precast specimens had almost the same or higher strength than the monolithic specimen. However, other seismic performance indicators were lower than for the monolithic specimen (e.g., ductility, secant stiffness, energy dissipation, etc.). Therefore, the proposed connection could be classified as semi-rigid and was recommended mainly for low seismicity regions and, in some cases, for moderate seismicity regions.

Krishnan and Purushothaman [22] have stated that both the concept of minimal structural damage and easily demountable precast systems can be achieved in proposed precast beam-to-column connection with single stiffener and unbonded dissipators. The demountability alone seems possible for all tested configurations of the connection, albeit dowel bar was in fact grouted. However, the replacement and reuse possibilities were not discussed in the paper. The RC column did not experience cracking nor damage and, therefore, there is a potential for its reuse. The possibility for reuse of RC beam should be examined in further research.

2.2 Connections with steel boxes or shoes embedded in beam

In order to simplify the construction of precast beam-to-column joints with dry connections, by excluding the necessity of temporary supports and complicated formwork, as well as to enable easier disassembly of the joint and replacement of the damaged parts of the connection, a few solutions with steel boxes and shoes embedded in beam were proposed.

Zhang et al. [24] have conducted experimental research on the dry connections realised with steel top connector and steel concealed corbel, as shown in Figure 3 a). Both of them are designed as steel boxes opened on the one side, top and bottom, respectively. A rectangular shear groove is left at the column interior face, corresponding to the concealed corbel, with bottom threaded anchor rods embedded in the column which extend out of the shear groove. After the concealed corbel is attached to the column by means of washers and nuts, the beam with the notch corresponding to the rest of the concealed corbel can be placed. At the bottom of the beam, threaded couplers ("screwed sleeves") with the threaded anchor rods are previously embedded. The bottom connection is realised by inserting and tightening the connecting bolts. At the top of the beam, the steel box connector is embedded, with long threaded anchor rods plug welded to it. In RC column, threaded couplers and anchor rods are previously embedded. The top connection is realised by inserting and tightening the connecting bolts. Boxes were not grouted.

The specimen was tested under cyclic load and compared to the equivalent, fully monolithic specimen. The results have shown that the precast joint had 40% lower initial stiffness than cast-in-situ joint and, therefore, the connection can be classified as semi-rigid. The strength of the precast joint had the same or higher resistance than cast-in-situ joint, depending on the load direction (higher resistance is achieved when bottom side is compressed, due to concealed corbel). However, the energy dissipation and displacement ductility were smaller, since the majority of the inelastic deformations were concentrated in the connectors (top box and concealed corbel, as well as connecting bolts), while the beam outside the connection experienced cracking. Failure of precast joint occurred due to concrete damage at the bottom of the beam, at the location of concealed corbel. The column experienced some cracking, but remained without significant damage.

Zhang et al. [24] have stated that the proposed dry connection can concentrate the plastic damage on the steel connectors that are easy to replace, which could improve the recoverability of structural performance and building functions. However, due to high yielding strength, steel boxes cannot be used as damping devices and, therefore, the authors have recommended that the material of the boxes can be replaced with mild steel or the reduction of cross-sectional area can be implemented, in order to improve energy dissipation capacity of the joint. In the author's opinion, further research is needed in order to establish the possibility for replacement and reuse of RC columns and beams. Since the columns remained elastic with slight cracking, the possibility for reuse is considerable.

Zhang et al. [25] have conducted experimental shaking table tests on 1/2-scale specimen of the precast concrete three-storey frame structure with fully demountable dry connections between all structural elements – slabs, beams, columns and foundations. The layout of the connection in beam-to-column joints was similar to the presented in Figure 3 a). The main difference was that all threaded anchor rods in RC beams were connected to steel box connector and concealed corbel by means of threaded couplers (or extended nuts). The same applies for the top connectors in RC column, while the bottom threaded rods were the same as shown in Figure 3 a). Furthermore, the longitudinal ribs in steel boxes were designed as energy-dissipation plates, made out of soft steel (ductile connection). The structure was designed with aim that the initial stiffnesses of steel box connectors in precast beam-to-column joints are equal to the elastic bending stiffnesses of cast-in-situ beam-column joints with the same cross-sections. The structure was designed to sustain peak ground acceleration (PGA) of 0.2g. The test results have shown that for PGA of 0.2g, cracks in the beams as well as relative sliding between steel box connectors and the beams occurred. For PGA of 0.4g, damage was mainly concentrated on the box connectors and the concrete of some beams (yielding of the beam reinforcement occurred), while the columns remained undamaged. The maximum storey drift was 0.166% for PGA of 0.07g (frequent earthquakes) and 0.935% for PGA of 0.4g (rare earthquakes). Apart from quick and relatively easy assembly of the structure with the proposed connections, Zhang et al. [25] have highlighted that damaged beams were easily removable and thus replaceable, which contributes to the structural sustainability in terms of seismic resistance. Since the columns remained elastic, the possibility for reuse is considerable.

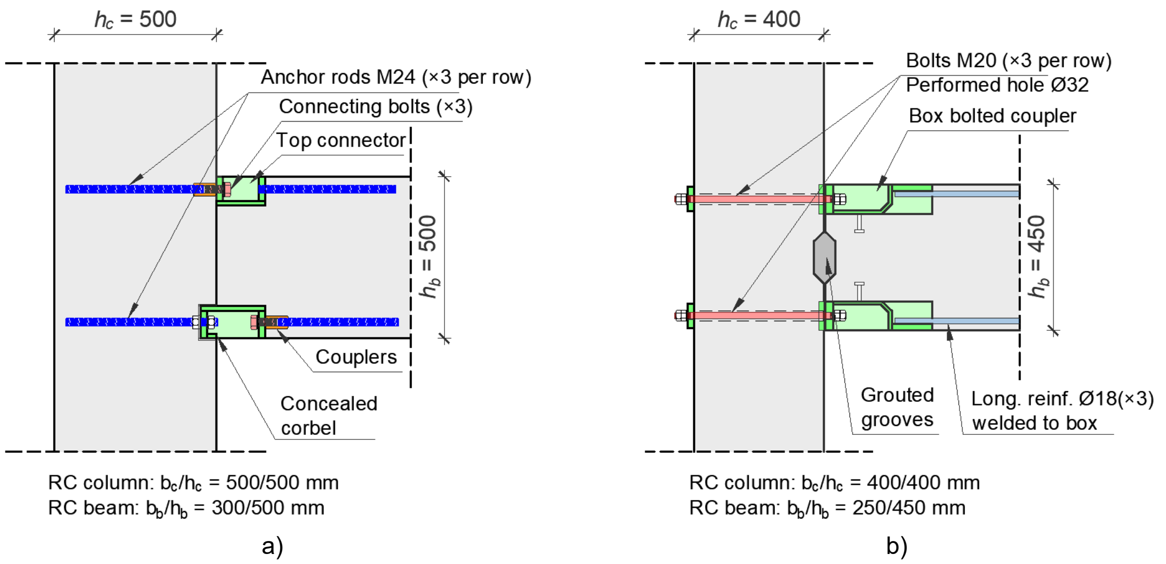


Figure 3. Beam-to-column joints with steel boxes: (a) connection with concealed corbel end threaded rods (adapted from Zhang et al. [24]), (b) connection with grouted grooves and through anchor bolts (adapted from Zhang et al. [26])

Zhang et al. [26] have proposed the alternative layout of beam-to-column connection with steel boxes, as presented in Figure 3 b). The top and bottom boxes are embedded in beam, with welded longitudinal reinforcement. The connection is realised by clamping and preloading horizontal through anchor bolts embedded in RC column. The shear key at beam-column interface was also provided, which was ultimately grouted. The basic concept was to use high strength bolts and boxes, in order to ensure that plastic hinges in the beams are formed away from the column face. Compared to the monolithic companion, the cyclic test results indicated that the proposed precast beam-to-column joint can achieve higher strength and similar secant stiffness, displacement ductility and cumulative energy dissipation. The possibilities for the disassembly, replacement and reuse were not discussed in the paper. However, due to nature of the connection that comprises embedded steel box with welded reinforcement and embedded anchors, it can be considered that the disassembly of the joint is possible while only the replacement of the beam and reuse of column after earthquakes can be considered reliable.

Liu et al. [27] have suggested that the demountable beam-to-column connection can be achieved by connecting steel shoes embedded in RC beam and mechanical couplers with rebar anchor embedded in RC column by demountable threaded bars, in a similar manner as it is usually done in column-to-foundation connections (see Section 3.2). It was argued that the connection can be achieved without using RC corbel; however, considering the effect of errors in elements' dimensions and installation on the construction, there is a certain gap between the contact plane of beam and column which can affect the shear resistance of the connectors. Therefore, the single shear tests on the single connectors were performed in direct shear and flexural shear conditions. It was demonstrated that in case of larger gaps between beam and column, the failure of couplers can occur. The simple method for prediction of shear resistance of the single connector with mechanical coupler in relation to the gap width was proposed.

2.3 Connections with steel end plates or steel angles

Although the solutions for demountable beam-to-column joints presented in previous subsections showed generally good performance in terms of strength and displacement capacity, with assured "strong column-weak beam" concept, most of them had lower stiffness and dissipation energy capacity than corresponding monolithic joints, which classifies them as semi-rigid with limited energy dissipation capacity. In order to improve these characteristics and the simplicity of demountable precast beam-to-column joints, a few research groups have conducted tests on the connections which employ steel end plates or steel angles, with demountable connecting bolts or through anchor bolts located outside beam cross-sectional dimensions.

Senturk et al. [28] have conducted experimental tests on precast beam-to-column joints realised by steel end plate with stiffeners embedded in beam and demountable bolts, as presented in Figure 4 a). Longitudinal reinforcement with rivet head (enlargement) at the end was inserted in the end plate. The extended nuts (couplers) were welded to the positioning steel plate and embedded in the precast column. In addition, the anchoring reinforcement was welded to embedded extended nuts. Different layouts of anchoring reinforcement in RC columns were examined. The connection is realised by connecting beam end plate to the column by inserting demountable bolts, which were pretensioned. The bolts and extended nuts were designed to remain in the elastic range (overdesigned connection).

The specimens were tested under cyclic load and compared to the equivalent, fully cast-in-situ (monolithic) specimen. The failure modes of precast joints were very similar to the failure modes of monolithic specimens, with considerably higher ductility, dissipation energy and ultimate deformation capacity while providing the same level of resistance and initial stiffness. Localized damage of the connection was also prevented by distributing the plastic deformations throughout the beam element. The columns remained undamaged. Therefore, it was stated that the design of the connection can be based on the traditional approaches.

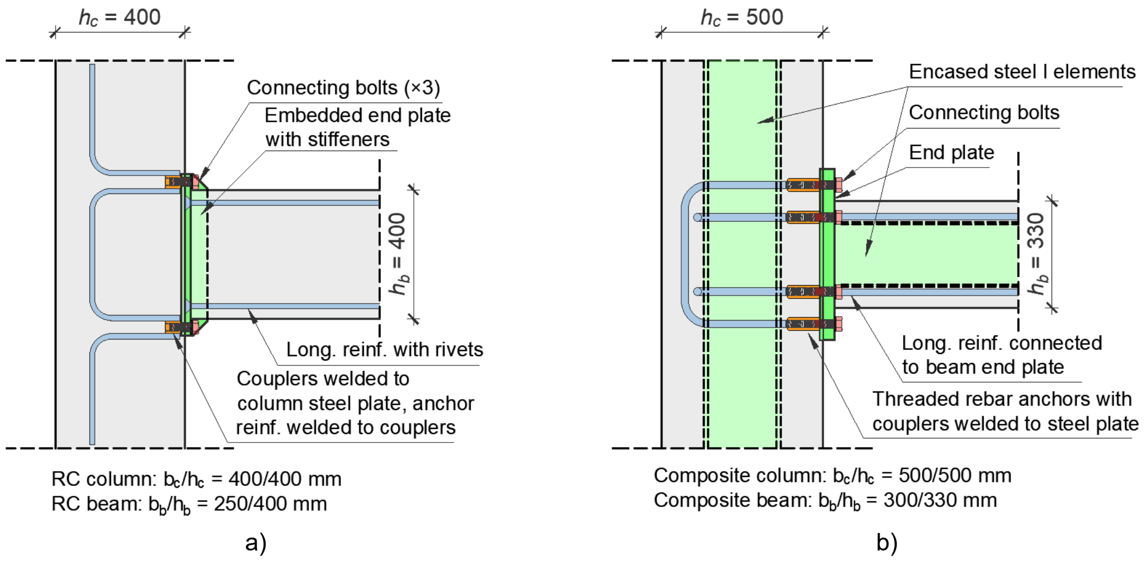


Figure 4. End plate connections in beam-to-column joints: (a) RC joint end plate and stiffeners (adapted from Senturk et al. [28]), (b) composite joint with extended end plate without stiffeners (adapted from Nzabonimpa et al. [29])

Senturk et al. [28] have stated that the developed bolted moment connection system for precast beam-column joints of RC frame structures in seismic regions, offers several advantages such as rapid assembly and disassembly, reusability, and replaceability if damaged during an earthquake event. It should be noted that the reuse possibilities relate solely to the RC column, while the reuse of the beams should be further tested.

Nzabonimpa et al. [29] have experimentally tested similar dry connection, with the use of extended end plate without stiffeners, in the composite precast beam-to-column joints, as shown in Figure 4 b). In precast beams and columns, steel elements with I-section were embedded. In the column, the threaded couplers were welded to the positioning steel plate, while anchoring was achieved with U-shaped reinforcement with threaded ends inserted in couplers. In composite beam, the steel element was welded to the end plate, while the type of connection between longitudinal reinforcement and end plate was varied (threaded ends with nuts, welded to plate). The connection is realised by inserting the demountable bolts in couplers. The same layout of the connection was proposed for the purely precast reinforced concrete joints, albeit this connection was not experimentally tested.

The specimens were tested under cyclic load and compared to the equivalent, fully cast-in-situ (monolithic) specimen of composite beam-to-column joint. The testing results showed that the best performance of the precast joint was exhibited when thick end plate is used (45 mm), with the similar overall behaviour as the monolithic joint, i.e., necking and fracture of the longitudinal reinforcement in the beam and without damage in column (overdesigned connection). The disassembly, replacement and reuse possibilities were not discussed in the paper. However, in the authors' opinion, the similar conclusions can be drawn as it was done by Senturk et al. [28].

It should be noted that the anchoring system in columns in the presented research, i.e., extended nuts and couplers with anchor reinforcement, provide relatively simple disassembly process and reuse of the column and embedded connectors. In order to optimise these types of connections, a group of researchers from University of Belgrade have conducted studies on single connector embedded in precast concrete elements, subjected to pure tension and pure shear loads [30]- [33]. The guidelines for load-displacement behaviour of single anchor were provided, which can be used for estimation of the load-displacement behaviour and optimisation of precast beam-to-column joints.

Aninthaneni et al. [34] have conducted experimental tests on the emulative dry connections in beam-to-column joints, with steel end plate embedded in the beam and through anchor bolts (rods), as shown in Figure 5 a). The cross-sectional dimensions of the beam were b_b/h_b = 350/400 mm, while the dimensions of the column were b_c/h_c = 700/600 mm. The end plate with vertical and horizontal stiffeners is embedded in the beam, with longitudinal reinforcement (25 mm in diameter) with bends welded to the end plate and anchored by passing through the slotted holes in horizontal stiffeners. In addition, two 25 mm diameter bent-up bars were also welded to the embedded plates to effectively transfer the shear force and relocate the plastic hinge to in front of the connection. The connection is realised by inserting through anchor bolts and clamping them with nuts to the steel plate at the back of the column. The anchor bolts were pretensioned. The larger lever arm of the through bolts and the use of the stiffeners was adopted in order to increase bending resistance and stiffness of the connection, which ensures that the connection remains rigid and elastic while the beam reaches its bending moment capacity (overdesigned connection).

Experimental cyclic tests were conducted on two specimens with the identical layout. In the first test, the joint was tested until the failure which occurred in the precast beam outside of the connection. Afterwards, the connection was disassembled and the beam was demounted. In the second test, the new beam was connected to the column which was used in the first test. The testing results of these two specimens showed that there was no significant difference between their behaviour, with the plastic hinging occurring in the beam outside the connection. It was demonstrated that the behaviour of the tested joint was the very much similar to the wet jointed/monolithic concrete beam-to-column joints. It was also demonstrated that the tested end plate connection has better energy dissipation than the connection with some ductile connectors.

Aninthaneni et al. [34] have stated that the proposed dry beam-to-column connection is demountable, and that in the laboratory the damaged beam could be replaced with a new beam without much difficulty. More detailed discussion regarding the feasibility and replacement of the beams of building structures in real-life conditions is discussed by Aninthaneni and Dhakal [35]. If the damage in a demountable precast RC frame structure is restricted solely to the beams, the replacement of the damaged beams after earthquake with the identical new ones would result in a frame structure as it was before the damage. In other words, the reusability of the RC column was confirmed by test results in the presented research.

Li et al. [37] have conducted the cyclic tests on the embedded end plate connection similar to the one presented in Figure 5 a), with the aim to optimize the components of the connection. The cross-sectional dimensions of the beam were b_b/h_b = 200/500 mm, while the dimensions of the column were b_c/h_c = 400/400 mm, with the 620 mm distance between M24 through bolts. The main tested parameters were: thickness of end plate (16 mm and 20 mm), use of vertical stiffeners (with and without) and welding of longitudinal reinforcement to horizontal stiffeners (with and without). The bent-up bar with 12 mm in diameter were not welded to the horizontal stiffeners.

The specimens were tested under cyclic load and compared to the equivalent, fully cast-in-situ (monolithic) specimen. The results have shown that the final failure mode of precast concrete connections depended highly on the rotational stiffness of the connection. In case of thinner end plate without vertical stiffeners, the failure occurred in the end plate, with significantly lower resistance than obtained for monolithic specimen, but with better ductility. For thicker end plate with stiffeners, the crushing of concrete occurred in the beam, at the location of the connection, with increased moment capacity, but with decreased ductility. Li et al. [37] have highlighted the importance of the adequate detailing of beam reinforcement as well as the connection itself, in order to obtain adequate performance of the joint. Since the column and joint core remained intact, it was stated that it is possible to rapidly replace the damaged components after earthquake.

Aninthaneni et al. [36] have conducted cyclic experimental tests on the dry connection realised with steel angles with stiffeners, and horizontal and vertical through anchor bolts, as shown in Figure 5 b). Steel ducts were embedded in the beam and column, with corresponding reinforcement detailing, in order to enable the insertion of through anchor bolts and the connection with the steel angles. Bolts were pretensioned after the assembly of the joint. In addition to the presented layout (type 1 connection), the second layout (type 2 connection) with the embedded steel web plates in both beam and column, which are connected with the four bolts at the beam-to-column interface, was tested. It should be noted that the beam edge distance from the vertical anchor bolts was smaller than in case of type 1 connection. In both cases, natural rubber sheet or dental plaster were used to achieve good surface contact between the steel connection and the precast elements, since the precast elements' surfaces were not perfectly levelled and legs of steel angle were not exactly perpendicular.

The cyclic test results showed the type 1 connection has the similar resistance as predicted for cast-in-situ joint, with considerable displacement ductility. At the final stages of testing, the slip between the connection and the beam exceeded the clearance between the bolts and the ducts, and the bolts started to bear against the steel duct which induced bearing stress (i.e., bursting stress) into the concrete resulting bearing and direct tensile split vertical cracks, which ultimately passed through beam cross-section. In case of type 2 connection, similar behaviour was noticed, with significantly higher degree of concrete damage and spalling due to smaller edge distance of the vertical bolts, with minor cracking outside of the connection. However, the damaged beam was removed, repaired with high-early-strength grout, and reused in the subsequent test. Ultimately, the repaired specimen was able to reach the nominal strength of the cast-in-situ joint; however, significant strength degradation after 2.5% lateral drift was detected.

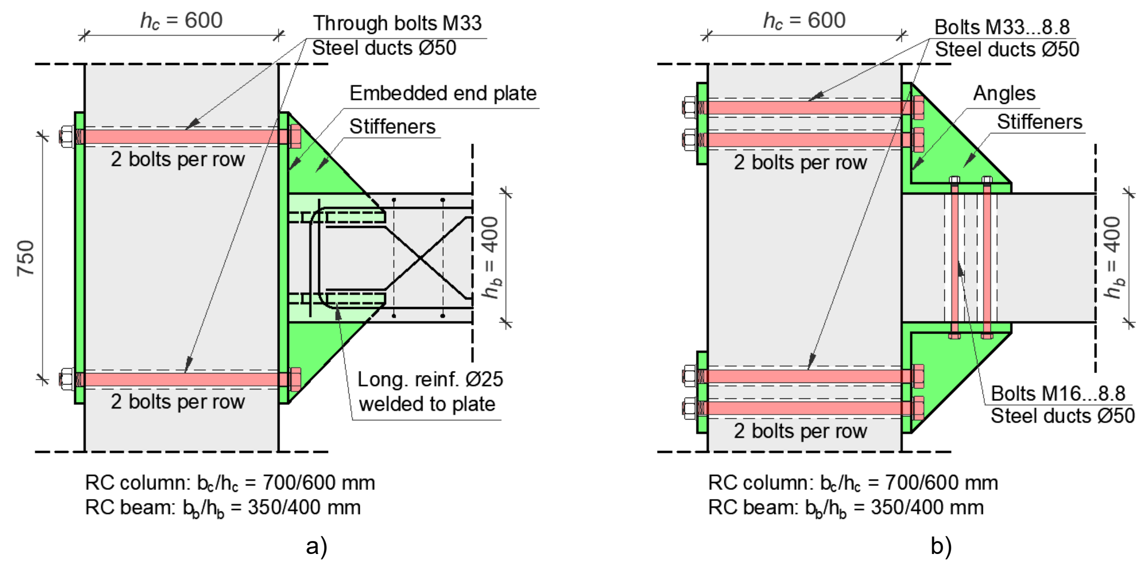


Figure 5. Moment beam-to-column connections with through bolts: a) connection with embedded steel end plate and stiffeners (adapted from Aninthaneni et al. [34]); b) connection with steel angles and stiffeners (adapted from Aninthaneni et al. [36])

Aninthaneni et al. [36] have stated that the precast elements can be demounted and that the undamaged components can be reused, while the damaged beams can be replaced with the new ones in a relatively short time.

2.4 Connections with steel dampers and fuses

In order to improve the shortcomings of previously presented connection solutions, in terms of reusability of all precast concrete elements, an effort was made by several research groups to investigate the possibility of using different steel (metallic) elements as damping devices and fuses – sacrificial elements to dissipate seismic energy. These steel elements would provide sufficient stiffness and damping simultaneously, while reducing the damage in the precast beams and columns (ductile connections). In addition, some of the proposed connections were oriented towards establishing detachable configurations, with aim not only to simplify the assembly and reduce construction time and costs, but also to provide the possibility of disassembly as well as replacement (repair) after earthquake events. It should be noted that the simplicity of the connection and manufacturing process, as well as the total amount of steel used in the connection vary between the proposed solutions.

Wu et al. [38] have experimentally tested the beam-to-column joint with demountable connection realised by steel damper, as shown in Figure 6 a). The metallic damper was composed from short "dog-bone" element (i.e., weakened I-section element, with gradually reduced flange width in the middle section by 50% compared to the full flange width), and thick end plates welded to the "dog bone" element. In precast beam, longitudinal reinforcement with threaded ends was placed in the positioning steel plate at the beam end, with threaded ends protruding from the beam. In the column, threaded couplers (sleeves) were welded to the positioning steel plate while U-shaped anchoring reinforcement with threaded ends was inserted into couplers. The connection is realised by firstly connecting the damper to the beam's end by nuts, then lifting the beam and damper to the expected location and finally connecting the damper to the column by inserting the demountable bolts into couplers.

The specimen was tested under cyclic load and compared to the equivalent, wet jointed precast specimen. Test results showed that the significant plastic damage was concentrated on the damper while column and beam remained elastic (tiny cracks appeared on the beam with no damage on the nuts and threads on the end of longitudinal reinforcement). The specimen with damper showed somewhat lower flexural strength and stiffness compared to specimen with wet joint, although the dog-bone damper was weakened to act as a sacrifice element (in order to protect the concrete components). In addition, the displacement capacity, energy dissipation, as well as strength and stiffness degradation were better than in case of wet jointed specimen.

Since the nuts and threaded ends of longitudinal reinforcement were not damaged, Wu et al. [38] have stated that the connection was easily fully disassembled. Although the possibility of reuse of precast elements was not explicitly analysed in the paper, in the authors' opinion, there is a high potential for reuse of beams and columns, with replaced "dog bone" dumper.

Bai et al. [39] have conducted experimental tests on connection with reduced beam steel section ("dog-bone" element), similar to the one presented in Figure 6 a). The main difference was that the anchoring reinforcement in column and beam were welded to embedded steel elements with end plates, which protruded from the column and beam. The connection was realised by connecting embedded steel elements to the "dog-bone" element by means of high strength bolts. Different configurations of the connections were tested. The test results showed that the strength of non-replaceable parts should be at least 50% higher than replaceable parts of the connection in order to obtain desirable behaviour of the joint – the plastic deformation in the "dog-bone" element and negligible damage of precast elements (ductile connection). In this regard, the authors have performed tests on the three "post-repaired" specimens with reused precast elements and replaced "dog-bone" element. These specimens exhibited stable seismic behaviour, thus proving the possibility of reuse for precast beams and columns.

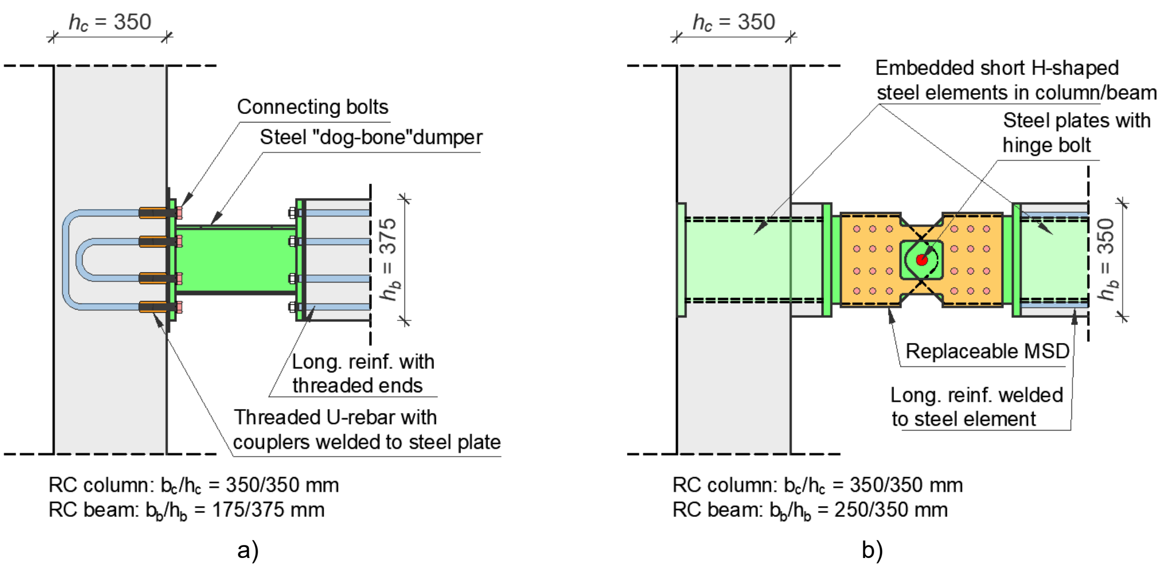


Figure 6. Beam-to-column joints with steel dampers: a) connection with steel "dog-bone" element (adapted from Wu et al. [38]), b) connection with steel "multi-slit devices – MSD" (adapted from Huang et al. [40])

Huang et al. [40] have experimentally investigated another type of energy dissipation connection system in precast beam-to-column joints, as shown in Figure 6 b). The connection consists of H-shaped steel elements embedded in column and beam, with beam longitudinal reinforcement welded to the element. The embedded H-shaped elements of beam and column have extension plates which protrude from their ends. These extension plates are connected with pinned joint at the centre of the connection, via hinge bolt, in order to resist shear loads. Afterwards, the “multi-slit devices – MSD” (steel plates with slits/holes) are connected to the protruding extension plates via bolts. These MSDs are used as one of the main structural elements, with aim to provide flexural capacity for the joint, and additional energy dissipation capacity through the plastic flexural deformation of steel. Different layouts of slits in MSD were analysed in the presented study.

The specimens were tested under cyclic load and compared to the equivalent, monolithic specimen. In case of the proposed connection, the plastic deformation and damage were mainly concentrated on the steel strips of MSD, while the precast beam and column remained within the elastic stage during the entire loading process. The overall seismic performance of the precast joints was better than that of the monolithic joint, in terms of the resistance, deformation capacity, energy dissipation capacity and ductility. In addition, the initial stiffness of the precast joints was also greater than or similar to that of the monolithic joint.

Huang et al. [40] have stated that MSD plates can be replaced directly during post-earthquake recovery in order to conduct the rapid repair of the structures. They discussed the feasibility and effectiveness of the proposed precast joint from the perspective of seismic safety performance.

The same concept of connection with hinge bolt and dissipating steel plates with slits, but with different layouts of the connection (steel elements, anchoring systems, level of complexity, etc.) were investigated by different authors [41], [42], [43]. The conclusions regarding the overall behaviour, damage distribution and the replacement possibilities of precast joints were similar.

2.5 Connections with partial in-situ concreting

Over the past years, a significant effort was made towards reducing the in-situ concreting and grouting in precast beam-to-column joints with nominally wet connections, in order to achieve emulation of monolithic joints and provide the possibility of deconstruction and reuse at the same time.

Ong et al. [44] have suggested that the demountable beam-to-column joints can be achieved with steel elements (I-section element with end plate) embedded in precast beam and protruding beam from the column, with minimal in-situ concreting, as shown in Figure 7 a). The longitudinal reinforcement is welded to the embedded I-section element. After the alignment of beam and column was achieved, the connection is realised with connecting bolts, tightened with the design torque. Finally, the joint is encased with a suitable cast-in-situ material. This type of connection can be used for the precast slab without topping (i.e., without continuity of longitudinal reinforcement, as shown in Figure 7 a or with cast-in-situ connection with the slab. In addition, for reused precast beam in the structure with different spans than the spans in the original building, the solutions with the span extension units (steel elements), connected by bolts and oversized holes or by welding, were proposed.

Instead of beam-to-column joints, four-point bending experimental tests under monotonic load were conducted on the precast beams with different configurations (without topping and continuity of longitudinal reinforcement, with topping and continuity of longitudinal reinforcement, and with bolted extension unit). These tests were conducted in order to simulate the behaviour of beam-to-column joints with hogging moments. After the initial tests under serviceability loads, further examination of reused beams was conducted. Tested reused specimens showed satisfactory behaviour.

Ong et al. [44] have provided detailed description of the deconstruction process of the initially tested beams under serviceability loads, as well as repair and reconstruction process for the reused beams, which were tested until

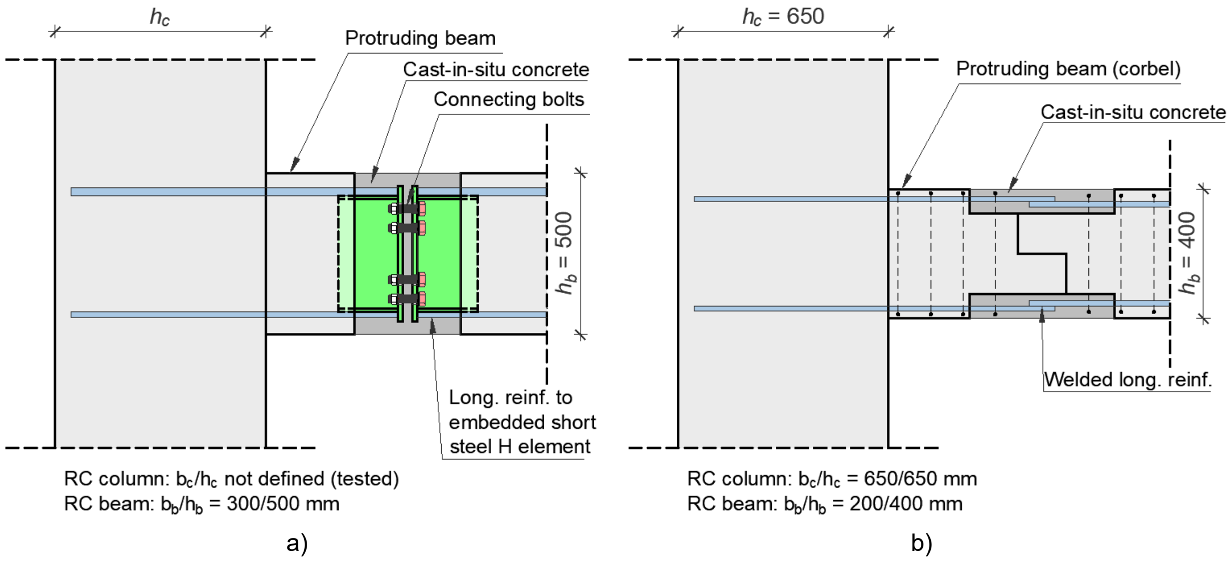


Figure 7. Beam-to-column joints with partial cast-in-situ concreting: (a) bolted connections with embedded steel plates (adapted from Ong et al. [44]), (b) connections with dapped-end beam and welded longitudinal reinforcement (adapted from Xiao et al. [45])

failure. During deconstruction, demolition work was noisy and time consuming, albeit small concrete volume which was removed. Furthermore, there was a necessity for cutting some bolts, due to damage of the threaded part of the bolts, while some damage to the nearby stirrups and embedded steel elements was also noticed. Several recommendations were given for mitigating the observed difficulties in deconstruction.

Xiao et al. [45] have conducted cyclic experimental tests on the demountable precast beam-to-column joints, with dapped-end beam and welded longitudinal reinforcement, as shown in Figure 7 b). The connection included protruding beam extending from the column, with small corbel for supporting precast beam. After the seating of the beam was ensured, the longitudinal reinforcement was overlapped and welded in-situ. Finally, the small amount of in-situ concreting was provided. The connection was moved away from the column face, in order to ensure plastic hinging at the beam near the column, and avoid major damage in the connection. In order to improve sustainability of the proposed solution, specimens with natural aggregate concrete (NAC) and recycled aggregate concrete (RAC) were tested.

The specimens were tested under monotonic and cyclic loads, and were compared to the equivalent, monolithic (cast-in-situ) specimen, both with NAC and RAC. The testing results have shown that the proposed precast beam-to-column joint has similar strength, initial stiffness, stiffness deterioration, energy dissipation and ductility as monolithic joints, while the majority of the damage occurred in the protruding beam. In general, RAC specimens showed relatively lower ultimate strength compared to the NAC specimens, both for monolithic and precast joints, but within a reasonable limit.

Xiao et al. [45] have conducted deconstruction of precast specimens after testing. The small portion of cast-in-place concrete was carefully removed with mechanical jack-hammer, in order to ensure minimal damage as possible. Afterwards, the welded longitudinal reinforcement was cut off, and the specimens were separated in two parts. It was stated that during the execution of deconstruction process, mechanical removal process was not so difficult and very little debris was generated. The suggestions for improving the connection were highlighted, primarily the increase of length of the welded longitudinal reinforcement in order to achieve full continuity of reused beam. However, the behaviour of the reused precast joint was not examined. Since the connection is placed outside the critical region, with major damage occurring in the protruding beam of RC column, the proposed connection can provide potential reuse only of precast beam.

In the subsequent research, Xiao et al. [46] and Ding et al. [47] have performed cyclic experimental tests on the similar precast beam-to-column joint as shown in Figure 7 b), with aim to experimentally investigate the reuse possibility of precast beams. The main difference in the connection layout was the addition of embedded steel elements, both in precast beam and protruding beam from precast column, in order to provide shear transfer between beam and column. Specimens of interior beam-to-column joints, with natural aggregate concrete (NAC) and recycled aggregate concrete (RAC), were tested. The specimens with reused beams and new columns were constructed from the original specimens which were previously tested under cyclic displacements corresponding to 2% interstorey drift ratios. It was demonstrated that both original and reused precast joints have adequate seismic performance, similar to the monolithic joints. Xiao et al. [46] have conducted Life cycle assessment (LCA) and showed that the proposed precast beam-to-column joint would increase the carbon emission at the first construction process, but would reduce total carbon emission. In addition, the use of RAC in the proposed precast joint had the lowest carbon emission, reduced by approximately 13% compared with monolithic NAC joint.

2.6 Post-tensioned connections with dissipating elements

Although generally classified as non-emulative joints, precast beam-to-column joints connected with unbonded post-tensioned strands were proven to be an adequate solution for multi-storey precast moment frame building structures, with quicker construction speed, large self-centring capabilities (small residual deformations) and an ability to undergo large nonlinear lateral displacements without significant damage of structural elements (columns, beams and unbonded strands remain in the elastic range). However, their greatest setback is that the lateral displacement demands under seismic action may be higher than acceptable, due to insufficient energy dissipation capacity [48]. In order to address this issue, several solutions which could enhance energy dissipation of the joints were proposed in the past decades, although generally increase the cost and difficulty in construction. In recent years, apart from the enhancement of energy dissipation, the focus of the research was on the disassembly and replacement of the specific connection components which are expected to dissipate energy during earthquakes, i.e., steel components such as unbonded mild steel bars, steel angles, steel braces, etc.

Wang et al. [49] have conducted cyclic experimental tests on the interior post-tensioned precast beam-to-column joints with steel jackets and mild steel bars, as shown in Figure 8 a). Steel tubes were horizontally embedded into the column and adjacent beams at the centreline of the beams, in order to accommodate the unbonded post-tensioned strand used to pre-stress the beams against the column. The steel jackets with welded studs were anchored to the end of precast beams, which were used to prevent concrete spalling and crushing at the interface with the column during loading. For the same reason, the steel jacket was used for the column. Supporting angles were connected to the column, which were used to provide seating of the beam and transmission of shear forces. Replaceable steel angles were connected to the beam jacket via bolts (four angles per beam). Replaceable mild steel bars with reduced cross-sectional area along the length were inserted into previously embedded tubes into column, and were subsequently welded to the replaceable angles. The mild steel bars were used as dissipaters. Similar layout of the connection was also examined, with mild steel bars bolted to the steel jacket at the beam's end, without replaceable angles, albeit partially encased beam jackets (with only vertical plates and end plate) and column without steel jacket were used.

The specimens were tested under cyclic loads, and were compared to the equivalent, monolithic (cast-in-situ) specimen. For precast specimens, yielding mainly occurred in the mild steel bars, while the post-tensioned strand and longitudinal reinforcement in the beams remained in the elastic region. The width of concrete cracks outside the connection remained below the value of 0.4 mm at 2% drift ratio. Residual deformations were negligible, even after drift ratios of approximately 5.5%. The strength of precast joint was similar to the monolithic joint. It was demonstrated that the precast joint has somewhat higher cumulative energy dissipation capacity up to 2%, while for higher drifts

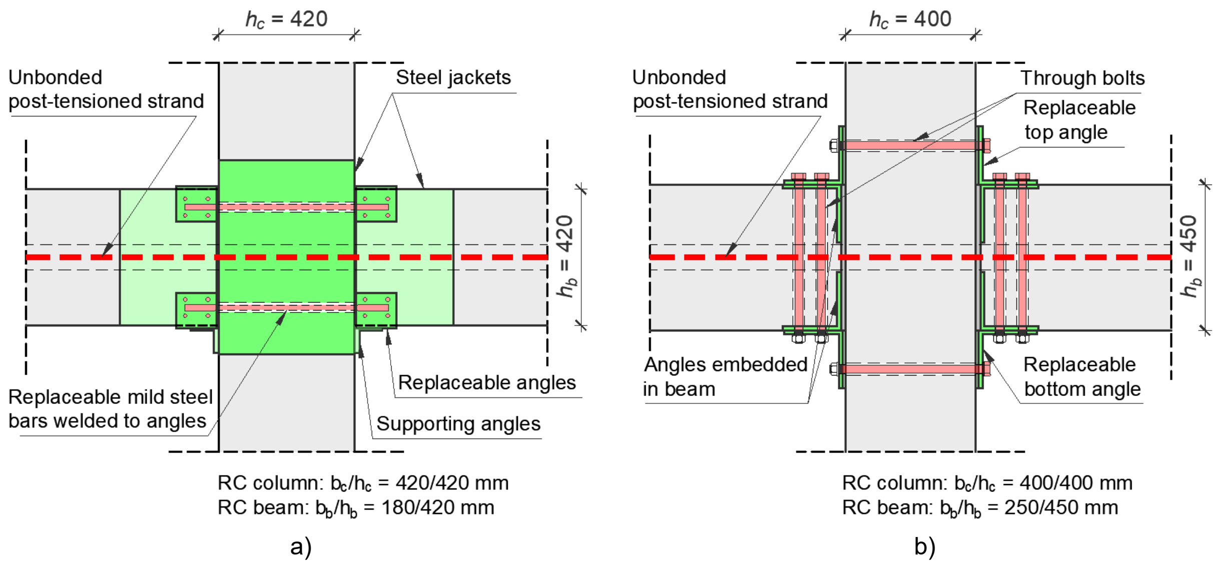


Figure 8. Beam-to-column joints with post-tensioned connections: (a) connections steel jackets and replaceable bar and angles (adapted from Wang et al. [49]), (b) connections with replaceable steel angles and through bolts (adapted from Cai et al. [50])

monolithic joint has considerably higher energy dissipation capacity. The initial stiffness of precast joint was the same as that of monolithic joint until the opening of the gap between beam and column; afterwards, however, the stiffness of precast joint declines.

After first testing, the precast specimen was retrofitted by replacing mild steel bars and post-tensioned strands, and reused for additional testing. The mild steel bars and the replaceable angles were removed by cutting off mild steel bars (welded to the replaceable angles). It was stated that the replacement of mild steel bars after loading of the original specimen took only 2 hours, with 2 workers. The seismic performance of the reused precast joint was very similar (and even better for some parameters) to the performance of original precast joint. Thus, the possibilities of disassembly, replacement and reuse of both precast column and beam were confirmed.

Cai et al. [50] have conducted cyclic experimental tests of the exterior post-tensioned precast beam-to-column joints with replaceable steel angles and through bolts. The layout of the joint is presented in Figure 8 b) for interior joint, for simplicity reasons. The replaceable angles were used as dissipaters (dissipating fuses). The bottom replaceable angles were also used for seating of the beam. The connection is realised by connecting replaceable top and bottom angles to precast beam and column, with vertical and horizontal through bolts. The bolts were pretensioned after the assembly process was completed, and designed for strength. In addition, at the end of the beams, the embedded steel angles were provided in order to prevent local damage and crushing of the concrete at the beam-column interface. The effect of different parameters was investigated, such as initial post-tensioning force, thickness of steel angles, angle dimensions, beam depth, etc.

Test results have shown that cracking only occurred at the beam end, and near the steel angles in all specimens. It was demonstrated that an increase in the initial post-tension force, beam depth, or leg thickness of the steel angles could lead to an improvement in load-carrying capacity and increase in the initial stiffness of tested specimens. The increase of thickness of the steel angles also could lead to a significant increase in the dissipated energy.

After initial testing, specimen with the best performance was retrofitted by replacing demountable steel angles and was reused for additional test. This reused specimen exhibited almost identical hysteretic behaviour as the original specimen, with slight reduction im strength as well as the reduction of initial stiffness to some extent, due to cracking of beam after the initial testing of the original specimen. However, in the late loading stages, the stiffness, energy dissipation and maximal displacement of both specimens were very similar. Somewhat larger residual drifts of reused specimen were detected. Since the overall behaviour of the reused specimen was similar to the original specimen, the possibilities of disassembly, replacement and reuse were confirmed.

3 Column-to-column and column-to-foundation connections

Connections between precast columns and foundations, as well as between adjacent column parts, play a critical role in ensuring reliable force transfer and overall structural integrity. These joints must provide adequate strength, stiffness, and ductility while maintaining stable behaviour under service and seismic loads. In seismic regions, particular attention is given to the ability of connections to dissipate earthquake-induced energy. This can be achieved either by allowing the formation of plastic hinges, similar to conventional monolithic RC systems, or by enabling controlled rocking mechanisms at the column base, which reduce residual deformations and limit structural damage [51].

To satisfy these performance requirements, connection systems must be carefully designed in accordance with capacity design principles, ensuring that inelastic deformations occur in predetermined and ductile regions while protecting critical connection components [52]. In recent years, increasing emphasis has been placed on the development of innovative demountable connections, which facilitate not only structural performance but also disassembly, repair, and potential reuse of structural elements. Such systems are typically based on mechanical joining techniques and are classified as dry or semi-dry connections, distinguishing them from traditional cast-in-place solutions [52]–[54].

A review of the available literature indicates that demountable column-to-foundation (CF) and column-to-column (CC) connections can be broadly categorized according to the type of mechanical device used for force transfer. The most commonly investigated and applied systems include: steel flange plate connections, column shoe (steel shoe) connections, and steel jacket connections. Each of these systems exhibits distinct load transfer mechanisms, seismic performance characteristics, and levels of demountability and reusability, which are discussed in detail in the following sections.

3.1 Connections with steel flange plate

This type of connection is used for column-to-foundation (CF) and column-to-column (CC) connections. The connection consists of column longitudinal reinforcement (CLR), a steel flange plate, and external bolts or anchor bars. The CLR is first connected to the steel flange plate, after which the steel plate is fixed to the foundation or lower column using anchor bars or bolts. Figure 9 illustrates the typical configuration of CF and CC connections incorporating a steel flange plate. In practice, the flange plate is often strengthened by welded steel jacketing plates and stiffeners, as also shown in Figure 9. The dimensions of the steel flange plate generally exceed those of the column cross-section, allowing the anchor bars or bolts to be positioned outside the column perimeter. Various types of anchor bars are used in such connections, including threaded bars, reinforcing bars, and hybrid systems combining reinforcement, couplers, and bolts, as shown in Figure 9. Anchor bars may be either straight or bent, and may be threaded along their entire length or only over the portion extending from the foundation.

To ensure effective force transmission between the CLR and steel plate, several solutions have been proposed: welding, CLR with rivet head, and threading the CLR bar end followed by fastening with a nut or a coupler-bolt assembly [56]. These solutions are illustrated in Figure 10. In welded configurations, the CLR may be connected to a steel jacketing plate (Type A), a steel flange plate (Type B), or to weldable steel sleeves (Type C), as shown in Figure 10. Type D in Figure 10 represents solution where CLR had rivet

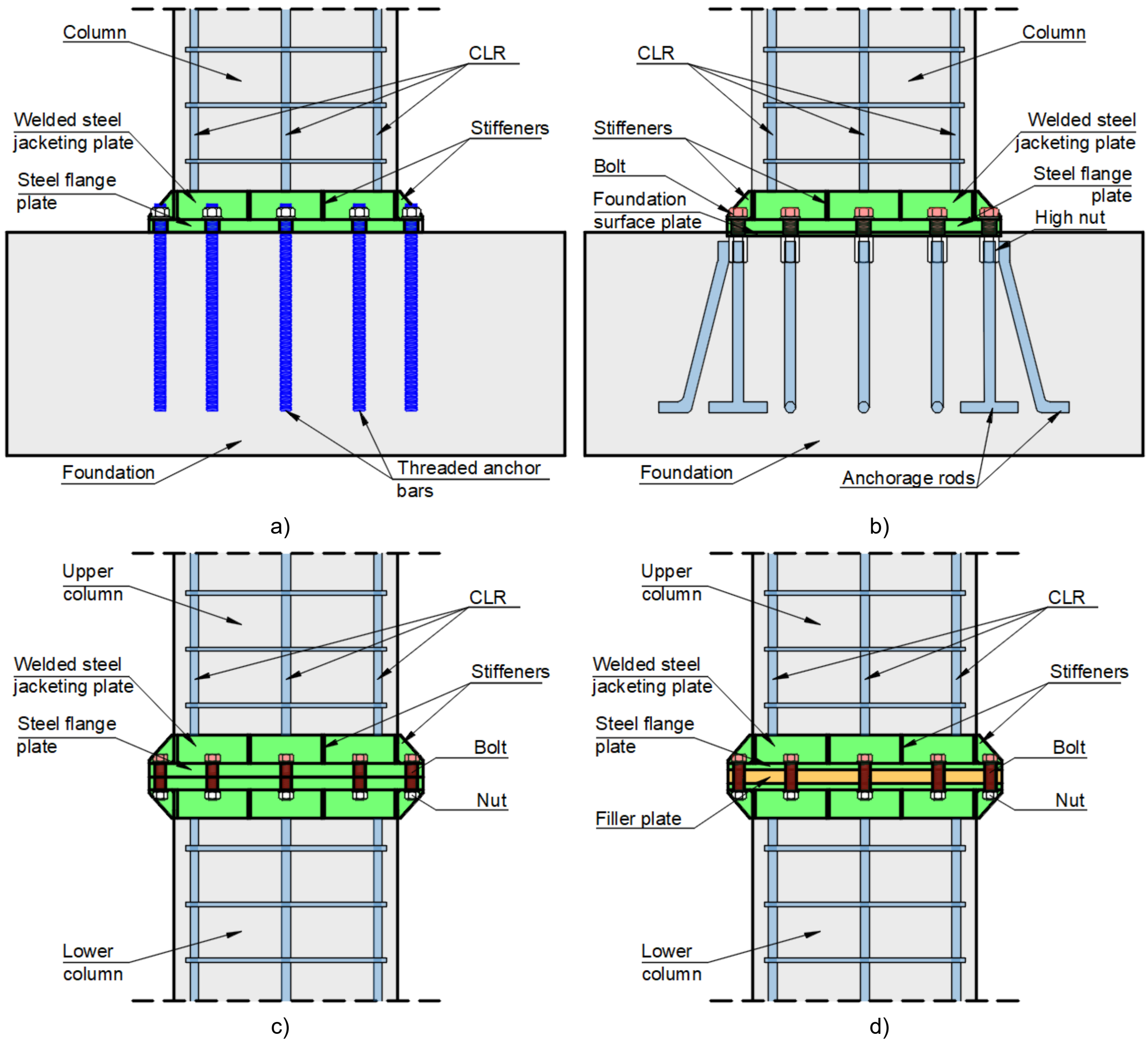


Figure 9. Typical configuration of steel flange plate connections: a) CF connection, b) CF connection adapted from Pul et al. [55], c) CC connection, d) CC connection with filler plate

head. Three types of connection with threading CLR end are also shown in Figure 10: threaded CLR with exposed nut (Type E), threaded CLR with nut installed in the pit (Type F) and threaded CLR secured with coupler and bolt (Type G).

In this section, different solutions for CF and CC connections presented in Figure 9 were discussed. The proposed solutions differ in the type of CLR and steel flange plate connection, as presented in Figure 10.

The welding procedure was commonly used in previous research [57]–[61]. Orlando and Piscitelli [58] had tested CF connection on cantilever columns subjected to monotonic and cyclic loads. In their study, the connection between the column longitudinal reinforcement (CLR) and the steel flange plate corresponded to Type A. The column cross-sectional dimensions were b_c/h_c = 400/400 mm, while the CLR was adopted as 8 Ø16. For the experimental setup, a steel beam (HE400B) was used instead of a reinforced concrete (RC) foundation. The steel flange plate was connected to the beam using eight M24 grade 10.9 bolts. The proposed connection corresponds to what is shown in the Figure 9 a).

The monotonic test results indicated ductile behaviour, characterized by significant plastic deformation beyond the elastic limit, with an ultimate drift ratio of approximately 5%. In cyclic tests, failure of the CLR was not observed, as the tests were terminated upon reaching the capacity of the loading equipment. The yielding force and maximum load of the CF connections under monotonic and cyclic loading occurred at similar drift ratios, with slightly higher drift capacity observed in the monotonic tests. The CF connection was intentionally overdesigned, resulting in cracking localized in the RC column.

Based on the experimental results, the authors concluded that the proposed CF connection satisfied capacity design requirements. Furthermore, they suggested that this type of connection could also be applied in column-to-column (CC) joints, where it is expected to behave as a full-strength connection. Although the possibility of demountability was not addressed in the study, the overdesigned configuration implies that failure would likely occur in the CLR after reaching the ultimate limit state, meaning that only the foundation could potentially be reused.

A direct welding solution was also proposed by Aktepe et al. [59] and Akduman et al. [60], in which a Type B connection between the column longitudinal reinforcement (CLR) and the steel flange plate was adopted. To enhance the force transfer mechanism between the reinforced concrete (RC) column and the steel components of the connection, anchorage rods (steel bars) were welded to the steel jacketing plate. The column dimensions were b_c/h_c = 250/150 mm with 6 Ø10 used as CLR. The connection between the steel end plate and the RC foundation was achieved using embedded threaded bars. After positioning the column on the foundation, high-strength nuts were tightened onto eight threaded rods with a diameter of 14 mm. This configuration corresponds to the connection detail shown in Figure 9 a).

Akduman et al. [60] compared behaviour of cantilever columns with monolithic CF connection and proposed demountable CF connection under cyclic load. The CF connection zone showed no damage during the testing. All deformation appeared in the end of the RC column with more concrete spalling and crushing compared to the monolithic specimen. The failure of demountable specimen was due to crushing confinement concrete zone and CLR buckling, while the failure of monolithic specimen was only due to the crushing of confinement concrete zone. Authors [60] demonstrated that demountable specimens had better performance than monolithic specimens, with higher drift capacity, greater energy dissipation and higher initial stiffness as well as maximum lateral load capacity.

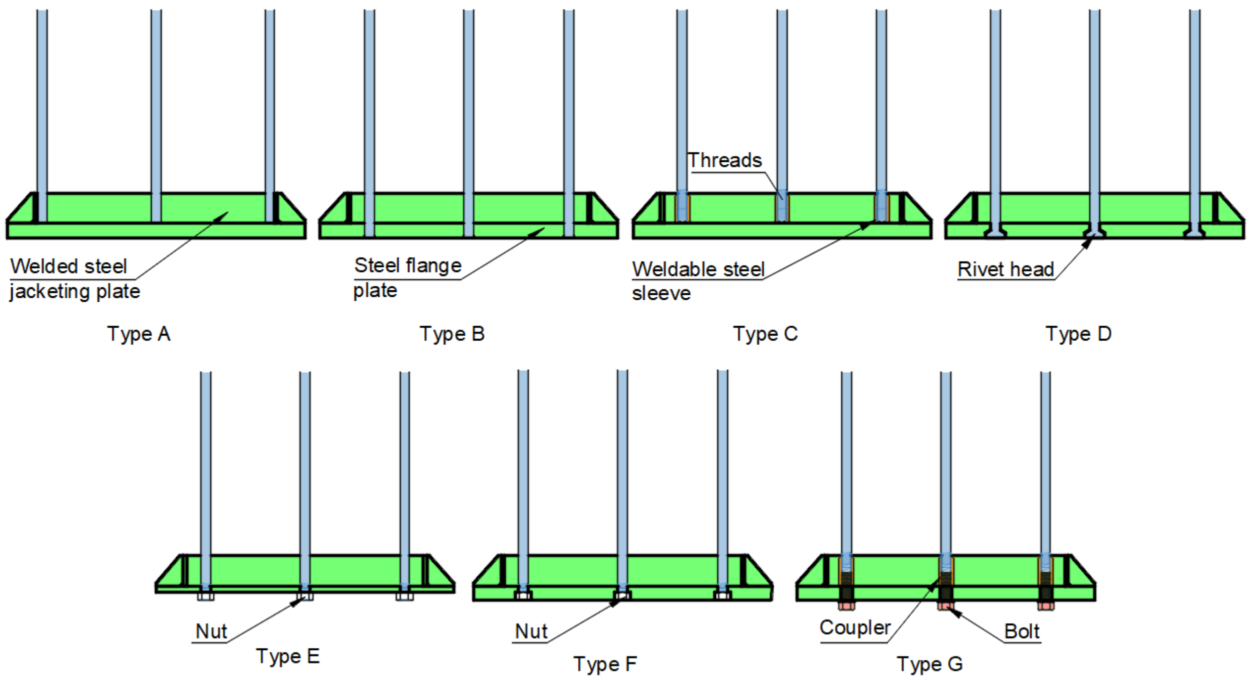


Figure 10. CLR-steel flange plate connections

In addition, the same authors proposed an alternative connection type [8]. They used RC flange instead of steel end plate. The RC flange was reinforced with bars, steel plates or combination of these two and castes together with the column. The column longitudinal reinforcement was bent 90 degrees and welded to the RC flange reinforcement or steel plates. During the RC flange casting holes for threaded rods were made. They observed that under cyclic load, CF connections with RC flange failed to achieve proper capacity design behaviour due to the damage that appeared in the RC flange.

Although the authors stated that the proposed connection is fully demountable, the disassembly procedure was not explicitly demonstrated. It can be concluded that for lower drift ratios the RC column and foundation could potentially be reused. However, at higher drift demands, significant damage of RC column would occur, leaving only RC foundation with anchor rod reusable. Usage of connection with steel plate was highly recommended for both beam and column applications in seismic active zones [59], [60].

Indirect welding of longitudinal reinforcement bar to the steel plate was proposed by Zhang et al. [57] and Liu et al. [61]. Both studies adopted type C concept presented in Figure 10, in which weldable steel sleeves were screwed to the column threaded rebar and then welded to steel plate. Zhang et al. [57] tested CC connection under eccentric compression, the layout of connection is schematically shown in Figure 9 c). The study considered variations with or without steel jacketing plate and stiffeners. In addition, different types of shear keys were used: shear connectors (steel beam) and shear studs, all welded to the steel flange plate. The column had square cross section with dimensions b_c/h_c = 250/250 mm, and CLR was four 16-mm-diameter bars. The connection detail at the end of upper and lower column were identical, and the CC connection was achieved using preloaded M20 high-strength bolts, with grade 10.9. For cast-in-place and demountable connected CC connections the failure occurred due to yielding of CLR on the tension side and concrete crushing on compression side. Under eccentric compression load demountable CC connections outperformed cast-in-place columns in terms of deformability, but cast-in-place columns performed better in terms of load bearing capability.

Liu et al. [61] tested both CC and CF connection in cantilever columns under cyclic load. The section size of all specimens was b_c/h_c = 400/400 mm, with different CLR. The cast-in-place column was reinforced with 12 Ø18, whereas the columns with CC connection had 12 bars with diameters of 18, 22, 28 mm. The column with CF connection had 12 Ø22 for CLR. The CC connection was completed with the utilization of high-strength bolts for connection between two steel end plates. Specifically, 16 M24 bolts were used for the demountable CC connections, while the CF connection used 20 M32 anchor bars embedded in the RC foundation. The threaded anchor bars were welded to the anchor plate embedded inside the foundation.

All specimens were tested under cyclic loading, and the performance of CC and CF connections was compared with that of monolithic columns and columns incorporating grouted sleeve CC connections. Failure of all specimens occurred due to flexural mechanisms. The crack propagation patterns were similar for monolithic columns and those with CC and CF connections. An increase in CLR ratio resulted in reduced crack spacing. The authors concluded that the location of the connection had a negligible influence on the development of flexural cracks.

Demountable specimens exhibited higher cumulative energy dissipation and greater resistance compared to the monolithic specimen. The highest ultimate drift ratio was observed in the proposed CC connection, while both the grouted sleeve and CF connections also achieved higher drift capacities than the cast-in-place column. The CC and CF connections demonstrated comparable performance in terms of strength, ductility, and energy dissipation. Following the tests, the demountability of the connections was verified. The column was successfully detached from the foundation, and subsequent removal of concrete revealed that the steel flange plate, welds, and connections between the CLR and steel sleeves remained intact. Based on these observations, the authors [61] concluded that the proposed connection system is reliable.

The innovative demountable CF connection was proposed by Pul et al. [55]. Instead of welding rebar to the steel plate CLR with rivet head was used (Type D in Figure 10). The steel end plate was manufactured with rivet head rod holes. Again, the steel jacketing plate and stiffeners were also used in proposed solution. At the upper surface of RC foundation, the foundation surface plate was installed, as presented in Figure 9 b). The high nuts were welded to the surface steel plate and rebar anchors were screwed into them. High nuts and anchor rods remain embedded into the RC foundation. The steel flange plate was fixed to the surface foundation plate with the M20 10.9 grade high-strength bolts. The CLR was ten 16-mm diameter bars, whereas the cross-section of columns was b_c/h_c = 400/250 mm. The monolithic and demountable cantilever columns were tested under cyclic load.

Cracking in the demountable specimens initiated at lower drift ratios compared to the monolithic specimen; however, yielding occurred earlier in the monolithic column. Both systems exhibited a flexural failure mode, characterized by concrete cover spalling and buckling of the CLR. Notably, the onset of concrete cover spalling was delayed in the demountable specimens. Pul et al. [55] concluded that the proposed CF connection exhibits behaviour comparable to monolithic systems in terms of energy dissipation, initial stiffness, ultimate drift ratio, and ductility. The design philosophy aims to maintain the steel plates and bolts within the elastic range, thereby concentrating damage within the RC column. Consequently, only the foundation remains suitable for reuse.

Nzabonimpa et al. [62], [63] and Hong et al. [64] developed new way of connecting longitudinal rebar to the steel plate. Their approach was to use steel end plate with holes for rebar. The CLR that was threaded at the end and it was fixed to the steel plate with nuts. Two different solutions were proposed the one with thin plate and the other with thick plate (Type E and F in Figure 10). The thick plate enables making the pits for the nuts and threaded rebar to fit into. The surface of the end steel plate remains flat, and position of rebar in upper and bottom column remains same. In contrast, the use of a thin plate requires the addition of a filler plate (steel or concrete) to accommodate the protruding nuts and to fill the gap between opposing steel plates. In this case, the nuts remain exposed on the plate surface. The connection between the two steel end plates is achieved using 18 M20 bolts. The configuration without a filler plate is illustrated in Figure 9 c), while the configuration with a filler plate is shown in Figure 9 d). Unlike other solutions, these studies [62]–[64] did not incorporate steel jacketing plates or stiffeners. The column dimensions were b_c/h_c = 500/500 mm and CLR 4 Ø25.

The cyclic behaviour of cantilever columns with CC connections using thin and thick plates was evaluated and compared with that of cast-in-place columns. For specimens with thick plates, failure occurred due to concrete cover spalling and buckling of the CLR, while the steel flange plate remained intact. In contrast, specimens with thin plates exhibited noticeable deformation of the steel plate. The results indicated that connections with sufficiently stiff and strong (i.e., thick) steel plates could achieve structural behaviour comparable to monolithic joints and effectively act as rigid connections with similar stiffness and deformability. However, specimens with thin plates demonstrated inadequate performance, including a significant reduction in strength and an inability to form rigid joints. Disassembly of the connections was successfully performed, confirming the feasibility of reusing both the upper and lower column segments.

The type G connection, presented in Figure 10, was adopted and experimentally tested by Zhou et al. [56]. Instead of welding, the CLR with threaded end was connected to the steel flange plate using a coupler-bolt assembly. The welding procedure was performed on steel jacketing plate and steel stiffeners. The overall configuration of the tested connection is similar to that shown in Figure 9 a). A total of 24 pre-stressed M24 grade 12.9 high-strength anchor bolts, together with an anchor plate, were embedded in the RC foundation. The square column section with dimensions b_c/h_c = 800/800 mm was reinforced with 32 bars of 20 mm diameter. The cantilever columns were tested under cyclic load. At the end of testing, longitudinal rebars buckling was observed with significant concrete crushing. Authors [56] concluded that CF connection had favourable flexural mode characterized by concrete cracking, yielding of the CLR, concrete cover spalling and crushing, and eventual fracture of the reinforcement. Although only a limited analysis of the results was presented, the authors concluded that the connection showed adequate flexural performance.

Following the tests, the steel flange plates were successfully demounted from the column. The steel components and bolted connections remained intact, demonstrating the feasibility of demountability for this connection type. Both the foundation and the steel elements of the connection were deemed reusable.

3.2 Connections with steel shoe embedded in column

In the literature, several solutions for column-to-foundation (CF) and column-to-column (CC) connections using column shoes have been presented [51], [52], [54], [65]. Column shoes are currently manufactured by various companies as prefabricated elements or manufactured by welding steel plates. The connection mechanism is based on load transfer from the CLR to the column shoe through lap splicing. Reinforcing or steel bars are pre-welded to the column shoe, and the CLR is lapped with these welded bars. A single column shoe may be designed to accommodate one or multiple CLR bars. All proposed solutions had two types of welded rebars: (1) Type 1 rebar used for overlapping with column longitudinal rebar, and (2) Type 2 rebar used for improving the column shoe-concrete connection and stress transfer [51], [52], [54], [65]. The CC and CF connections are completed after fixing column shoes for foundations or lower column. The embedded anchor bars or coupler-bolt assembly are used for connecting column shoe and foundation/lower column. The upper and lower nuts were used, lower one for adequate positioning of column and upper one for fixing the column shoe. Commonly the column shoes are either welded to the integral plate or used as standalone elements. The difference from steel flange plate connection solution is in use of grout. After erecting column on foundation or lower column, the gap between column shoe and concrete surface is poured with non-shrink grout. Contrary to the steel flange connections, the column cross-sectional dimensions remain unchanged at the connection zone. The characteristic example of column shoe connections is presented in Figure 11.

The cyclic response of the CF connections in cantilever columns was investigated by Nascimbene and Bianco [52]. In their study, four column shoes welded to an integral steel plate were used, as shown in Figure 11. The protruding anchoring bolts anchored into the foundation were used for connecting the column shoes to foundation. Three diameters of anchor bolts and corresponding column shoe were used M24, M30 and M39. The CLR was 12 Ø 16 and 4 Ø 20, and column cross-section b_c/h_c = 400/400 mm. For shear resistance, they added steel pins, which were embedded half in foundation half in grout.

Based on the experimental results, the authors concluded that the behaviour of the anchor bolts governs the overall response of the CF connection. The anchor bolts were the only components that exhibited nonlinear behaviour, while no significant damage was observed in other parts of the specimens. The connections demonstrated sufficient strength and ductility to resist seismic actions. Although reusability was not explicitly addressed, the minimal damage to the column suggests that reuse may be feasible. The authors also recommended a more rational design of the column, because the CLR did not reach the yielding point during testing.

Similar connection set-up, without an integral plate was tested by Wang et al. [65]. Researchers tested seismic performance of both CF and CC connections and compared results to the cast in situ column. All columns had CLR as four 18 mm- diameter bars, and b_c/h_c = 300/300 mm cross-section dimension. The column shoes were produced by welding steel plates. The M22 grade 10.9 studs threaded at both ends were used for anchoring. For the CC connections the studs were anchored in the lower column, with the lower threaded end of the stud connected to the CLR using rigid coupling sleeves. When it is about the CF connections the same principle is applied, instead of the column longitudinal rebar bent bar were used. To increase shear capacity of connection shear keys were added and one stirrup between two precast elements. Only one sample without shear keys and additional stirrup was tested.

The failure point was defined as detaching the column shoe from the concrete. The CF connection specimens exhibited the lowest lateral load at first cracking, while the CC connections showed minor damage. After testing cantilever columns under cyclic load, author suggested usage of these connection only in the region of minimal bending moment. They reported poor seismic performances of the CF connections compared to the cast in situ columns. The weak bond between the grout and the column shoe was identified as the critical issue, leading to early cracking, pinched hysteretic response, and reduced energy dissipation capacity. The authors [65] also concluded that shear keys are not necessary when the connection is located in the central region of the column.

Dal Lago et al. [54] tested different connection types under cyclic load among them the connection with column shoes, similar to the one shown in Figure 11. All columns had a square cross section of 400 mm sides, and CLR as 8 Ø16.

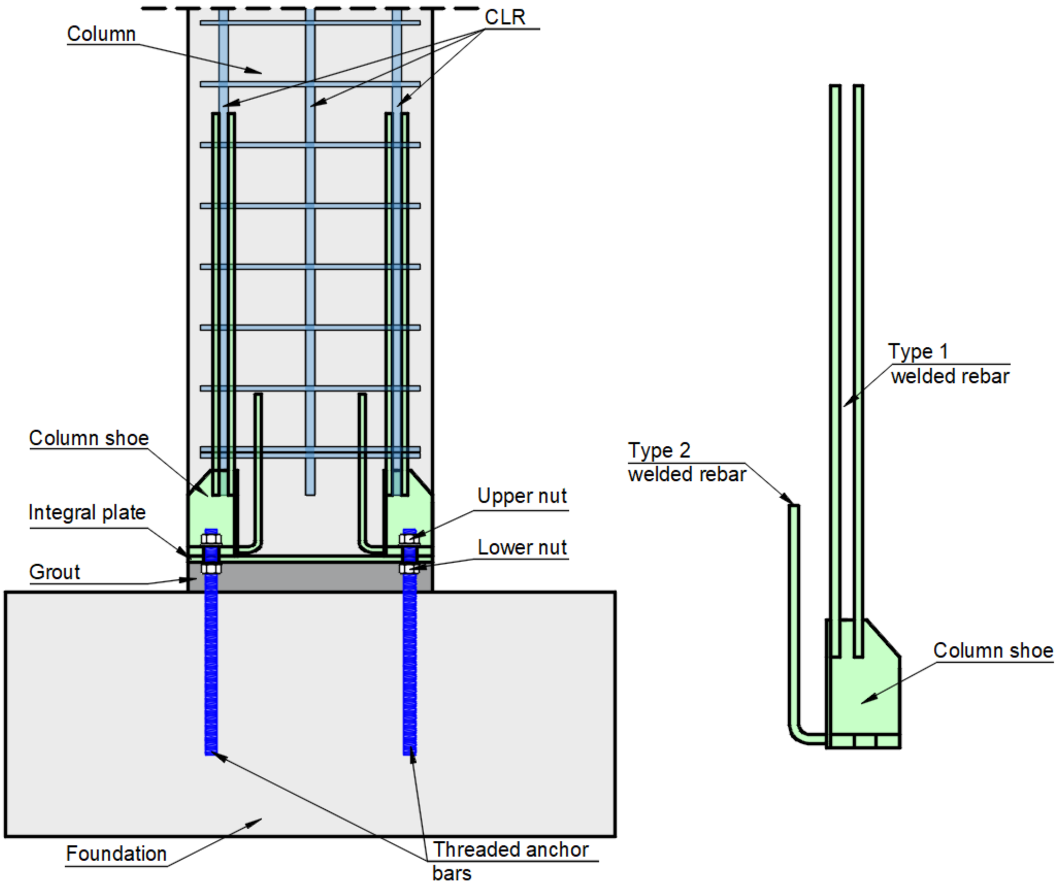


Figure 11. The CF connection with the column shoe

Threaded anchors were used for fixing column to the foundation. The bar anchor was threaded only on the part that is protruding from the foundation. After testing column cantilever samples under cyclic load, a good overall ductility of the specimen is shown by the envelope curve that should correspond to the monotonic behaviour. The connection presents the combination of rocking model and forming of plastic hinges. The failure occured due to the exceeding the capacity of anchors. The pinching effect that occurred significantly reduced seismic response characteristics of connections. However, the authors suggested that over dimensioning of anchors and overlapping of column longitudinal rebar should be more carefully taken into account in the capacity design.

The CF connection with column shoes and unbonded prestressing tendons was tested by Sripongngam et al. [51]. The column shoes were used for transferring forces from column to foundation, corresponding to the solution showed in Figure 11. Unbonded prestressed tendons are installed along the column and extended into the foundation to provide rocking behaviour. The dimensions of cross-section of column were b_c/h_c = 320/320 mm, and the adopted CLR was 8 bars with diameter of 16 mm. Authors [51] used energy-dissipating (ED) bolts (DB16, DB20, and DB25) to connect the column shoes to the foundation. These bolts are designed to enhance seismic performance by dissipating energy under cyclic loading. Columns with column shoes and cast-in-place column were tested and their responses were compared.

For the specimens with the demountable connection, the cracks were localized around the joint region, while the cast-in-place column displayed cracks throughout the upper and lower column region. The significant concrete spalling was considered as failure point during the testing. Overall, the demountable system exhibited reduced flexural damage compared to the cast-in-place column. However, the cast-in-place specimen showed higher ductility and greater overall energy dissipation capacity. The use of ED bolts led to concentration of inelastic deformations within the bolts, allowing the RC column to remain largely elastic. The authors classified this system as a low-damage solution, demonstrating stable cyclic behaviour with only minor reductions in strength and stiffness compared to the monolithic specimen. It was also observed that the performance of the system was sensitive to the level of overdesign of the connection. An appropriate overdesign ratio is necessary to achieve the desired low-damage behaviour. Higher overdesign ratios result in behaviour similar to cast-in-place systems with greater damage of columns, while lower ratios lead to reduced energy dissipation capacity with lower damage of columns.

Orlando and Piscitelli [58] tested a connection system conceptually similar to column shoe connections based on lap splicing. The column dimensions and connection to the steel foundation beam were the same as in case of steel flange plate connection. Instead of separated column shoes, they used single flanged unions, where the bars welded to steel flange were overlapping with column longitudinal rebar.

They concluded that overlapping gives higher moment resistance of column cross section compared to the continuous bars. Therefore, the plastic hinge can be formed in the section that is outside the connection, which should be taken into account. Compared to the connection type with directly welded column longitudinal bars to the steel end plate, this connection type showed lower dissipation capacity, but higher strength.

Another connection type that can be classified within this group was proposed by Quing et al. [66], [67]. The authors used a system similar to column shoes, using a steel box; however, instead of a lap-splice mechanism, force transfer from the CLR was achieved by threading the bar ends and screwing them into the steel box. The steel box was anchored with bent high-strength rebar embedded into the foundation. The upper and lower reinforcing bars had the same diameter, but the lower bars were of higher strength and were designed to remain elastic during testing, in order to prevent local plastic deformations and potential loosening of the threaded connection. The shear keys were also adopted, as presented in Figure 12. In the first study, the authors [66] compared behaviour of the cantilever precast specimens with monolithic specimens under cyclic loads. The tested columns had a square cross-section of 400 mm, with CLR consisting of eight bars of 25 mm diameter.

The monolithic specimens exhibited concrete cracking and spalling over a relatively shorter region compared to the columns with the proposed CF connection. In contrast, columns with steel boxes showed a greater number of cracks and a more extensive damage zone. Inelastic deformations in these specimens were concentrated above the top of the steel box, resulting in a longer plastic hinge region compared to the monolithic columns. Failure occurred in flexure, accompanied by rupture of the CLR. The demountable specimens exhibited higher lateral load capacity but relatively lower deformation capacity compared to the monolithic specimens. Additionally, their energy dissipation capacity was lower.

In a subsequent study, the authors [67] investigated cantilever columns with different mechanical connection devices, including grouted steel sleeve systems, steel box connections, and hybrid configurations comprised of these two solutions. Among these, only the steel box connections demonstrated potential for reuse. The steel box specimens exhibited strength and stiffness degradation comparable to those with grouted sleeve connections, while the hybrid systems showed improved overall performance but lacked reusability. The author [66], [67] did not discuss demountability process or reuse of elements. Although the demountability process and reuse were not explicitly examined, it can be inferred that, after reaching the ultimate limit state, only the foundation and the lower reinforcement—remaining in the elastic range—would be suitable for reuse.

3.3 Connections with column steel jacket

The steel jacket connections represent a hybrid solution that combines characteristics of RC columns and concrete-filled tube (CFT) columns [68], [69]. The lower portion of the column is manufactured as a CFT element, while the remaining height consists of a conventional precast RC column. The CF and CC connections are placed at the CFT end of the column, where the steel tube provides a robust interface for load transfer. Additional external steel plates are connected either to the CFT ends of adjacent columns or between the CFT end and the foundation. The CLR is bent and welded to the steel part of CFT end of the column, ensuring continuity of force transfer between the concrete and steel elements.

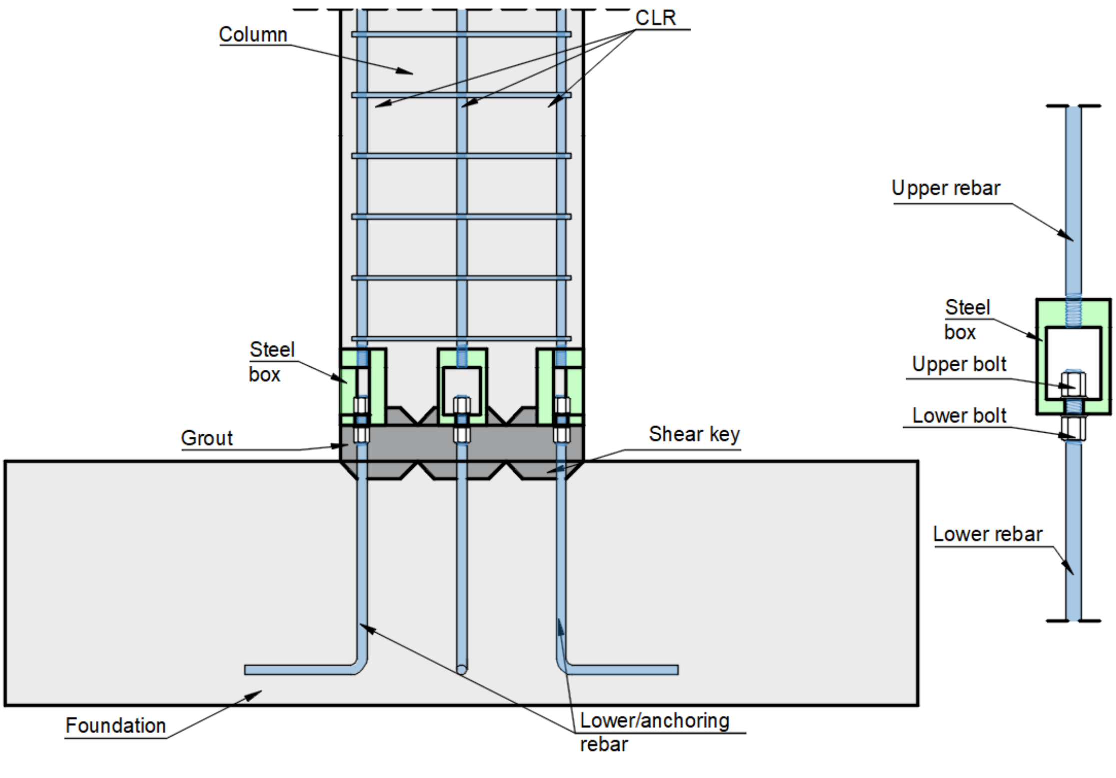


Figure 12. The CF connection with the steel box adapted from Quing et al. [67]

Guo et al. [68] proposed the replaceable solution of CF connection, which is illustrated in Figure 13. The column consisted of upper RC part and lower CFT part. The connection assembly included the CFT column part, embedded anchor bolts, the steel fuses (plates with slits), the buckling restraints and the padding blocks. The CFT part of column had two cross sections, with the narrower end. The gap between the steel fuse and narrow CFT cross section was filled with padding blocks. The steel fuse was connected to the embedded anchor bolts in CFT part of column. The two preinstalled bolts in the padding block were used to fix the buckling restraints against the steel fuses. Connection between the foundation and column was provided by connecting the steel fuses to the embedded anchor bolts in foundation.

The experimental investigation of cantilever column behaviour under cyclic load was performed. The cross-section of RC column part was b_c/h_c = 400/400 mm, while the narrow CFT column part had dimensions b_c/h_c = 240/240 mm. Twenty-four high strength anchor bolts with 24 mm in diameter were used to connect the steel fuses and CFT column and twenty embedded anchor bolts with 30 mm in diameter were used to connect the steel fuses and the foundation. Authors [68] varied padding block material and tested reparability of connection. Each specimen was tested twice: first to a drift ratio of 2%, and then, after disassembly and replacement of the steel fuses and padding blocks, to a drift ratio of 4%. From the experimental results, it was concluded that the seismic response of the specimen was mainly characterized by slipping and plastic deformations of steel fuses, followed by buckling at larger drift ratios. The rest of the connection elements remained unaffected. Up to the 2% drift ratio, the steel fuses deformed, but the disassembly procedure was performed without obstacles. The main conclusion of experimental testing was that the request of reparability was satisfied, and that the behaviour of the samples in the first and second testing was the same. Generally, in terms of seismic performance, all specimens showed adequate behaviour.

The comparable connection with steel jacket was developed by Yuan et al. [70]. They proposed the CC connection that is formed of two RC columns placed together into so-called the steel plate hoop. In this system, the CLR from the two columns is not directly connected; instead, the bars are bent at the column ends. During fabrication, holes are formed at the column ends, allowing bolts to pass through both the concrete sections and the surrounding steel hoop. The bolts are then secured with nuts, as illustrated in Figure 14 a). In this way two columns are connected without welding the CLR to the steel elements of connection or embedding steel elements into the concrete elements.

The seismic performance of the proposed system was evaluated through cyclic testing of cantilever columns and compared with cast-in-situ specimens. In addition, the length and the thickness of steel plate hoop were varied. The column cross-sectional dimensions were b_c/h_c = 400/400 mm, while the CLR were adopted as 8 Ø22. The embedded bolts were M22 with the steel category HRB400. The results showed that specimens with thicker and longer steel plate hoops did not exhibit tearing or buckling of the steel jacket during testing. Only minor concrete cover spalling was observed at the column base, and upon removal of the steel hoop, the internal concrete remained in good condition. In contrast, specimens with thinner steel hoops developed dense cracking at the column ends, accompanied by localized crushing of the cover concrete in the compression zone. Overall, the prefabricated columns demonstrated slightly higher strength and comparable stiffness relative to cast-in-situ columns. The proposed CC connection exhibited behaviour similar to monolithic construction, indicating its potential applicability in precast concrete frame systems in seismic regions.

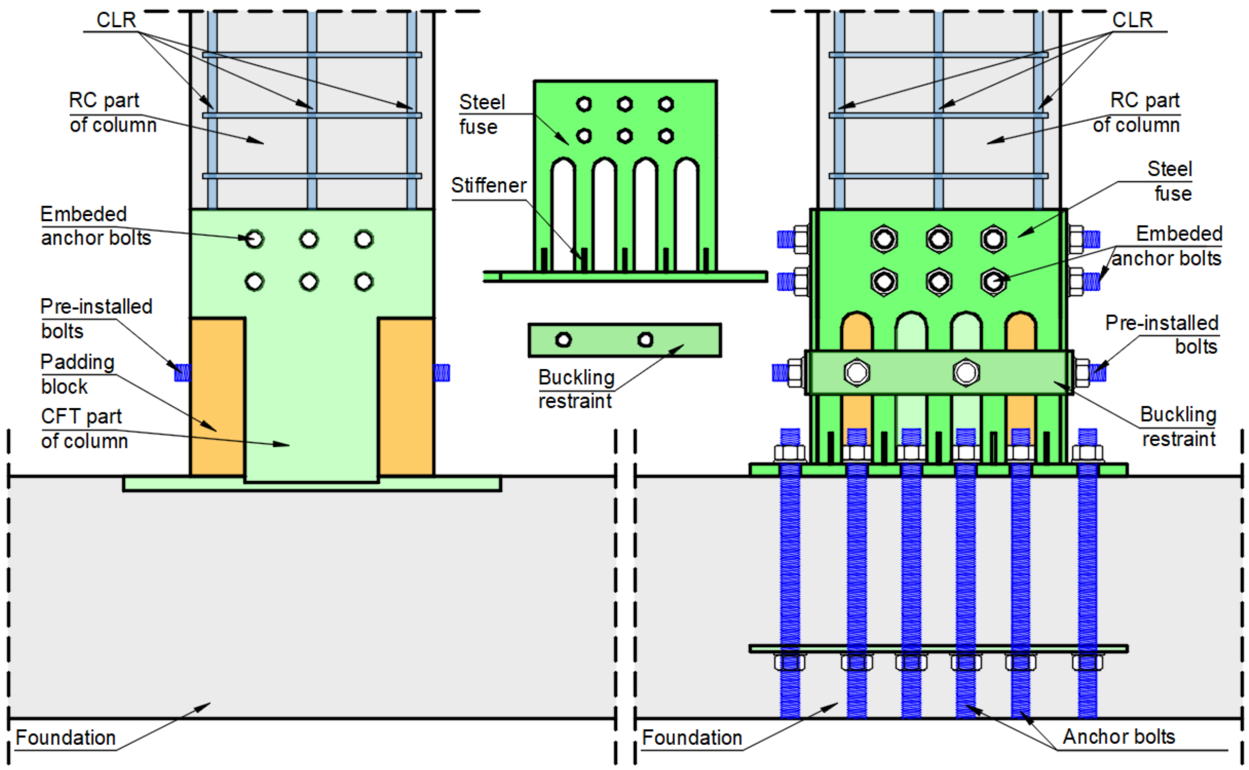


Figure 13. The configuration of the CF connection adapted from Guo et al. [68]

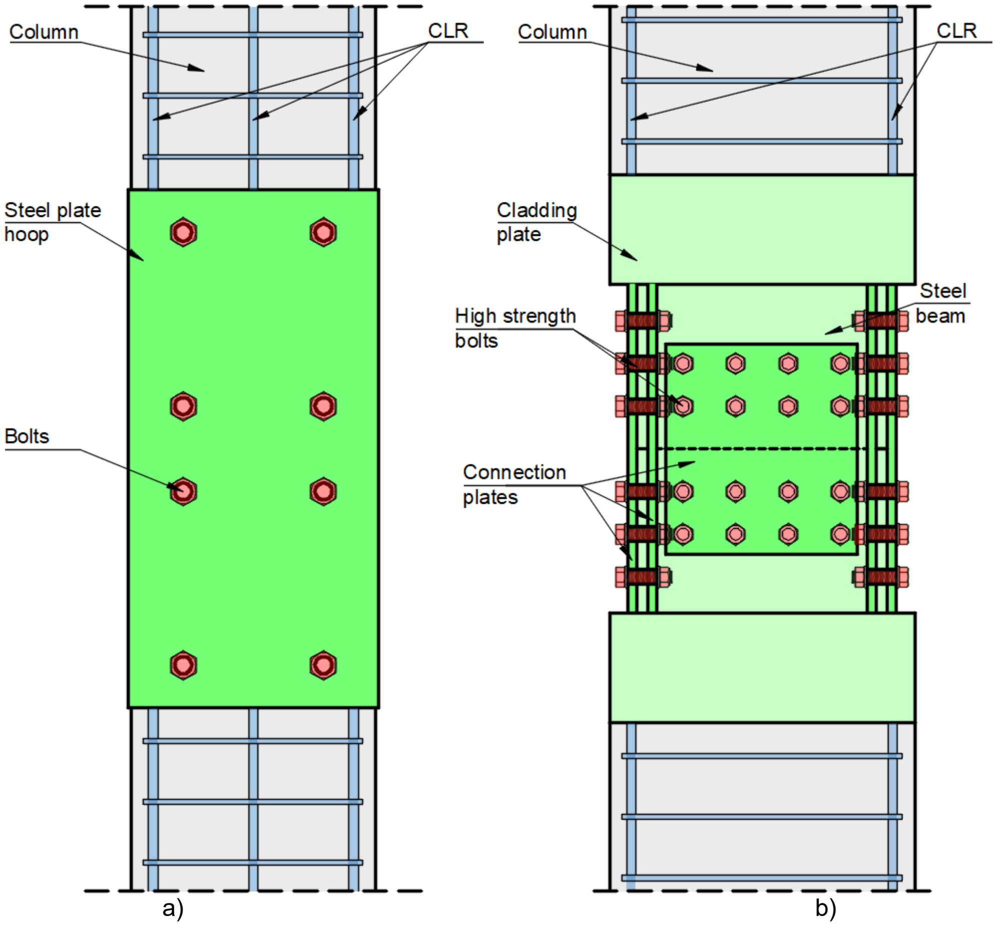


Figure 14. The configuration of the CC connection: a) adapted from Yuan et al. [70] and b) adapted from Zhan et al. [69]

The disassembly process was performed for the columns with thicker steel hoop plate, but the possibility of reuse was not discussed. It can be concluded that with the proper design of the steel hoop plate all connection elements could possibly be reused.

Zhan et al. [69] proposed similar solution for the CC connections, which is presented in Figure 14 b). The CFT part was connected to the RC part of the column with the studs, welded to the inner side of the cladding plate. In addition, CLR is welded to the interior of cladding plate. An I-shaped steel beam is also welded to the cladding plate, and the connection between two columns is achieved by joining the steel beams at the ends of the CFT segments using external steel plates, bolts, and nuts. The dimensions of column cross section were b_c/h_c = 500/500 mm, while the CLR were adopted as 4 Ø25.

The authors conducted only a finite element (FE) analysis to evaluate the behaviour of the proposed connection in comparison with a monolithic column under both monotonic and cyclic loading. The results indicated that the overall behaviour of the demountable connection is comparable to that of the monolithic system. At lower drift ratios, both systems exhibited similar responses, while at higher drift levels, the columns with CC connections showed slightly reduced strength. However, the ductility coefficient was higher for the columns with CC connections, and the yielding point remained similar for both samples. The demountability of the proposed connection was not addressed in the study. Furthermore, in the absence of experimental validation, it is difficult to discuss the level of damage of individual components, and the potential for reuse of the connection elements.

4 Discussion

4.1 Beam-to-column connections

In this section, different solutions for demountable precast beam-to-column joints were presented and discussed in terms of their strength, stiffness and ductility under lateral loads as well as their potential for disassembly, replacement and reuse after being subjected to the serviceability or ultimate load conditions. In general, the connections can be formed in two ways: (1) by welding or bolting reinforced bars, plates or steel embedments, with dry-packing or local grouting, or (2) by utilizing the unbonded post-tensioned tendons or rods.

It was demonstrated that the assumptions regarding the location of the connection (placed within or outside the critical region of the beam) and design of the connection (overdesigned or designed for ductility) can have major impact not only on the overall mechanical performance of the beam-to-column joints, but also on the extent of replacement or reuse possibilities.

A general, qualitative comparison of performance indicators of precast beam-to-column joints in respect to the monolithic joints or wet precast joints, is presented in Table 1. It can be seen that the precast joints with RC corbels and connections with steel boxes, with the connectors located within beam depth, have similar or higher bending resistance compared to the monolithic joints. However, the initial stiffness, displacement ductility and energy dissipation are either similar or lower, which may require additional elements for ensuring lateral stiffness of the building structure, such as steel braces or RC walls. The performance indicators of connections with steel end plates or angles are generally better, since the through anchor bolts were placed outside beam's depth. Connections with dampers and fuses have good performance, particularly in terms of displacement ductility and energy dissipation capacity. Precast joints with semi-dry connections have similar performance as monolithic joints. Finally, joints with post-tensioned connections and dissipaters also have similar performance as monolithic joints, except the energy dissipation capacity. However, these joints have large self-centering capabilities with small residual deformations.

Table 1. Qualitative comparison of performance indicators of precast beam-to-column joints in respect to monolithic or wet precast beam-to-column joints

Precast connection type / Study	Connection category	Bending resistance	Initial stiffness (Type of joint rigidity)	Displacement ductility	Energy dissipation
Connections with RC corbels [17], [19], [20], [21], [22]	Ductile	Similar or higher	Lower (semi-rigid)	Similar or lower	Similar or lower
Connections with steel boxes [24], [25], [26]	Ductile or overdesigned	Similar or higher	Similar or lower (semi-rigid)	Similar or lower	Similar or lower
Connections with end plates or steel angles [28], [29], [34], [36], [37]	Overdesigned	Similar	Similar (rigid)	Similar or higher	Similar or higher
Connections with dampers and fuses [38], [39], [41], [40], [42], [43]	Ductile	Similar or higher	Similar or higher (rigid)	Higher	Higher
Connections with partial in-situ concreting [45], [46], [47]	Placed outside critical region of beam	Similar	Similar or higher (rigid)	Similar	Similar
Post-tensioned connections with dissipating elements [49], [50]	Ductile	Similar	Similar (rigid)	Similar	Similar or lower

Possibilities of disassembly, replacement and reuse for each connection type are presented in Table 2. In all cases, the disassembly of the connections was considered possible or explicitly demonstrated by authors, albeit the difficulty and extent of the necessary works varied between the solutions. As stated earlier, the replacement and reuse possibilities strongly depend on the design assumptions and the location on the connection, i.e., the location where the major damage occurs. Since the "strong column-weak beam" concept was employed in almost all cases, with small to none damage occurring in columns, the majority of the proposed solutions could potentially provide at least the reuse of precast columns. The exceptions are joints with corbels or protruding beams which experienced high extent of the damage during testing. In case of connections with dampers and fuses, as well as the post-tensioned connections, which are designed to ensure practically elastic behaviour of precast beams and columns, it seems that both beams and columns have great potential for reuse.

It should be noted, however, that in the research presented in this section, the possibilities for reuse of precast beam-to-column joints are mainly commented and examined after the failure of the tested specimens was reached. In other words, these situations mainly correspond to the repairment of structure after strong earthquakes. It is a completely different question if the structural elements can be disassembled, relocated and reused after frequent earthquakes or at the end-of-life scenario, with the similar behaviour as the original structure. In those circumstances, the level of damage to the structural elements is expected to be much lower, and possibility of reuse of both beams and columns is expected to be higher. Further research is needed to address these aspects of the replacement and reuse.

Finally, the presented solutions for demountable precast beam-to-column joints vary in their complexity, the construction time, difficulty of disassembly and the amount of steel components, which ultimately prevails in the decision-making process regarding the use of specific connection type.

Table 2. Qualitative presentation of disassembly, replacement and reuse potential possibilities for precast beam-to-column joints

Precast connection type (Study)	Major damage	Disassembly	Replacement	Reuse
Connections with RC corbels [17], [20], [21], [22]	Connection and/or beam	Yes, relatively easy	Connection and/or beam	Column or none
Connections with steel boxes [24], [25], [26]	Boxes, bolts and/or beam	Yes, relatively easy	Connection and/or beam	Column
Connections with end plates or steel angles [28], [29], [34], [35], [36], [37]	Beam	Yes, relatively easy	Beam and connection	Column
Connections with dampers and fuses [38], [39], [40], [41], [42], [43]	Connection	Yes, relatively easy	Connection	Column and beam
Connections with partial in-situ concreting [44], [45], [46], [47]	Protruding beam from the column	Yes, requires hammering and cutting	Connection	Beam
Post-tensioned connections with dissipating elements [49], [50]	Dissipating elements	Yes, can require cutting	Dissipating elements	Column and beam

4.2 Column-to-foundation and column-to-column connections

The reviewed studies demonstrate significant progress in the development of demountable CF and CC connections for precast reinforced concrete structures. A wide range of solutions has been investigated, primarily distinguished by the type of mechanical device used to transfer forces, including steel flange plates, column shoes, and steel jacket systems.

Table 3 summarizes the performance indicators of the previously reviewed connection types. The analysis includes only studies in which the behaviour of precast demountable connections was compared with that of equivalent monolithic specimens. Despite differences in detailing, most proposed connections aim to achieve structural performance comparable to monolithic systems while enabling partial or full demountability.

Across all connection types, experimental and numerical investigations consistently indicate that properly designed demountable connections can satisfy strength, stiffness, and ductility requirements. Flexural failure mechanisms dominated, characterized by concrete cracking, reinforcement yielding, spalling, and, in some cases, bar buckling or fracture. This confirms that capacity design principles can be achieved, with inelastic deformations concentrated in ductile regions.

However, the location and distribution of damage vary depending on the connection concept. Steel flange plate systems often rely on overdesign to ensure that damage occurs in the column rather than in the connection, resulting in stable and predictable behaviour but limited reusability of the column. The column shoe connections seismic performance strongly depends on the behaviour of anchor elements and the bond between grout and steel components. Capacity design requirements are generally satisfied, with damage concentrated either in the column or in designated ductile components such as anchor bolts or energy-dissipating devices. Steel jacket systems, particularly those incorporating replaceable components such as steel fuses or energy-dissipating bolts, allow better control over the location of inelastic deformations.

The qualitative presentation of demountable CC and CF connection in terms of reusability is also shown in table 3. Among the investigated systems, only a limited number of studies explicitly validated demountability through experimental disassembly and reassembly procedures [29], [56], [61]–[63]. The systems presented in these studies allowed damaged components to be removed and replaced after seismic loading, while the primary structural elements (e.g., foundation, steel plates, or even column segments) remained mostly intact. In contrast, many solutions rely on overdesign principles, which ensure that the connection remains elastic while damage is concentrated in the RC column. In such cases, reuse is typically limited to the foundation and embedded steel components, as the column itself experiences irreversible damage.

In terms of demountability and reuse, solutions proposed by Guo et al. [68] incorporating replaceable components show clear advantages. The steel fuses solution enables repair and reuse of key structural parts, while other systems indicate potential but lack experimental verification; consequently, reuse is typically limited to steel components and foundations.

Table 3. Qualitative comparison of performance indicators of precast CC and CF joints in respect to monolithic joints and qualitative presentation of disassembly and reuse potential possibilities for CF and CC joints

Study	Connection type	Mechanical device	Bending resistance	Initial stiffness	Displacement ductility	Energy dissipation	Major damage	Disassembly	Reuse
[59], [60]	CF	Steel flange plate/welding	Higher or similar	Higher or similar	Similar	Higher or similar	Column/buckling of CLR	Not performed/ potentionaly relatively easy	Foundation
[61]	CF	Steel flange plate/welding	Similar	Similar	Higher	Higher	Column	Performed without obstacles	Foundation
[61]	CC	Steel flange plate/welding	Higher or similar	Similar	Higher	Higher	Column	Performed without obstacles	Lower column
[55]	CF	Steel flange plate/rivet head CLR	Similar	Similar	Similar	Similar	Column/buckling of CLR	Not performed/ potentionaly relatively easy	Foundation
[62], [63], [64]	CC	Steel flange plate/threaded CLR	Similar	Similar	Similar	Similar	Column/buckling of CLR/steel plate deformation	Performed without obstacles	Upper and lower column
[65]	CF	Column shoe	Similar	Similar	Similar	Similar	Column/bond between grout and column shoe	Not performed/ potentionaly relatively easy	Foundation
[65]	CC	Column shoe	Similar or higher	Similar or higher	Similar or higher	Similar or higher	Column/bond between grout and column shoe	Not performed/ potentionaly relatively easy	Upper column
[54]	CF	Column shoe	Similar	Similar	Similar	Similar	Failure of anchor bolt	Not performed/ potentionaly relatively easy	Foundation
[51]	CF	Column shoe	Similar or lower	Similar or lower	Similar or lower	Similar or lower	Column	Not performed/ potentionaly relatively easy	Foundation
[66], [67]	CF	Column shoe/steel box	Similar	Similar	Similar	Similar	Column	Not performed/ potentionaly relatively easy	Foundation
[70]	CC	Steel jacket	Similar	Similar	Similar	Similar	Column	Performed without obstacles	Upper and lower column

5 Conclusions

The reuse of concrete elements has gained considerable attention in recent years as a promising approach for improving the sustainability of the concrete construction industry. Although comprehensive experimental and numerical investigations of the behaviour of demountable connections between precast frame elements has been conducted, the reusability of elements and their connections remains limited for many connection types. This lack of detailed knowledge represents one of the major barriers to the wider practical application of structural concrete reuse. Therefore, the authors aimed to address this challenge by reviewing and analysing the current state of the art for demountable connections in precast frame systems.

Based on the analysis of available experimental and numerical research of precast beam-to-column joints and frames, which is mainly focused on the performance under cyclic (seismic) loads, the following conclusions regarding precast beam-to-column connections can be drawn:

- Most of the presented connections were realised with bolts or threaded rods, couplers (threaded sleeves), and steel elements such as plates, boxes, angles, I-sections, etc. Other connections include welding of some components (mainly longitudinal reinforcement or threaded rods), partial in-situ concreting or prestressing via post-tensioned strands.

– The behaviour of precast beam-to-column joints depends strongly on the adopted design approach and on the location of the connection relative to the beam-to-column interface. Although adequate strength can be achieved in all the moment-resisting connections presented, other seismic performance indicators, such as stiffness, ductility and energy dissipation, vary across the solutions. In this regard, at present, the most promising solutions seem to be connections with steel end plates or angles (with connectors placed outside the beam's depth), partial in-situ concreting, and connections with steel dampers and fuses.

– All analysed connection solutions can be demounted. Apart from bolted connections, this also applies to connections that involve welding of some components and partial in-situ concreting. In the latter cases, although hammering and cutting were employed, some authors have stated that the mechanical removal process was not too difficult and time-consuming.

– It was demonstrated that the replacement and reuse possibilities also depend on the adopted design approach and the location of the connection, i.e., the location of the major damage under imposed load (monotonic or cyclic). The majority of the analysed connection solution could provide the reusability of precast columns. The exceptions are precast joints with protruding beams (long corbels) extending from precast column, which experienced damage during testing. However, in those cases, it was demonstrated by some authors that the reuse of precast beams is possible with new columns. The reuse of both beams and columns seems to be possible for the connection solutions that employ dissipating connections, which mitigate damage from beams and columns, such as connections steel dampers, fuses and dissipaters.

– For practical implementation, the reuse possibilities of precast beams and columns, as well as their connections, should be carefully examined and verified. The scenarios at the end-of-life of the building or after frequent (low intensity) earthquakes might contribute to these possibilities. Finally, the complexity, the construction time, difficulty of disassembly and the amount of steel components used for the connection, can ultimately prevail in the decision-making process regarding the use of specific connection type for precast frames.

From the literature review of demountable column-to-foundation (CF) and column-to-column (CC) connections, the following conclusions can be drawn:

– Demountable connections are primarily realised with steel-based mechanical devices, such as steel plates, bolts, threaded bars, and couplers, which enable force transfer and potential disassembly.

– The CF and CC connections are predominantly investigated under cyclic loading conditions, with focus on capacity design behaviour and comparison to monolithic reinforced concrete specimens

– Overall, demountable CF and CC connections represent a viable alternative to traditional monolithic construction, offering comparable structural performance with added benefits in terms of adaptability and potential reuse.

– Achieving a fully reusable system requires careful balance between capacity design, damage control, and connection detailing. While significant progress has been made, further experimental validation and design optimisation are needed to fully realise the potential of demountable precast concrete systems

– Experimental validation of demountability is still scarce, particularly regarding repeated assembly–disassembly cycles and long-term performance.

– The majority of studies focus on individual components or simplified configurations, with limited investigation of system-level behaviour in realistic structural frameworks.

Acknowledgements

The authors would like to express their gratitude to the CircBoost – Boosting Circular Economy in Construction Industry, funded by the European Union under the Horizon Europe research and innovation programme (Grant Agreement No. 101082068) and Ministry for Education, Science, and Technology, Republic of Serbia (Grant number 200092).

CRediT authorship contribution statement

Milica Vidović: Conceptualization, Methodology, Data curation, Visualization, Writing - original draft, Writing - review & editing.

Ivan Milićević: Conceptualization, Methodology, Data curation, Visualization, Writing - original draft, Writing - review & editing.

Jelena Carević: Visualization, Writing - review & editing.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

References

[1] M. Stamenić, "Mix design of concrete incorporating fine recycled concrete aggregates – a review," Build. Mater. Struct., vol. 68, no. 3, p. 2500008S, 2025.

[2] A. Radović, V. Carević, and S. Marinković, "Impact of the water-curing time on the carbonation initiation period of high-volume limestone powder concrete," Build. Mater. Struct., vol. 68, no. 2, p. 2500004R, 2025, doi: 10.5937/grmk2500004r.

[3] J. Dragaš, S. Marinković, V. Radonjanin "Prediction models for high-volume fly ash concrete practical application: mechanical properties and experimental database," Build. Mater. Struct., vol. 64, no. 1, pp. 19–43, 2021.

[4] A. Favier, C. De Wolf, K. Scrivener, and G. Habert, "A sustainable future for the European Cement and Concrete Industry Technology assessment for full decarbonisation of the industry by 2050," 2018, doi.org/10.3929/ethz-a-010025751

[5] J. Devènes, M. Bastien-Masse, and C. Fivet, "Reusability assessment of reinforced concrete components prior to deconstruction from obsolete buildings," *J. Build. Eng.*, vol. 84, no. June 2023, p. 108584, 2024, doi: 10.1016/j.jobe.2024.108584.

[6] I. Bertin, M. Saadé, R. Le Roy, J. M. Jaeger, and A. Feraille, "Environmental impacts of Design for Reuse practices in the building sector," *J. Clean. Prod.*, vol. 349, no. February, 2022, doi: 10.1016/j.jclepro.2022.131228.

[7] Z. Al-Faesly, "Feasibility of Reuse in the Concrete Industry," University of Ottawa, 2021.

[8] J. Carević, I. Milićević, M. Vidović, "Demountable connections for structural concrete reusability - State of the art and future directions for reinforced concrete, Part I: slabs" Build. Mater. Struct., (under review).

[9] T. M. Fayyad and A. F. Abdalqader, "Demountable reinforced concrete structures : A review and future directions," *Civ. Eng. Res. Irel. 2020 Demountable*, pp. 218–222, 2020.

[10] fib, *fib bulletin 27 - Seismic design of precast concrete building structures*, vol. 1994. 2003.

[11] European Committee for Standardization, *Eurocode 8: Design of structures for earthquake resistance — Part 1: General rules, seismic actions and rules for buildings*. 2015.

[12] B. Zoubek, T. Isakovic, Y. Fahjan, and M. Fischinger, "Corrigendum to 'Cyclic failure analysis of the beam-to-column dowel connections in precast industrial buildings' [Eng. Struct. 52 (2013) 179-191]," *Eng. Struct.*, vol. 57, p. 276, 2013, doi: 10.1016/j.engstruct.2013.09.036.

[13] N. Batalha, H. Rodrigues, A. Arêde, A. Furtado, R. Sousa, and H. Varum, "Cyclic behaviour of precast beam-to-column connections with low seismic detailing," no. June 2021, pp. 1–19, 2022, doi: 10.1002/eqe.3606.

[14] C. W. French, M. Hafner, and V. Jayashankar, "Connections between precast elements - failure within connection region," vol. 115, no. 12, pp. 3171–3192, 1990.

[15] R. E. Englekirk, "Development and Testing of a Ductile Connector for Assembling Precast Concrete Beams and Columns," *PCI J.*, vol. 40, no. 2, pp. 36–51, 1995.

[16] K. S. Elliott, G. Davies, M. Ferreira, H. Gorgun, and A. A. Mahdi, "Can precast concrete structures be designed as semi-rigid frames? Part 1 – The experimental evidence," *Struct. Eng.*, vol. 81, no. 16, pp. 14–19, 2003.

[17] D. A. Bournas, P. Negro, and F. J. Molina, "Pseudodynamic tests on a full-scale 3-storey precast concrete building : Behavior of the mechanical connections and floor diaphragms," *Eng. Struct.*, vol. 57, pp. 609–627, 2013, doi: 10.1016/j.engstruct.2013.05.046.

[18] P. Negro, D. A. Bournas, and F. J. Molina, "Pseudodynamic tests on a full-scale 3-storey precast concrete building : Global response," *Eng. Struct.*, vol. 57, pp. 594–608, 2013, doi: 10.1016/j.engstruct.2013.05.047.

[19] Y. Zhong, F. Xiong, J. Chen, A. Deng, W. Chen, and X. Zhu, "Experimental Study on a Novel Dry Connection for a Precast Concrete Beam-To-Column Joint," *Sustainability*, vol. 11, no. 17, p. 4543, 2019, doi: doi:10.3390/su11174543.

[20] K. Ding, Y. Ye, and W. Ma, "Seismic performance of precast concrete beam-column joint based on the bolt connection," vol. 232, p. 111884, 2021, doi: 10.1016/j.engstruct.2021.111884.

[21] J. Liu, Y. Liu, and D. Yu, "Experimental and Numerical Studies on the Seismic Performance of New Assembled Concrete Frame," *Buildings*, vol. 13, p. 329, 2023, doi: https://doi.org/10.3390/ buildings13020329.

[22] T. Krishnan and R. Purushothaman, "Effect of stiffeners in cleat angle based precast beam-column connections under reverse cyclic loading," *Structures*, vol. 25, pp. 161–172, 2020, doi: 10.1016/j.istruc.2020.03.012.

[23] R. Vidjeapriya and K. P. Jaya, "Experimental Study on Two Simple Mechanical Precast Beam-Column Connections under Reverse Cyclic Loading," *J. Perform. Constr. Facil.*, vol. 27, no. July/August, pp. 402–414, 2013, doi: 10.1061/(ASCE)CF.1943-5509.0000324.

[24] R. Zhang, Y. Zhang, A. Li, and T. Y. Yang, "Experimental study on a new type of precast beam-column joint," *J. Build. Eng.*, vol. 51, p. 104252, 2022, doi: 10.1016/j.jobe.2022.104252.

[25] R. Zhang, T. Guo, and A. Li, "Thin-Walled Structures Experimental investigation into demountable dry connections for fully precast frame structures through shaking table tests," *Thin-Walled Struct.*, vol. 201, p. 112014, 2024, doi: 10.1016/j.tws.2024.112014.

[26] H. Zhang, W. Huang, J. Li, and W. Quan, "Seismic behavior of precast concrete beam-to-column connection with double box bolted couplers," *Eng. Struct.*, vol. 326, p. 119499, 2025, doi: 10.1016/j.engstruct.2024.119499.

[27] C. Liu, X. Chen, X. Mao, B. Zhu, and B. Liu, "Shear resistance of an anchoring coupler applied in demountable concrete connection at varying shear positions," *Structures*, vol. 55, pp. 866–875, 2023, doi: 10.1016/j.istruc.2023.06.088.

[28] M. Senturk, S. Pul, A. Ilki, and I. Hajirasouliha, "Development of a monolithic-like precast beam-column moment connection : Experimental and analytical investigation," *Eng. Struct.*, vol. 205, p. 110057, 2020, doi: 10.1016/j.engstruct.2019.110057.

[29] J. D. Nzabonimpa, W. K. Hong, and S. C. Park, "Experimental investigation of dry mechanical beam–column joints for precast concrete based frames," *Struct. Des. Tall Spec. Build.*, vol. 26, no. 1, Jan. 2016, doi: 10.1002/tal.1302.

[30] B. Milosavljević, I. Milićević, M. Pavlović, and M. Spremić, "Static behaviour of bolted shear connectors with mechanical coupler embedded in concrete," *Steel Compos. Struct.*, vol. 29, no. 2, pp. 257–272, 2018, doi: 10.12989/scs.2018.29.2.257.

[31] I. Milićević, B. Milosavljević, M. Pavlović, and M. Spremić, "Bolted connectors with mechanical coupler embedded in concrete: Shear resistance under static load," *Steel Compos. Struct.*, vol. 36, no. 3, pp. 321–337, 2020, doi: 10.12989/scs.2020.36.3.321.

[32] I. Milićević, B. Milosavljević, M. Spremić, R. Mandić, and M. Popović, "Local behaviour of the connector with mechanical coupler and rebar anchor under tension load," *Build. Mater. Struct.*, vol. 66, no. 2, p. 2300002M, 2023, doi: 10.5937/grmk2300002m.

[33] I. Milićević, B. Milosavljević, and M. Spremić, "Behavior of Demountable Connections with Taper Threaded Mechanical Coupler (TTC) and Rebar Anchor Under Tension Load," *Buildings*, vol. 15, no. 6, pp. 1–30, 2025, doi: 10.3390/buildings15060928.

[34] P. K. Aninthaneni, R. P. Dhakal, J. Marshall, and J. Bothara, "Nonlinear Cyclic Behaviour of Precast Concrete Frame Sub-Assemblies With 'Dry' End Plate Connection," *Structures*, vol. 14, pp. 124–136, 2018, doi: 10.1016/j.istruc.2018.03.003.

[35] P. K. Aninthaneni and R. P. Dhakal, "Demountable Precast Concrete Frame–Building System for Seismic Regions: Conceptual Development," *J. Archit. Eng.*, vol. 23, no. 4, pp. 1–10, 2017, doi: 10.1061/(asce)ae.1943-5568.0000275.

[36] P. K. Aninthaneni, R. P. Dhakal, J. Marshall, and J. Bothara, "Seismic performance of sub-assembly of a demountable precast concrete frame building," in *Proceedings of the Tenth Pacific Conference on Earthquake Engineering, Building an Earthquake-Resilient Pacific*, 2015, no. 6-8 November 2015, pp. 189–198.

[37] Z. Y. Li, S. B. Kang, H. He, W. Q. Lu, H. J. Liu, and C. J. Lu, "Seismic behaviour of precast concrete beam-column connections with bolted end plates," *Structures*, vol. 58, no. October, p. 105343, 2023, doi: 10.1016/j.istruc.2023.105343.

[38] C. Wu *et al.*, "A detachable configuration for the precast beam-column connection using metallic damper as connector : Experimental investigation," *J. Build. Eng.*, vol. 65, p. 105597, 2023, doi: 10.1016/j.jobe.2022.105597.

[39] J. Bai, J. He, C. Li, S. Jin, and H. Yang, "Experimental investigation on the seismic performance of a novel damage-control replaceable RC beam-to-column joint," *Eng. Struct.*, vol. 267, p. 114692, 2022, doi: 10.1016/j.engstruct.2022.114692.

[40] W. Huang, G. Hu, and J. Zhang, "Experimental study on the seismic performance of new precast concrete beam-column joints with replaceable connection," *Structures*, vol. 35, pp. 856–872, 2022, doi: 10.1016/j.istruc.2021.11.050.

[41] W. Li, W. Chen, X. Lin, L. Chen, H. Lv, and K. Shao, "Cyclic behavior and damage mechanisms of a demountable RC joint : Integrated experimental and analytical study," *Structures*, vol. 79, p. 109609, 2025, doi: 10.1016/j.istruc.2025.109609.

[42] L. Xie, J. Wu, J. Zhang, and C. Liu, "Experimental study of mechanical properties of beam-column joint of a replaceable energy-dissipation connector-precast concrete frame," *J. Build. Eng.*, vol. 43, p. 102588, 2021, doi: 10.1016/j.jobe.2021.102588.

[43] Z. Wang, D.-C. Feng, and G. Wu, "Experimental Study on Seismic Behavior of Precast Bolt-Connected Steel-Members End-Embedded Concrete (PBSEC) Beam-Column Connections," *Buildings*, 2022.

[44] K. C. G. Ong, Z. S. Li, L. R. Chandra, C. T. Tam, and S. D. Pang, "Experimental investigation of a DfD moment-resisting beam – column connection," *Eng. Struct.*, vol. 56, pp. 1676–1683, 2013, doi: 10.1016/j.engstruct.2013.08.006.

[45] J. Xiao, T. Ding, and Q. Zhang, "Structural behavior of a new moment-resisting DfD concrete connection," *Eng. Struct.*, vol. 132, pp. 1–13, 2017, doi: 10.1016/j.engstruct.2016.11.019.

[46] J. Xiao, Z. Chen, T. Ding, and B. Xia, "Effect of recycled aggregate concrete on the seismic behavior of DfD beam- column joints under cyclic loading," *Adv. Struct. Eng.*, vol. 24, no. 8, pp. 1–15, 2020, doi: 10.1177/1369433220982729.

[47] T. Ding, J. Xiao, E. Chen, and A.-R. Khan, "Experimental Study of the Seismic Performance of Concrete Beam-Column Frame Joints with DfD Connections," vol. 146, no. 4, p. 04020036, 2020, doi: 10.1061/(ASCE)ST.1943-541X.0002576.

[48] M. J. N. Priestley and J. R. Tao, "Seismic Response of Precast Prestressed Concrete Frames With Partially Debonded Tendons," *PCI J.*, vol. 38, no. 1, pp. 58–67, 1993.

[49] H. Wang, E. M. Marino, P. Pan, H. Liu, and X. Nie, "Experimental study of a novel precast prestressed reinforced concrete beam-to-column joint," *Eng. Struct.*, vol. 156, pp. 68–81, 2018, doi: 10.1016/j.engstruct.2017.11.011.

[50] X. Cai, Z. Pan, Y. Zhu, N. Gong, and Y. Wang, "Experimental and numerical investigations of self-centering post-tensioned precast beam-to-column connections with steel top and seat angles," *Eng. Struct.*, vol. 226, p. 111397, 2021, doi: 10.1016/j.engstruct.2020.111397.

[51] N. Sripongngam, N. Wonghiran, P. Thongou, S. Thianthong, E. Yooprasertchai, and C. Jaturapitakkul, "Development and experimental study of innovative precast column – footing connections with self-centering and energy-dissipating bolts," *Structures*, vol. 86, no. December 2025, p. 111305, 2026, doi: 10.1016/j.istruc.2026.111305.

[52] R. Nascimbene and L. Bianco, "Cyclic response of column to foundation connections of reinforced concrete precast structures: Numerical and experimental comparisons," *Eng. Struct.*, vol. 247, no. August, 2021, doi: 10.1016/j.engstruct.2021.113214.

[53] D. Figueira, A. Ashour, G. Yıldırım, A. Aldemir, and M. Şahmaran, "Demountable connections of reinforced concrete structures: Review and future developments," *Structures*, vol. 34, no. September, pp. 3028–3039, 2021, doi: 10.1016/j.istruc.2021.09.053.

[54] B. Dal Lago, G. Toniolo, and M. Lamperti Tornaghi, "Influence of different mechanical column-foundation connection devices on the seismic behaviour of precast structures," *Bull. Earthq. Eng.*, vol. 14, no. 12, pp. 3485–3508, 2016, doi: 10.1007/s10518-016-0010-9.

[55] S. Pul, M. Senturk, A. Ilki, and I. Hajirasouliha, "Experimental and numerical investigation of a proposed monolithic-like precast concrete column-foundation connection," *Eng. Struct.*, vol. 246, no. September, p. 113090, 2021, doi: 10.1016/j.engstruct.2021.113090.

[56] Q. Zhou, Y. Liu, and Y. Li, "Load transfer mechanism of precast concrete piers with demountable connections," *Eng. Struct.*, vol. 261, Jun. 2022, doi: 10.1016/j.engstruct.2022.114287.

[57] Q. Zhang *et al.*, "Behavior and design of demountable and replaceable RC columns with bolted flange connections under eccentric compression," *Eng. Struct.*, vol. 310, no. March, 2024, doi: 10.1016/j.engstruct.2024.118138.

[58] M. Orlando and L. R. Piscitelli, "Experimental investigation on static and cyclic behaviour of flanged unions for precast reinforced concrete columns," *Eur. J. Environ. Civ. Eng.*, vol. 22, no. 8, pp. 927–945, 2018, doi: 10.1080/19648189.2016.1229226.

[59] R. Aktepe *et al.*, "Fully demountable column base connections for reinforced CDW-based geopolymer concrete members," *Eng. Struct.*, vol. 290, no. May, p. 116366, 2023, doi: 10.1016/j.engstruct.2023.116366.

[60] S. Akduman *et al.*, "Structural performance of construction and demolition waste-based geopolymer concrete columns under combined axial and lateral cyclic loading," *Eng. Struct.*, vol. 297, no. May, p. 116973, 2023, doi: 10.1016/j.engstruct.2023.116973.

[61] J. Liu, Y. Xue, C. Wang, J. Nie, and Z. Wu, "Experimental investigation on seismic performance of mechanical joints with bolted flange plate for precast concrete column," *Eng. Struct.*, vol. 216, no. September 2019, p. 110729, 2020, doi: 10.1016/j.engstruct.2020.110729.

[62] J. D. Nzabonimpa, W. K. Hong, and J. Kim, “Mechanical connections of the precast concrete columns with detachable metal plates,” *Struct. Des. Tall Spec. Build.*, vol. 26, no. 17, pp. 1–14, 2017, doi: 10.1002/tal.1391.

[63] J. D. Nzabonimpa and W. K. Hong, “Structural performance of detachable precast composite column joints with mechanical metal plates,” *Eng. Struct.*, vol. 160, no. June 2017, pp. 366–382, 2018, doi: 10.1016/j.engstruct.2018.01.038.

[64] J. Y. Hu, W. K. Hong, and S. C. Park, “Experimental investigation of precast concrete based dry mechanical column–column joints for precast concrete frames,” *Struct. Des. Tall Spec. Build.*, vol. 26, no. 5, pp. 1–15, 2017, doi: 10.1002/tal.1337.

[65] G. Wang, Y. Li, Z. Li, and J. M. Ingham, “Experimental and numerical study of precast concrete columns with hybrid bolted splice connections,” *Structures*, vol. 28, no. August, pp. 17–36, 2020, doi: 10.1016/j.istruc.2020.08.042.

[66] Y. Qing, C. Wang, S. Meng, and B. Zeng, “Experimental study on the seismic performance of precast concrete columns with thread-bolt combination couplers,” *Eng. Struct.*, vol. 251, no. PA, p. 113461, 2022, doi: 10.1016/j.engstruct.2021.113461.

[67] Y. Qing, Y. Yuan, C. L. Wang, S. Meng, and B. Zeng, “Experimental study of precast columns with a hybrid joint incorporating bolted couplers and grouted sleeves,” *J. Build. Eng.*, vol. 74, no. February, pp. 1–17, 2023, doi: 10.1016/j.jobe.2023.106878.

[68] Y. T. Lu, Z. X. Guo, Y. Liu, and S. H. Basha, “Performance of prefabricated RC column with replaceable Column-Base connection under cyclic lateral loads,” *Eng. Struct.*, vol. 240, no. April, p. 112343, 2021, doi: 10.1016/j.engstruct.2021.112343.

[69] H. Zhan, M. Ye, J. Jiang, Y. Gao, C. W. Zheng, and S. C. Duan, “Structural performance of detachable precast concrete column-column joint,” *Heliyon*, vol. 10, no. 5, pp. 1–17, 2024, doi: 10.1016/j.heliyon.2024.e27308.

[70] D. Yuan, Q. Shang, C. Han, J. Sun, and Q. Li, “Seismic Performance of Precast Concrete Column-to-Column Joint Using the Steel Plate Hoop and Bolts Connection,” *Adv. Civ. Eng.*, vol. 2023, 2023, doi: 10.1155/2023/6636781.