Strain evolution of mechanical joints for L-shaped precast columns
Part II: Application to a tall building

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ABSTRACT
This study succeeded in an experimental investigation of the authors’ previous studies in which L-shaped dry mechanical joints were developed and tested to demonstrate applicability to precast concrete frames. In their early study, the use of L-shaped dry mechanical joints assembling precast concrete frames was verified both experimentally, demonstrating all structural components at joint experimentally functioned well, supporting both cyclic and monotonic loads. In the present study, design recommendations and collateral benefits were provided to design tall building frames based on an extensive strain analysis, while improving joints using interior stiffeners to greatly enhance the structural performance of mechanical joints. How fast strains of joints are developed as deflections progressed was also explored based on rates of strain increase relative to deflection, while identifying degradation mechanism of mechanical joints (including rebars and steel sections attached to mechanical plates) that led to brittle failure modes. This study aims to provide design protocol for a 20-story apartment building with L-shaped precast columns spliced by mechanical plates. Beam-column joints were assembled by extended beam endplates, which were bolted to plates embedded in columns. A sufficient beam depth was determined by avoiding stress concentration exerted by extended beam endplates.

1. Introduction
1.1. Motivation and the significance of the study
The use of precast members has begun to replace the conventional wet construction cast-in-place method over the past several years. Application of precast concrete members has been improved substantially by many studies that investigated the erection and installation process of precast frames, including a study conducted by Proverbs, Holt, and Olomolaiye (1998).

D’Aniello, Cassiano, and Landolfo (2016) described the results of an experimental investigation devoted to characterize the monotonic and cyclic tensile behavior of European pre-loadable grade 10.9 bolts (HR and HV bolts) commonly used for structural applications. They concluded that HR bolt assemblies were characterized by shank necking failure, whereas nut stripping occurred for HV bolt assemblies. The failure mode from nut stripping to shank necking was modified using HV assemblies. D’Aniello, Cassiano, and Landolfo (2017) also simplified criteria for finite element modeling of European pre-loadable bolts. They modeled criteria for European pre-loadable grade 10.9 HR and HV bolts in two ways depending on different levels of complexity and refinement; (1) the simplified equivalent shank model and (2) the refined ductile damage model. Both proposed modeling approaches were compared to accurately simulate all stages of bolt assembly response, including elastic response, plasticity onset, softening, and initiation of failure.

One common way to connect precast columns is to use cylindrical steel sleeves to splice reinforcing bars that are capable of providing full tension and...
column joints can be utilized for building assemblies to support both cyclic and monolithic loads.

2. Nonlinear finite element analysis of L-type columns with/without foundations

2.1. Choice of elements and discretization

FE models for columns and beams including proposed mechanical joints were discretized based on continuum elements (eight-node linear brick) using three types of elements (C3D8R and R3D4 and B31, refer to Table 1) as indicated in Figure 2(a,b), respectively. Element types, including the C3D8R element (known as reduced integration element), were used to model proposed mechanical joints. Column rebars were modeled by B31 elements (representing a beam element) to reduce the number of degrees of freedom in the FE model. Column-to-column and column-to-beam joints of a building are shown in Figure 1. A total of 201,632 and 113,432 elements were established for column-to-column and beam-to-column joints as shown in Figure 2(a)-(1) and (b)-(1), respectively, in the FE model. This includes R3D4 elements, each having four nodes with three DOFs. Stresses and strains of column connections and column-to-beam joints shown in Figure 2(a,b) were investigated based on microstrain evolution. A displacement-controlled monotonic load was applied at a reference point located by a rigid body, which was modeled by three-dimensional quadrilateral elements (R3D4).

2.2. Modeling surface element, defining interactions, surface-to-surface contact

ABAQUS offers various methods for assigning contact interactions between deformable and rigid

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**Table 1. Description of FEA elements.**

| Element | Type          | Description                                           |
|---------|---------------|-------------------------------------------------------|
| C3D8R   | 3D solid element | Eight-node linear brick, reduced integration with hourglass control |
| B31     | Beam element   | Two-node linear beam                                    |
| R3D4    | Rigid element  | Four-node, bilinear quadrilateral                       |
(1) FEA model for a mechanical column joint

(2) Element types used in the mechanical column joints
(a) Description of finite element model for column joints (Nguyen, Hong, and Kim 2020)

(1) FEA model for beam-to-column joint

(2) Element types used in the mechanical beam-to-column joints
(b) Description of the finite element model for column-girder joints

Figure 2. Finite element models of frame shown in Figure 1
As shown in Figure 3, contact interactions between two surfaces that interacted with each other during analysis were defined. In ABAQUS, two contact regions must be selected to assign contact pairs. Choices of contact discretization, tracking approach, and assignment of master and slave roles to contact surfaces were addressed to formulate contacts, defining contact interaction based on a surface-to-surface approach (DSS 2014). Figure 3 depicts master and slave surfaces used to study nonlinear behaviors of the proposed Lego-type columns. Surface-to-surface contact properties were defined based on tangential and normal behavior. Tangential behavior was defined as frictionless, while normal behavior was assigned based on a penalty method, with hard contact following a pressure-overclosure relationship in which surface-to-surface contacts were constrained by a penalty stiffness (DSS 2014). A penalty method allows some minor penetrations to occur between master and slave surfaces. Contact force is proportional to penetration distance. Contact surfaces, including surfaces between different plates, plates and bolts, and concrete and plates, were implemented in a numerical model for mechanical connections. As shown in Figure 3, nine interactions for column-to-column (refer to Figure 3 and Table 2(a)) and beam-to-column (refer to Table 2(b)) mechanical joints were established to define contacts. Nodes
| Interaction type | Master surface | Slave surface | Interaction type | Master surface | Slave surface |
|-----------------|----------------|---------------|-----------------|----------------|---------------|
| Int-1           | Upper steel plate | Upper concrete | Int-4           | Rebar, nuts    | Plate holes for nuts |
| Int-2           | Lower steel plate | Lower concrete | Int-5           | Exterior Bolts  | Plate holes for bolts |
| Int-3           | Upper steel plate | Lower steel plate |               |                |               |
| Int-6           | Bolts            | Plate holes for bolts | Int-8           | Rebar          | Plate holes for rebar |
| Int-7           | Beam endplate    | Beam-Column concrete | Int-9           | Rebar, nut     | Plate hole for nut |

Table 2. Description of contact surfaces for column-to-column joints.
of metal plates were shared with those of steel section at their contacts in FE models, resulting in complete composite action in these two materials.

2.3. Defining constraints and material properties

ABAQUS allows users to define constraints between reinforcing bars and concrete, by placing embedded elements (reinforcing bars) into host elements (concrete). ABAQUS tracks embedded elements, which are then constrained by the response of host elements. ABAQUS eliminates translational DOFs of nodes in cases when embedded elements lie within host region. These nodes are referred to as embedded nodes. The translational movement of embedded elements is controlled by host elements. Embedded elements, including L-shaped steels, rebars, and hoops were located inside host elements (upper and lower concrete columns) in present FE models, as shown in Figure 3. As described in a previous study of authors (Nzabonimpa and Hong 2019) and Table 3, material properties for embedded and host elements were defined based on tested samples. The average concrete strength obtained from test samples was 29.1 MPa. Embedded steels and rebars were also tested, and their results indicated that the average yield strength of L-shaped steels was 387.1 MPa, while an average of tensile yield stress of rebars was 646.7 MPa.

2.4. Review of experimental investigation

2.4.1. Details for rebar anchoring

The detailed drawings including all details and dimensions of mechanical joints are shown in Figure 4 where rebar nuts are accommodated in counterbores to anchor rebars to steel plates. As shown in Figure 15 (c), bolts were used to anchor beam endplate to column concrete.

2.4.2. Details of the connection between steel plate and rebars

Connection details between steel plate and rebars are demonstrated for the frame modules as shown in Figure 5 where the three types of connection details between

| Table 3. Summary of material properties of rebar, metal plates, and concrete (Nzabonimpa 2018). |
|---------------------------------|----------------|------------------|
| Category                       | Size [mm]      | Material         |
| Concrete column                | L shape: 800 × 800 (width: 200) | Concrete compressive strength: 40 MPa |
| Concrete girder                | B x H: 400 × 200 |                  |
| Upper plate for the columns    | L shape: 1000 × 1000 (width: 400, thickness: 35) | Material (steel, SM490), tensile yield stress ($f_y = 325$ MPa) |
| Lower plate for the columns    | L shape: 1000 × 1000 (width: 400, thickness: 35) | Ultimate strength ($f_u = 490$ MPa) |
| Extended end plate for the girders | 480 x 200 x 30 |                  |
| Filler plate for the girders    | 480 x 200 x 5  |                  |
| Bolts for all specimens        | M22 (diameter: 22) | Tensile yield stress ($f_y = 900$ MPa) |
| Rebars for all specimens       | HD25 (diameter: 25) | Ultimate strength ($f_u = 1000$ MPa) |
| Hoops for the columns and stirrups for the girders | HD10 (diameter: 10) | Tensile yield stress ($f_y = 600$ MPa) |

Figure 4. Detailed drawings of mechanical joint of L-shaped column.
(1) Parallel type
Exposed nuts on the face of the lower column plate

(a) Exposed nuts on the face of column plate (Nzaborinmpa, Hong, and Kim 2017a)

(2) Installation

(1) Pit type (Nzaborinmpa, Hong, and Kim 2017a)

Marking plate holes using electric drilling machine

(2) Counterbores at the rear end of the plates for anchoring rebars (Nzaborinmpa, Hong, and Kim 2017a)
steel plate and rebars are summarized. Rebars from upper and lower columns are fixed to column plates via nuts. In Figure 5(a) (Nzabonimpa, Hong, and Kim, 2017a), all rebars are bent to avoid nuts of different rebars from running into each other in filler plates. Locations of nuts and size of nuts’ holes are determined so as not to bump into filler plates during the deformation of column plates. Nuts from both upper and lower columns occupy each hole in filler plates after columns are connected. A metal filler plate with a thickness...
sufficient to encompass the height of one nut is used in this type; however, costs for bending all vertical rebars are required. In Figure 5(b)-(1) to 5(b)-(3) (Nzabonimpa, Hong, and Kim, 2017a) with a pit type of connection, nuts threaded with rebars are accommodated in the counterbores of column plates without using a filler plate. Column plates encasing nuts should be thick enough to encase and protect nuts completely. A bent rebar is not necessary because nuts are kept in column plates, not in a filler plate. The counterbores of plates with and without nuts are shown in Figure 5(b)-(2). The stability of this type of column joints is contributed by the shear strength of a neck of plates having counterbores. Figure 5(b)-(3) shows the fabrication of L-shaped steel column with a metal column plate. A stacked type of connection is installed on the top of each other, requiring filler plates twice as thick (equivalent to a height of the two combined nuts) as those of a parallel type shown in Figure 5(a) whereas Figure 5(c) (Nzabonimpa, Hong, and Kim, 2017a) highlights the use of thick plates to accommodate upper and lower nuts when counterbores are not required in plates. The thinnest plate without the use of thick filler plate at the cost of bending vertical rebars are used for a parallel type shown in Figure 5(a) while filler plates used for stacked save the costs for bending rebars and making counterbores in the column plates. Filler plates of both parallel and stacked types are not used as a structural element; instead, it is used only to hide nuts connecting column rebars to metal plates, not contributing to the flexural strength of the joints.

2.4.3. Role of the interior bolts
Tension exerted from rebar and steel sections may require an uneconomical plate thickness, resulting in an impractical design if column plates do not have enough stiffness. The use of interior bolts with laminated metal and filler plates is to increase the flexural stiffness of column plates between joints. Interior bolts were used to improve the efficiency of designs, splicing two precast column components using a pair of stiffened column plates with high-strength interior bolts. The influence of stiffened plates on flexural capacity and structural behavior of mechanical connections increased. The flexural resisting capacity of spliced columns was enhanced to almost a level of conventional monolithic cast-in-place columns, significantly reducing the deformation of stiffened mechanical plates with interior bolts. An impractical design of mechanical connections was avoided by increasing the stiffness of column plates. The recessed area for installing interior bolts (refer to Figure 6(a)) should be grouted with nonshrinking high-strength mortar as shown in Figure 6(b). The authors presented in their earlier publication shown in Figure 7, mechanical connections incorporating interior bolts were implemented to reduce the thickness of laminated plates. In Figure 7(a), interior bolts are installed to increase the stiffness of splicing plates. Sufficient stiffness and strength of plates contributed to the transfer of loads through joints, preventing a prying action of plates. Figure 7(b-e) demonstrates an installation sequence, showing bent rebars, nuts, and heads of interior bolts buried in the lower column. The filler plate accommodated nuts threaded onto the ends of rebars at the face of the lower column plate. In Figure 7(b-e), the upper column plate was installed encasing interior bolts. In Figure 7(e), interior bolts installed in the recessed access shown in Figure 7(d) were then grouted with nonshrinking high-strength mortar.

2.4.4. Load path
Column loads moved down to upper steel plates through upper column rebars and steel sections (if any) anchored by nuts as shown in Figure 8. Lower steel plates picked loads up via exterior and internal bolts with nuts and, further, finally, column loads were picked up by vertical column re-bars and steel sections below (Hu and Hong, 2017).

2.4.5. Test set-up
The test setup of L-shaped columns is described in Figure 9 where the L-shaped column was connected to a rigid floor through a strong foundation to keep specimens stable during testing. Lateral cyclic loads were controlled by lateral cyclic displacements which were exerted at a level of 2700 mm height. These displacements were created by an actuator. Gauges were installed on columns to measure strains of concrete, rebars, steel, and plates (Nzabonimpa and Hong, 2019). Figure 13(c) shows test set-up of beam-column joint.

Figure 10 presents the load protocol and the lateral load-drift ratio of the specimens (Nzabonimpa and Hong, 2019).

2.4.6. Test results
(1) Column-column test
Test results of L-shaped dry mechanical joints splicing precast concrete frames can be found in our past work (Nzabonimpa and Hong, 2019; Hong, 2019). Constitutive relationships of embedded elements (L-shaped steels, rebars) were determined based on elastic-plastic property and constitutive relationships defined by elastic-softening behavior were utilized for interior stiffeners (headed studs). Latter accounted for low cycle fatigue proposed by (Nzabonimpa, Hong, and Kim 2017b) for headed studs used as interior stiffeners, while concrete was defined in Figure 11(a) (Nzabonimpa and Hong, 2019) based on a damaged plasticity model constructed using stress-strain curve for unconfined concrete suggested by Kent–Park (Park and Paulay 1975). Low cycle fatigue (elastic-softening)
(a) showing recessed area before being grouted; nuts fastening interior bolts in recessed area (Nzabonimpa, Hong 2019)

(b) showing recessed area after being grouted with non-shrinking high strength mortar

*Figure 6.* Spliced precast columns using a pair of stiffened column plates with high-strength interior bolts.
(a) Bent rebars, nuts threaded with the ends of rebars, and heads of interior bolts buried in the lower column.

(b) Assembly of the filler plate.

(c) Erection of the column.

(d) Nuts fastening interior bolts at recessed area.

(e) Grouted recessed area.

Figure 7. Installation of interior bolts and rib stiffeners (Nzabonimpa, Hong, and Kim 2017a; Nzabonimpa, Hong 2019).
assigned to headed studs which were used to stiffen metal plates decreased flexural behavior of columns after around 80 mm, as shown in Figure 11(b) (Nzabonimpa and Hong 2019). Strains in the interior bolts stiffening metal plates were seven times higher than those in the exterior bolts at a stroke of 54 mm (refer to the load–displacement relationship in Legend 5 of Figure 11(b)), indicating that the interior bolts significantly contributed to the flexural capacity of the joint.

The stroke (26 mm) of Specimen LC2 (with interior bolts) was smaller than that (36.4 mm) of Specimen LC1 (without interior bolts) when a concrete compressive strain reached 0.003 (design limit) as shown in Figure 12(a,b). Metal plates stiffened with interior bolts exhibited a small deformation of 0.4 mm, while metal plate without interior bolts experienced a large deformation of 1.5 mm (see Figure 12(a,b) (Nzabonimpa and Hong 2019). At a concrete compressive strain of 0.003 (design limit), strains in upper and lower column plates were also small (0.0014 (0.7εy) and 0.0013 (0.7εy), respectively), while they were greater (0.006 (3εy) and 0.007 (5εy), respectively) when interior bolts were not used. This indicates that interior bolts (0.009, 2εy) significantly enhanced the structural performance of the proposed mechanical joint. Strains (0.002, 1.0εy) of steel sections attached to plates in Specimen LC2 with interior bolts were more highly activated than those (0.0016, 0.8εy) in
Specimen LC1 with no interior bolts. However, a much smaller tensile strain was exerted on exterior bolts (0.002, 0.4εx) in LC2 than on that (0.028, 6εx) in LC1 due to interior bolts, demonstrating that structural behavior became more stable with reduced strain levels in exterior bolts. Interior bolts were well activated compared to exterior bolts for Specimen LC2. For interior bolts, a strain of 0.009 (2εx) was observed at a concrete strain of 0.003 (refer to Figure 12(b)), and a strain of 0.03 (7εx) was reached at a stroke of 54 mm (refer to Figure 8, Nzabonimpa and Hong 2019). Table 5 summarizes strains of proposed column connection identified in Legends 2 and 5 in Figure 11(b) for a specimen with the interior bolts (LC2-WF) (Nzabonimpa and Hong 2019).

(2) Column-beam test

Table 4 shows beam-column tests performed to verify beam-column joints having plate connections (Nzabonimpa, Hong, and Park 2017). Tie (cohesive) model (Specimen B5) indicated by Legend 2 of Figure 13(a) provided a better prediction of the load–displacement relationships than an embedded model shown by Legend 4 of Figure 13(a) did when the elasto-plastic material property was implemented in rebars and steel sections. In Figure 13(b) and (c), a measured deformation of beam endplate shown for Specimen B5 (refer to Table 4) predicted a plate deformation of 13–17 mm of a mechanical joint, well correlating that based on nonlinear finite element model (Hong 2019; Nzabonimpa, Hong, and Park 2017; Nzabonimpa, Hong, and Kim 2017b). A reliable strain evolution of structural components was retrieved at joints including column and beam plates assembled by high-strength bolts.

### 2.5. Design verification

#### 2.5.1. Dynamic analysis of high-rise buildings with multi-bay L-type composite precast frames

Frame analysis based on dynamic response spectrum analysis for the selected 20-story apartment building shown in Figure 1(a) (Nzabonimpa and Hong 2019) was performed to obtain moment demand (M0), shear demand (V0), and axial load demand (P0) using Midas (refer to Table 5, Nzabonimpa and Hong 2019). These loads must be supported by the proposed L-type composite precast frames with mechanical joints. The present study investigated the strain evolution of mechanical joints for L-shaped precast columns based on monotonic nonlinear finite element analysis.

#### 2.5.2. Nonlinear numerical model and description for beam-column frames

The boundary conditions of the beam-column model were described in Figure 14. Material properties and nonlinear numerical parameters used to study the structural behavior of the proposed frames (while considering concrete plasticity) are summarized in Table 7. Drucker-Prager hyperbolic plastic potential function is constructed for a dilation angle of 30° to determine the direction of the plastic increment vector. A detailed discussion of plastic parameters can be found in many references including the study by (Hong 2019).

### 2.6. Determination of nominal strength at a concrete strain of 0.003 to design beam-column joints

Nominal flexural strengths of mechanical joints of girders 200 mm wide × 400 mm deep at Mu and at
a concrete strain of 0.003 (based on average section strain of 0.003) are identified in Figures 15 and 16, respectively. Nominal flexural strengths of girders 200 mm wide \( \times \) 500 mm deep at \( M_u \) and at a concrete strain of 0.003 are also shown in Figure 17 in which nominal flexural strengths were obtained considering neutral axis determined based on an average compressive concrete strain of 0.003 (refer to Figure 17 (b)-(1)) of composite column section without interior bolts when an axial force of 5,000 kN was exerted. In Table 7, moment demands \( (M_u) \) are obtained as 355 kN-m for the column with a column concrete strain of 0.00048 (refer to Figure 15(a)) whereas, in Figure 16(a), nominal flexural and shear capacities of the column at a concrete strain of 0.003 are determined to be 1175.3 kN-m and 691 kN, respectively. Moment demands \( (M_u) \) are obtained as 238 kN-m for the beam. For a beam with 200 mm wide \( \times \) 400 mm deep section, flexural capacities of columns are shown in Figure 15(a) at \( M_u \) and 7(a) at a concrete strain of 0.003 whereas the flexural capacity of the beam is shown in Figure 15(c) at \( M_u \) and in 7(c) at
a concrete strain of 0.003. The flexural capacity of a beam with 200 mm wide \times 500 mm deep section is also shown in Figure 17(a) at the \( M_u \) and 8(b) at a concrete strain of 0.003, respectively. Strains are also presented at mechanical joints of the L-shaped column. Mechanical joints of column-girder joint with a 200-mm-wide \times 400-mm-deep girder provided a nominal flexural strength of 208 kN\cdot m (refer to Figure 16(c)) was based on a neutral axis using an average compressive strain of 0.003, which is not sufficient to resist moment \( M_u \) (238 kN\cdot m) shown in Table 6. This indicates that the girder needed to be re-sized. New 200-mm-wide \times 500-mm-deep girder offered a flexural strength of 311 kN\cdot m when a concrete strain of girder reached 0.003, as shown in Figure 17(b)-(2). Strains (0.000612 to 0.000969) on a face of concrete column at a concrete strain of a girder corresponding to 0.003 for 200-mm-wide \times 400-mm-deep girder (refer to Figure 16(d)) increased to 0.00149 – 0.00265 when a girder size increased to 200 mm wide \times 500 mm deep (refer to Figure 17(c)). This resulted in an increase in flexural strength from 208 kN\cdot m to 311 kN\cdot m. A sufficient shear strength was offered by a 13-mm thick neck of counterbores, providing a shear strength greater than punching shear stresses (6.9 MPa) around a neck of counterbores at \( M_u \) as shown in Figure 15(b).
Table 4. Observed test results (Nzabonimpa, Hong, and Park 2017).

| Specimen (plate thickness) | Deformation | Prying action | Max. load (Max. displacement) |
|---------------------------|-------------|---------------|-------------------------------|
| B1 (20 mm)                | Welded rebar| No deformation (premature failure at rebar welding) | Welding fracture | +77 kN (45 mm) |
| B2 (45 mm)                | Embedded nut| No deformation | Fully restrained | +137 kN (95 mm) |
| B3 (20 mm)                | Concrete filler plate | 14–18 mm | Partially restrained | +84 kN (105 mm) |
| B4 (16 mm)                | Concrete filler plate | 15–20 mm | Partially restrained | +72 kN (225 mm) |
| B5 (20 mm)                | Metal filler plate | 13–17 mm | Partially restrained | +84.7 kN (105 mm) |
| B6 Control monolithic specimen | - | - | - |

Table 5. Strains of the proposed column connection identified in Legends 2 and 5 in Figure 4(b): specimen with the interior bolts (LC2-WF) (Nzabonimpa and Hong 2019).

| Load Displacement | FEA results (Legend 5) | Test results (Legend 2) |
|-------------------|-------------------------|-------------------------|
| Concrete Strain   | 0.003                   | 0.002                   |
| Stress            | 26 MPa                  | 7 MPa                   |
| Rebar Strain      | 0.0017                  | 0.006                   |
| (Average) Stress  | 440 MPa                 | 530 MPa                 |
| Steel Strain      | 0.002                   | 0.006                   |
| (Average) Stress  | 490 MPa                 | 550 MPa                 |
| Upper plate Strain| 0.00141                 | 0.0033                  |
| Stress            | 318 MPa                 | 452 MPa                 |
| (Average) Stress  | 3.11 mm                 | 0.67 mm                 |
| Lower plate Strain| 0.00138                 | 0.0031                  |
| Stress            | 317 MPa                 | 447 MPa                 |
| (Average) Stress  | 0.4 mm                  | 0.7 mm                  |
| Exterior bolt Strain| 0.002                   | 0.005                   |
| Stress            | 478 MPa                 | 1005 MPa                |
| (Average) Stress  | 1015 kN                 | 1005 kN                 |

Design shear strength of counterbores was calculated to be 322.4 kN at a concrete strain of 0.003. This was sufficient to resist punching shear stresses of 318.7 MPa, as shown in Figure 16(b), ensuring structural safety at the joint plate.

2.7. Strain evolutions of the mechanical joints

As presented in Figure 18, monolithic columns (moment–displacement relationship in Legend 1) with an axial force of 5,000 kN delivered the greatest moment strength, as indicated by flexural moment capacity corresponding to a concrete strain of 0.003. Moment strengths of columns with mechanical joints regardless of interior bolts at $M_u$ which is similar to those of monolithic column were provided, demonstrating that stiffening of laminated metal plates was sufficient to transfer loads for a given load demand. Note also that moment strength of column with interior bolts (refer to Legend 2 of Figure 18) greater than that of the column without interior bolts was achieved. Average strains in upper and lower plates were $\varepsilon_y = 0.000025$ (average, 0.016$\varepsilon_y$) at $M_u$ (355 kN·m) as shown in Figure 15(a), where a concrete column strain at $M_u$ was found as 0.00048. However, these increased to $\varepsilon_y = 0.0015$ (average, 0.94$\varepsilon_y$) when a concrete strain and flexural strength reached 0.003 and 1175.3 kN·m, respectively, as shown in Figure 16(a). Strains of vertical rebars connected directly to 30-mm-thick metal plates were 0.00006 (0.26$\varepsilon_y$, refer to Figure 15(a)) at $M_u$ and 0.0026 (0.89$\varepsilon_y$, refer to Figure 16(a)) at moment corresponding to a concrete strain of 0.003. Flexural moment capacity of a beam corresponding to a moment demand $M_u$ of 238 kN·m (marked as Point (2) on moment–displacement relationship indicated by Legend (2) of Figures 15 (c) and 19) was greater than nominal moment strength of 208 kN·m. The latter corresponds to a concrete strain of 0.003 (marked as Point (1) on moment–displacement relationship indicated by Legend (1) of Figures 16 (c) and 19), indicating
(a) Load-displacement relationships with tie and embedded

(b) Numerical and experimental observations of the metal plate at a stroke of 115 mm for Specimen B5

(c) Experimental observations of the metal plate at a stroke of 115 mm for Specimen B5

Figure 13. Numerical and test observations for Specimen B5 (Hong 2019; Nzabonimpa, Hong, and Park 2017; Nzabonimpa, Hong, and Kim 2017b).
an insufficient girder size (200 mm wide × 400 mm deep). As shown in Figure 20, a beam flexural moment strength of 311 kN·m corresponding to a concrete strain of 0.003 was achieved when resizing girder to 200 mm width × 500 mm depth (marked as Point (4) on moment–displacement relationship indicated by Legend (4) of Figures 17(b) and 20). This strength was greater than $M_u$ of 238 kN·m. In Legend (2) of Figure 20, Legend (2) of Figures 19(a) and 21(a), a concrete strain of only 0.00099 at $M_u$ for 200-mm-wide × 500-mm-deep girder was indicated in moment–displacement relationship (relieved from 0.006 (refer to Figure 15(c) for 200-mm-wide × 400-mm-deep girder). Strains identified in mechanical beam-column joint with metal plates and bolts are also summarized in Table 8. As shown, the design strength of the proposed girder section was sufficient to support load demand at $M_u$ when the girder was resized to a width of 200 mm and a depth of 500 mm.

### 2.8. Strain evolution of structural components attached to plates

The rate of strain increase of concrete, rebars, and steel flanges attached to plates relative to deflection (strain–stroke relationships of the beam) is presented in Figure 21. Flexural strengths contributed by structural components attached to plates were identified based on microscopic strains as presented in Table 8 and Figure 21. No structural degradations of plates or degradations of joints were observed. However, considerable strains were developed at structural elements attached to plates as shown in Figure 21. Figure 21(a) shows that strains of a concrete beam section having mechanical joints with a depth of 400 mm (refer to Legend 4 of Figure 21(a)) increase rapidly compared to those with a monolithic control beam section of a depth of 400 mm (refer to Legend 3 of Figure 21(a)). Design strengths for control monolithic beam-column joint at a concrete strain of 0.003 were sufficient to resist moment demand ($M_u$) corresponding to a concrete strain of 0.0024 as shown in Table 8 and Legend 3 of Figure 21(a), whereas a strain of concrete beam section (having mechanical joints) greater than 0.006 was reached at moment demand ($M_u$) as shown in Table 8, Figure 15(c) and Legend 4 of Figure 21(a) for a beam depth of 400 mm, in which $M_u$ is rapidly reached. The design strength of the latter is not sufficient to resist the required moment ($M_u$). Design strengths of both monolithic beam-column joint and beam-column joint having a mechanical plate are sufficient to resist the required moment ($M_u$) at a concrete strain of 0.003 (refer to Legends 1 and 2 Figure 21(a)) when a beam depth was increased to 500 mm. A strain of beam section (having a mechanical plate) at $M_u$ reached a 0.00099 (refer to Legend 2 of Figure 20, Legend (2) of Figures 17(a) and 21(a)) in which stresses concentrated by plates were relieved from 0.006 (refer to Figure 15(c)), indicating that a beam depth should be increased to 500 mm to reach a design strength greater than a required moment ($M_u$) at a concrete strain of 0.003. Rates of the strain increase in rebars and steel flanges were similar to those of concrete as shown in Figure 21(b) and (c) where rates of strain increase relative to deflection represented by slopes indicate how fast strains developed as deflections progressed. Rates of strain increase in steel flanges of the 500-mm-deep beam were similar among all beam-column joints, whereas strains in
(a) Strains at mechanical joints of L-shaped column

(b) Punching shear stress and shear strength at a neck of counterbores

(c) Strain at mechanical joints of column-girder joint (Kim 2017)

Figure 15. Strain at $M_o$ (refer to Table 5 for load demands for design) based on average strain; girder is 200 mm wide $\times$ 400 mm deep.
Figure 16. Nominal strength at a concrete strain of 0.003 based on average strain; girder is 200 mm wide × 400 mm deep.
(a) Strain at mechanical joints of column-girder joint at $M_u$

(1) Average compressive concrete strain of 0.003

(2) Strain at mechanical joints of column-girder joint at a concrete strain of 0.003

(b) Strain identification at a concrete strain of 0.003

(c) Strain on a face of concrete column at rebar locations

Figure 17. Nominal strength (based on average strain); girder is 200 mm wide $\times$ 500 mm deep (Kim 2017).
rebars of mechanical beam-column joints for 500-mm-deep beam increased fastest. It is worth noting that the concrete beam can be confined by carbon polymers to sustain higher strains to yield strength, which helps beam to resist larger moment ($M_u$).

2.9. Cyclic energy dissipating capability

The authors investigated the experimental cyclic behavior of three specimens, LC1, LC2, and LC3 which were loaded to failure. The test specimens were fabricated with bolted metal plates, highlighting the use of interior bolts to increase the flexural capacity of precast concrete columns connected via metal plates. The load–displacement relationship modeled with concrete-damaged plasticity for beams and columns,
demonstrating that beam and column sections did not undergo any significant concrete degradation when joints plate absorbed most of the inelastic energy, preventing damages of concrete sections. In a previous study of the authors (Nzabonimpa and Hong 2019), the hysteretic energy dissipation capacity (defined as an area under a hysteresis curve) for monolithic specimen was observed to be similar to the energy dissipated by specimen having a mechanical joint with interior bolts, indicating that use of interior bolts can effectively enhance a ductility of the proposed mechanical joint for columns with irregular shapes to a level similar to that of a monolithic column. However, the hysteretic energy dissipation capacity measured for the specimen with an absence of interior bolts was significantly less than the energy dissipated by specimen having a mechanical joint with interior bolts, indicating that an absence of interior bolts degrades the ductility of the proposed mechanical joint for columns. Inelastic energy dissipation was evaluated based on a 20% load decrease with respect to the load at the maximum load limit state.
Figure 21. Strain evolution of the structural components attached to plates (Kim 2017).
3. Conclusion

This study succeeded in an experimental investigation of the authors’ previous studies where L-shaped dry mechanical joints were developed and tested for their assembling capability of precast concrete frames. In their study, all structural components at the joint experimentally functioned well, resisting both cyclic and monotonic loads. In the present study, microscopic strains of the structural behavior of mechanical joint and structural components attached to L-shaped laminated plates implemented for a 20-story apartment building were numerically explored. Extensive non-linear finite element models were developed, well-describing fractures and strength degradations of spliced columns by mechanical joints. The degradation mechanism of mechanical joints, rebar, and steel sections attached to plates was identified at a concrete strain of 0.003 to find if designs of mechanical joints can avoid brittle failure modes of joints. For that, strain evolution was explored to determine beam depth sufficient to relieve stress concentrations of concrete beam sections exerted by extended beam endplate. Design verification of beam-column frames, design recommendations, and collateral benefits, leading to an understanding of the structural performance of mechanical joints for the design of tall buildings are summarized based on strain analysis.

(1) Resizing based on strains

As shown in Figure 15(c), stress concentration reaching a strain of 0.006 in a beam section was exerted by extended beam endplates when a concrete beam section reached $M_u$. These strains were relieved down to a strain of 0.00099 at $M_u$ (refer to Figure 17(a)), resulting in a sufficient flexural strength to resist $M_u$ when sufficient beam depth with 500 mm was provided. Beam depth must be adjusted until nominal flexural strengths at a concrete strain of 0.003 exceed moment demand $M_u$ when strains of concrete sections of beams exceed a strain of 0.003 at $M_u$, not being able to provide a sufficient flexural strength to resist loads at $M_u$. In the design of the proposed frames, a moment strength of 311 kN·m (corresponding to a concrete strain of 0.003 (refer to Figure 17(b)-(2))) was achieved when the beam was resized to the 200 mm width $\times$ 500 mm depth, resulting in reduced concrete strain. This moment strength was greater than $M_u$ of 238 kN·m.

(2) Joint plate

Tensile loads exerted on metal plates were transferred more effectively when interior bolts were located near the embedded L-shaped steel section. Smaller strains (0.00022) were found at $M_u$ in interior bolts compared to those (0.00024) at exterior bolts (refer to Figure 15(a)). However, large strains (0.00076) in interior bolts that were twice those of exterior bolts were observed, while small strains (0.0034) were computed at exterior bolts at a concrete strain of 0.003 (refer to Figure 16(a)) for girder having 200 mm width $\times$ 400 mm deep). This elucidated that interior bolts started to contribute to the increase of flexural capacity of plate connection as column concrete strains increased. Stains in upper and lower plates that did not yield at a concrete strain of 0.003 were limited to 0.00176 and 0.00124 (refer to Figure 16(a)), respectively, due to use of interior bolts which activate high tensile strains of rebar and steel sections attached to plates. The use of interior bolts with for metal plates led to increased stiffness of the mechanical joint, suggesting an efficient way to obtain a flexural capacity that is as large as possible.

(3) Application to a tall building

Design for a selected 20-story apartment building having multibay frames (Figure 1(a) (Nzabonimpa and Hong 2019) was performed based on an extensive strain analysis. Design for proposed L-type composite precast frames with mechanical joints was validated, by numerical analysis considering concrete-damaged plasticity and elastic-softening properties of headed studs which reflected low cycle fatigue of the metal. The FEA model implemented in this design verification was validated by the previous experimental study (Nzabonimpa and Hong 2019). Design recommendations and collateral benefits were verified for a 20-story tall building application. Extensive strain analysis showed that joints were efficiently designed using interior bolts to greatly enhance the structural performance of mechanical joints. Architectural flexibility at corners similar to that of wall frames was achieved using L-shaped columns while accounting for axial loads.

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References

Alias, A., M. A. Zubir, K. A. Shahid, and A. B. A. Rahman. 2013. "Structural Performance of Grouted Sleeve Connectors with and without Transverse Reinforcement for Precast Concrete Structure." Procedia Engineering 53: 116–123. doi:10.1016/j.proeng.

Belleri, A., M. Torquati, and P. Riva. 2014. "Seismic Performance of Ductile Connections between Precast Beams and Roof Elements." Magazine of Concrete Research 66 (11): 553–562. doi:10.1680/macr.13.00092.

Belleri, A., and P. Riva. 2012. "Seismic Performance and Retrofit of Precast Concrete Grouted Sleeve Connections." D’Aniello, M., D. Cassiano, and R. Landolfi. 2016. "Monotonic and Cyclic Inelastic Tensile Response of European Preloadable GR10. 9 Bolt Assemblies." Journal of Constructional Steel Research 124: 77–90. doi:10.1016/j.jcsr.2016.05.017.

D’Aniello, M., D. Cassiano, and R. Landolfi. 2017. "Simplified Criteria for Finite Element Modelling of European Preloadable Bolts.” Steel and Composite Structures 24 (6): 643–658.

DSS (Dassault Systèmes Simulia Corp). 2014. "ABAQUS Analysis User’s Manual 6.14-2." Providence, RI, USA.

Hong, W. K. 2019. "Hybrid Composite Precast Systems: Numerical Investigation to Construction." Woodhead Publishing 105–109. 9780081027219, Elsevier.

Hu, J. Y., and W. K. Hong. 2017. "Steel Beam–column Joint with Discontinuous Vertical Reinforcing Bars." Journal of Civil Engineering and Management 23 (4): 440–454. doi:10.3846/13923730.2016.1210217.

Kim, J. 2017. "Bolted Assembly of the Precast Structural Frames with Mechanical Joints." Master’s thesis, Kyung Hee University.

Ling, J. H., A. B. A. Rahman, and I. S. Ibrahim. 2014. "Feasibility Study of Grouted Splice Connector under Tensile Load." Construction and Building Materials 50: 530–539. doi:10.1016/j.conbuildmat.2013.10.010.

Nzabonimpa, J. D., W. K. Hong, and J. Kim. 2017a. "Mechanical connections of the precast concrete columns with detachable metal plates..." The Structural Design of Tall and Special Buildings 26 (17): e1391. doi:10.1002/tal.1391

Nguyen, D. H., W. K. Hong, and J. Kim. 2020. "Strain Evolution of L-shaped Precast Columns Spliced by Laminated Metal Plates, Part I." Journal of Asian Architecture and Building Engineering. doi:10.1080/13467581.2020.1869017.

Nzabonimpa, J. D. 2018. “Development of Lego-type Column-to-column Connections.” Ph. D. thesis, Kyung Hee University.

Nzabonimpa, J. D., and W. K. Hong. 2019. "Experimental Investigation of Hybrid Mechanical Joints for L-shaped Columns Replacing Conventional Grouted Sleeve Connections." Engineering Structures 185: 243–277. doi:10.1016/j.engstruct.2019.01.123.

Nzabonimpa, J. D., W. K. Hong, and J. Kim. 2017b. "Nonlinear Finite Element Model for the Novel Mechanical Beam-column Joints of Precast Concrete-based Frames." Computers & Structures 189: 31–48. doi:10.1016/j.compstruc.2017.04.016.

Nzabonimpa, J. D., W. K. Hong, and S. C. Park. 2017. “Experimental Investigation of Dry Mechanical Beam–column Joints for Precast Concrete Based Frames.” The Structural Design of Tall and Special Buildings 26 (1): e1302. doi:10.1002/tal.1302.

Ou, Y. C., P. H. Wang, M. S. Tsai, K. C. Chang, and G. C. Lee. 2010. "Large-scale Experimental Study of Precast Segmental Unbonded Posttensioned Concrete Bridge Columns for Seismic Regions." Journal of Structural Engineering 136 (3): 255–264. doi:10.1061/(ASCE)ST.1943-541X.0000110.

Ozturan, T., S. Ozden, and O. Ertas. 2006. "Ductile Connections in Precast Concrete Moment Resisting Frames." Concrete Construction 9: 11.

Park, R., and T. Paulay. 1975. Reinforced Concrete Structures. New Jersey: John Wiley & Sons.

Proverbs, D. G., G. D. Holt, and P. O. Olomolaiye. 1998. "Factors Impacting Construction Project Duration: A Comparison between France, Germany and the U.K." Building and Environment 34 (2): 197–204. doi:10.1016/S0360-1323(98)00004-3.

Rave-Arango, J. F., C. A. Blandón, J. I. Restrepo, and F. Carmona. 2018. “Seismic Performance of Precast Concrete Column-to-column Lap-spline Connections.” Engineering Structures 172: 687–699. doi:10.1016/j.engstruct.2018.06.049.

Tullini, N., and F. Minghini. 2016. “Grouted Sleeve Connections Used in Precast Reinforced Concrete construction – Experimental Investigation of a Column-to-column Joint." Engineering Structures 127: 784–803. doi:10.1016/j.engstruct.2016.09.021.

Yuan, H., Z. Zhenggeng, C. J. Naito, and Y. Weijian. 2017. “Tensile Behavior of Half Grouted Sleeve Connections: Experimental Study and Analytical Modeling.” Construction and Building Materials 152: 96–104. doi:10.1016/j.conbuildmat.2017.06.154.