Strain evolution of L-shaped precast columns spliced by laminated metal plates, part I

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ABSTRACT
Mechanical joints for both irregular steel-concrete composite precast frames and reinforced concrete precast frames were developed to replace conventional cast-in-place concretes in the previous study of the authors. Novel connections modified from conventional steel joints having fully restrained moment connections offer rapid and facile erections. This study aims to numerically identify microscopic strain evolution and failure modes of the laminated metal plates splicing L-shaped precast composite columns. Numerical parameters including material properties of structural components of mechanical joint were implemented to determine damages and strength degradations with associated fractures. Comprehensive numerical results agreed well with test data, which explored structural behavior and contribution of each structural element of mechanical joint to flexural capacity. Strain evolution showed that degradation of the mechanical joints was retarded by interior bolts to a level similar to monolithic columns, resulting in contribution of mechanical joint to flexural capacity of columns. Influence of axial loads on the behavior of columns with mechanical joints was also evaluated to provide design recommendations.

Figures:
Details of numerical modeling (LC1–LC3)

1. Introduction
1.1. Significance of the study
Conventional precast assembly is based on casting of joints or using couplers that have been traditionally implemented to splice longitudinal reinforcing bars to ensure sufficient bonding between reinforcing bars to transfer moments. Considerable experimental studies (Rave-Arango et al. 2018; Tullini and Minghini 2016; Alias et al. 2013; Yuan et al. 2017; Ling, Rahman, and Ibrahim 2014) have spliced precast column components for rigid connections. Tullini and Minghini (2016) performed full-scale tests on precast reinforced concrete column-to-column connections made with grouted sleeve splices. A considerable amount of numerical work regarding sleeve connectors is also available. The mechanical performance of a grouted splice sleeve (GSS) under axial tension was investigated by a three-dimensional nonlinear finite element model with nonlinear material behavior of the component (Kuang and Zheng 2018).

D’Aniello, Cassiano, and Landolfo (2016) described the results of an experimental investigation devoted to characterize monotonic and cyclic tensile behaviour 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 modelling 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 modelling
approaches were compared to accurately simulate all stages of bolt assembly response, including elastic response, plasticity onset, softening and initiation of failure.

Dry mechanical connections for precast concrete frames were proposed in the previous study of (Hu, Hong, and Park 2017), where mechanical joints for both irregular steel-concrete composite precast frames and reinforced concrete precast frames were developed to replace conventional cast-in-place concretes. An experimental investigation of "L"-shaped precast concrete column joints with laminated mechanical plates was also performed to verify that L-shaped precast columns spliced by laminated metal plates provided fully and partially restrained moment connections (Nzabonimpa and Hong 2019) where brief numerical investigations were presented with limited discussions of microscopic strains to explore damage evolution of structural components of mechanical joints. The present study provided extensive numerical investigations to identify strength degradations and ductility. Strength of the mechanical joints with irregular plates stiffened by interior bolts was predicted based on nonlinear finite element analyses (FEAs) under monotonic loads, exploring failure modes and fractures. This study also explored strain evolutions of each structural element composing mechanical joint based on non-linear finite element models considering concrete-damaged plasticity which led to understanding of flexural capacity and ductility of L-type columns with mechanical joints. Novel connections modified from conventional steel joints having fully restrained moment connections offer rapid and facile erections. The L-shaped sections were preferred by architects due to their architectural flexibility at the corners of the walls in residential buildings because L-shaped column can replace rectangular columns, which do not fit at the corners (Nzabonimpa and Hong 2019).

1.2. Frame with L-shaped columns

L-shaped column system with 3D details is shown in Figure 1 where L-shaped columns are connected to a girder via dry mechanical joints using bolts and end-plates. An assembly test was performed at the site as also shown in Figure 1 to investigate an interconnection of components (columns, girders, bolts, and plates).

![3D drawing](image-url)

![Erection test](image-url)

**Figure 1.** Frame with L-shaped columns.
2. Overview of experimental investigation of L-type columns

2.1. Choice of elements and discretization

In a previous study (Nzabonimpa and Hong 2019), the authors performed a brief finite element (FE) analysis and compared load-displacement relationships with test data obtained from three types of L-shaped columns. In the present study, the authors extended earlier numerical study to explore microscopic strains of mechanical joints of L-shaped columns. FE models were developed based on continuum elements (eight-node linear brick) of type C3D8R, as indicated in Figure 2(a). In a previous study (Nzabonimpa and Hong 2019), the authors modeled Specimen LC1 with 35 mm thick plates without interior bolts, whereas Specimen LC2 was discretized with interior bolts stiffening joints. Coarse meshes were used for monolithic Specimen LC3. In Figure 2(b), mechanical joints including rebars, bolts, metal plates, upper and lower columns, and nuts were discretized using reduced integration elements (C3D8R), whereas a rigid body with a constant monotonous load controlled by a displacement was modeled by linear quadrilateral elements (R3D4). Specimen LC1 was discretized using a total mesh of 736,263 solid elements.

Figure 2. Three-dimensional FE models for the proposed joint.
The largest number of elements was observed in Specimen LC2, in which interior bolts were used to increase flexural capacity of the proposed connections. A total mesh size of 757,996 solid elements was used to model Specimen LC2. Alternatively, Specimen LC3, monolithic column having no plates, was discretized with 251,761 solid elements. Details of numerical modeling (LC1–LC3) are depicted in Figure 2(b). Readers are referred to the previous study (Nzabonimpa and Hong 2019) for the details of test specimens.

2.2. Experimental set-up, loading protocol and lateral load-drift ratio

The experimental setup of the L-shaped columns is illustrated in Figure 3. The L-shaped column was connected to rigid floor through a strong foundation to keep the specimen stable while testing. Lateral cyclic loads controlled by lateral cyclic displacements were exerted at the level of 2700 mm height. These displacements were created by an actuator. Gauges were disposed of all positions on the column to measure strains of the column component (concrete, rebars, steel, plates) and, thus, all test results were recorded in real time during testing.

Loading protocol and lateral load-drift ratio of the specimens were provided as shown in Figure 4.

2.3. Defining interactions: surface-to-surface contact

In Figure 5, two surfaces interacting with each other during an analysis are defined. In ABAQUS, two contact regions must be selected to assign contact pairs. Definition of contact interaction was based on surface-to-surface approach (Dassault Systèmes Simulia Corp. 2014). Figure 5 depicts master and slave surfaces used to study nonlinear behaviors of

| Loading step | Interstory drift Angle (rad) | Number of cycles | Lateral displacement (mm) |
|--------------|-----------------------------|------------------|--------------------------|
| 1            | 0.00250                     | 3                | 6.75                     |
| 2            | 0.00375                     | 3                | 10.2                     |
| 3            | 0.00500                     | 3                | 13.5                     |
| 4            | 0.00750                     | 3                | 20.3                     |
| 5            | 0.01000                     | 3                | 27.0                     |
| 6            | 0.01500                     | 3                | 40.5                     |
| 7            | 0.02000                     | 2                | 54.0                     |
| 8            | 0.03000                     | 2                | 81.0                     |
| 9            | 0.04000                     | 2                | 108.0                    |
| 10           | 0.05000                     | 2                | 135.0                    |
| 11           | 0.06000                     | 2                | 162.0                    |
| 12           | 0.07000                     | 2                | 189.0                    |

Drift angle ‘x’ column height (Loading point: 2700 mm)
The proposed Lego-type columns. Surface-to-surface contact properties were defined based on tangential and normal behaviors. Tangential behavior was defined as frictionless, while normal behavior was assigned based on penalty method. Penalty method considers hard contact following pressure-overclosure relationship in which surface-to-surface contacts are constrained by a penalty stiffness to
enhance solver efficiency (Dassault Systèmes Simulia Corp. 2014). Many users prefer penalty method because it allows minor penetrations to occur between master and slave surfaces, where contact force is proportional to penetration distance.

2.4. Defining constraints and material properties

Defining constraints between reinforcing bars and concrete is a common practice for modeling reinforced concrete members. ABAQUS eliminates translational DOFs of nodes when embedded elements lie within host region. These nodes are referred to as embedded nodes. 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 the present FE models as shown in Figure 6. Material properties for embedded and host elements were defined based on test samples performed during experiments. Average concrete strength (29.1 MPa), average yield strength of L-shaped steels (387.1 MPa), and average tensile yield stress of rebars 646.7 MPa were obtained from test samples (Nzabonimpa and Hong 2019). In Figure 7(a), stress-strain curve for unconfined concrete (Park and Paulay 1975) is adopted to define concrete-damaged plasticity model.19752019201220192019

Figure 7. Numerical investigation validated by the test data (LC1–LC3).

2.5. Microscopic strains of the laminated metal plates provided for the L-shaped precast concrete columns

2.5.1. Modeling headed studs

Elastic-plastic property was assumed for constitutive relationships of embedded elements (L-shaped steels and rebars) whereas elastic-softening behavior representing interior stiffeners was implemented for constitutive relationships of headed studs. Roy et al. (2012) investigated dependency of peak tensile stress on number of cycles and strain amplitude vs. number of cycles (N) is shown in Figure 7(b) where elasto-plastic and elasto-softening behavior of materials under cyclic loading for different strain amplitude was presented. Elasto-softening behavior of steel and rebars was idealized as shown in Figure 7(c) (Nzabonimpa, Hong, and Kim 2017) in which strain hardening model were compared with elasto-softening models. Stress levels were reduced to 0.95 and 0.99f, at the strain of 50 in Figure 7(c) to predict load-displacement relationship of L-type mechanical connection. Low cycle fatigue representing cyclic softening would lead to a continually increasing strain ranges and early fracture as shown in Figure 7(b). Elastic-softening (considering low cycle fatigue) assigned to headed studs decreased flexural behavior of columns after around 80 mm, as shown in Figure 7(d). In Figure 7(d), the peak strength and descending branch of the load-displacement relationship
resulted by fracture of the interior headed studs indicated by Legend 5 are accurately predicted.

2.5.2. Microscopic strains of laminated metal plates

Interior headed studs (LC2-WF) were well activated, reaching a strain of 0.009 (2$\varepsilon_y$) at a concrete strain of 0.003 (refer to Figure 8(b) and Table 1) and a strain of 0.027 (6.3$\varepsilon_y$) at a concrete strain of 0.01 (refer to Table 1), contributing to structural stability. More than twelve times higher strains in interior headed studs were exhibited than those in exterior bolts at a concrete strain of 0.01 as shown in Table 1 which indicates that interior headed studs significantly contributed to flexural capacity of the joint. Notably, much smaller tensile strain was exerted on exterior bolts (0.002, 0.4$\varepsilon_y$) in LC2 than on that (0.028, 6$\varepsilon_y$) in LC1.
Table 1. Strains of the proposed column connection identified in Legends 2 and 5 in Figure 4(d): specimen with the interior bolts (LC2-WF) (Nzabonimp'a and Hong 2019).

| Legends 2 and 5 in Figure 4(d) | FEA results (Legend 5) | Test results (Legend 2) |
|--------------------------------|------------------------|------------------------|
| Load                           | Concrete strain of 0.003 | Concrete strain of 0.01 |
| Displacement                   | 435 kN                  | 596 kN                  | 500 kN |
| Drift ratios (rad)             | 0.00963                 | 0.020                  | 0.037 |
| Concrrete Stress               | 0.003                   | 0.012                  | 0.003 |
| Rebar (Average)                | 0.7ε                  | 0.006                  | 0.0013 |
| Steel (Average)                | 0.002                   | 0.006                  | 0.0012 |
| Upper plate Deformation        | 0.31 mm                 | 0.67 mm                | Not measured |
| Lower plate Deformation        | 0.00138                 | 0.0031                 | 0.0072 |
| Exterior bolt Deformation      | 0.002                   | 0.005                  | Not measured |
| Interior bolts Stress          | 1,015 MPa               | 1,005 MPa              | Not measured |

Figure 9. Activation of the interior bolts of Specimen LC2 at 54 mm; strains in the interior bolts.

(refer to Figure 8(a)) due to interior stiffeners, demonstrating that structural behavior became more stable with reduced strain levels in exterior bolts. Figure 9 shows activation of strain (0.03 (6ε₀)) of interior stiffeners of Specimen LC2 at 54 mm compared to that of exterior bolts (0.005 (1ε₀)). Interior stiffeners reduced strains in the upper and lower column plates to 0.0014 (0.7ε₀) and 0.0003 (0.7ε₀), respectively, from greater strains of 0.006 (3ε₀) and 0.007 (5ε₀), respectively, at a concrete compressive strain of 0.003 (design limit), when interior head studs were not used. These strains led to Figure 8(a) where a small deformation of 0.4 mm from metal plates stiffened with headed studs was found, while metal plate without interior stiffeners experienced a large deformation of 1.5 mm as shown in Figure 8(b). Deflection (26 mm) of Specimen LC2 (with interior stiffeners) became smaller than that (36.4 mm) of Specimen LC1 (without interior stiffeners) at a concrete compressive strain of 0.003 (design limit). As a consequence of interior bolts (0.009, 2ε₀), structural performance of the proposed mechanical joint was significantly enhanced, activating strains (0.002, 1.0ε₀) of steel sections attached to the plates in Specimen LC2 having interior stiffeners more than those (0.0016, 0.8ε₀) in Specimen LC1 having no interior stiffeners. In Figure 10, the observed failure mode of LC3 compared to that obtained by numerical investigation is shown for a stroke of around 108 mm. Failure
Figure 10. Observed failure mode of LC3 compared with that obtained by the numerical investigation for a stroke of around 108 mm.

(a) FEA modeling with load applied at shear center

(b) Flexural strength of FE models with/without foundations (Nzabonimpa 2018)

(c) Load-displacement relationships for Specimens LC1, LC2, and LC3; load applied at shear center and at test center

Figure 11. FE models without foundations (Nzabonimpa 2018).
mode of the lower part of the column was similarly observed in both the experimental and numerical results.

2.6. Strain evolutions of the L-type plates with/without foundations

2.6.1. Modeling test specimens without a foundation

Three FE models (LC1-NF, LC2-NF, and LC3-NF, *NF = no foundation) were constructed without a foundation. Geometric configurations were kept the same relative to the previous models except for the lack of a foundation, as indicated in Figure 11(a). In Figure 11(b), an influence of a foundation on structural performance of the proposed column-to-column connections was explored for six FE models (LC1-NF, LC2-NF, LC3-NF, LC1-WF, LC2-WF, and LC3-WF, *WF = with foundation) for nonlinear finite element analysis when a load was applied at a test center. The largest discrepancy of a deflection between models with/without foundations was observed for monolithic Specimen LC3. Specifically, this is clear by looking at Discrepancy 1 in a load-displacement relationship in Legends 1 and 4 of Figure 11(b), which shows the greatest influence of foundation rotation. However, the least Discrepancy 3 (Legends 3 and 6 of Figure 11(b)), which was caused by a foundation of Specimen LC1 with mechanical joints without interior bolts, was not as prominent as that of monolithic Specimen LC3. This is because metal plates in Specimen LC1 were deformed, making deflection of Specimen LC1 caused by foundation less significant. Specimen LC2, which was stiffened with interior bolts, by which a rigid joint was created, preventing metal plates from deforming to increase flexural capacity of mechanical joint up to the level of the monolithic column. Rotation caused by foundation of Specimen LC2 became more prominent than that of Specimen LC1. Discrepancy 2 (Legends 2 and 5 of Figure 11(b) of Specimen LC2) was larger than Discrepancy 3 (Specimen LC1) without interior bolts, but smaller than that of monolithic specimen.

Table 2. Strains of the proposed column connection identified in Legend 4 of Figure 8(c): monolithic specimen without a foundation (LC3-NF) (Nzaborimpa 2018).

| Location in Figure 8(c) | FEA results |
|-------------------------|-------------|
| FEA results             | Concrete strain of 0.003 | Concrete strain of 0.01 |
| Load at test center     | 958 kN       | 1,277 kN       |
| Displacement            | 31 mm        | 81 mm          |
| Drift ratios (rad)      | 0.011        | 0.03           |
| Concrete                | 0.003        | 0.011          |
| Stress                  | 33 MPa        | 7.0 MPa        |
| Rebar                   | 0.0086       | 0.027          |
| (Average)               | (2.7ε)       | (9ε)           |
| Stress                  | 647 MPa       | 583 MPa        |
| Steel                   | 0.0096       | 0.03           |
| (Average)               | (2.7ε)       | (9ε)           |
| Stress                  | 413 MPa       | 582 MPa        |

Table 3. Strains of the proposed column connection identified in Legend 5 in Figure 8(c): specimen with interior bolts (LC2-NF) (Nzaborimpa 2018).

| Location in Figure 8(c) | FEA results |
|-------------------------|-------------|
| FEA results             | Concrete strain of 0.003 | Concrete strain of 0.01 |
| Load at test center     | 697 kN       | 626 kN       |
| Displacement            | 32.4 mm      | 50 mm        |
| Drift ratios (rad)      | 0.012        | 0.019        |
| Concrete                | 0.003        | 0.010        |
| Stress                  | 32 MPa        | 67.6 MPa     |
| Rebar                   | 0.0076       | 0.014        |
| (Average)               | (9ε)         | (4.7ε)       |
| Stress                  | 670 MPa       | 682 MPa       |
| Steel                   | 0.0029       | 0.0052       |
| (Average)               | (1.5ε)       | (2.7ε)       |
| Stress                  | 440 MPa       | 518 MPa       |
| Upper plate             | 0.0042       | 0.0071       |
| Deformation             | (2.2ε)       | (2.2ε)       |
| Stress                  | 439 MPa       | 430 MPa       |
| Lower plate             | 0.0044       | 0.0076       |
| Stress                  | (2.2ε)       | (2.2ε)       |
| Deformation             | 0.8 mm        | 3.5 mm        |
| Stress                  | 440 MPa       | 430 MPa       |
| Exterior bolts          | 0.019        | 0.068        |
| Stress                  | (4ε)         | (14ε)        |
| Deformation             | 0.14         | 0.37         |
| Stress                  | (28ε)        | (74ε)        |
| Stress                  | 531 MPa       | 355 MPa       |

Table 4. Strains of the proposed column connection identified in Legend 6 of Figure 8(c): specimen without interior bolts (LC1-NF) (Nzaborimpa 2018).

| Location in Figure 8(c) | FEA results |
|-------------------------|-------------|
| FEA results             | Concrete strain of 0.003 | Concrete strain of 0.01 |
| Load at test center     | 480 kN       | 532 kN       |
| Displacement            | 30 mm        | 42 mm        |
| Drift ratios (rad)      | 0.011        | 0.016        |
| Concrete                | 0.003        | 0.010        |
| Stress                  | 31 MPa        | 9.8 MPa      |
| Rebar                   | 0.004        | 0.006        |
| (Average)               | (74ε)        | (2ε)         |
| Stress                  | 670 MPa       | 671 MPa       |
| Steel                   | 0.0024       | 0.0043       |
| (Average)               | (13.2ε)     | (2.3ε)       |
| Stress                  | 358 MPa       | 388 MPa       |
| Upper plate             | 0.011        | 0.018        |
| Stress                  | (6ε)         | (9ε)         |
| Deformation             | 453 MPa       | 461 MPa       |
| Stress                  | 453 MPa       | 461 MPa       |
| Lower plate             | 0.012        | 0.019        |
| Stress                  | (6ε)         | (9ε)         |
| Deformation             | 2 mm         | 4.9 mm       |
| Stress                  | 455 MPa       | 464 MPa       |
| Exterior bolts          | 0.035        | 0.077        |
| Stress                  | (7ε)         | (15ε)        |
| Stress                  | 1,051 MPa     | 888 MPa       |

2.6.2. Location of a load application

In Figure 11(c), an influence of where a load is applied on flexural capacity was explored in the three FE models (LC1-NF, LC2-NF, and LC3-NF) when lateral loads were applied at a shear center. Loads corresponding to
a concrete strain of 0.003 are indicated in the load-displacement relationship. Lateral load was applied at a shear center in the new numerical model to study the influence of load locations that are applied to columns on flexural capacity of the proposed mechanical joint. In Figure 11(c), a smaller flexural capacity was observed compared to when loads were applied at a test center where torsional modes in addition to flexural mode were produced, resulting in a greater flexural capacity accordingly. Tables 2 to 4 demonstrate structural performance of these specimens, summarizing activation of each structural element and its contribution to flexural capacity of the proposed mechanical joint when lateral loads were applied at a test center. Increased activation of column rebars and steel attached to column plate was observed with presence of interior bolts in Specimen LC2-NF than in specimens without interior bolts, while substantially reducing deformation of metal plates. At a concrete strain of 0.003, a rebar strain of 0.0076 in Specimen LC2-NF is greater than a strain of 0.004 in Specimen LC1-NF without interior stiffeners. However, as shown in Tables 3 and 4, metal plates were more deformed in specimens in which interior bolts were absent, resulting in smaller strains of L-shaped steel sections attached to plates of Specimen LC1-NF than those of Specimen LC2-NF with interior bolts.

2.6.3. Strain evolution of L-type columns (monolithic and mechanical joints with no axial force) with/without a foundation

Strain evolutions of rebars, steels, and metal plates in Specimens LC1-NF, LC2-NF, and LC3-NF without a foundation are illustrated in Figure 12. Metal plates of column stiffened with the interior bolts led to full activation of rebars attached to metal plates, whereas less activation of rebars was observed due to large the deformation of metal plates of column without interior bolts (LC1-NF). Rebars and steel column were activated more in mechanical joint reinforced with interior stiffeners than in mechanical joint without interior bolts. Rebars were activated similarly to those of monolithically designed column, as can be seen in Figure 12(a). It was experimentally and numerically shown that large deformations were exhibited at unstiffened metal plates, causing a rapid rate of strain increase (refer to Legend 1 of Figure 12(c,d); Legends 1 and 3 of Figure 13(c,d) without foundation). However, interior
stiffeners play an important role in increasing flexural strength of the proposed mechanical joints for precast concrete columns (refer to Legend 2 of Figure 12(c,d); Legends 2 and 4 of Figure 13(c,d) with foundations). Rebars and steels attached to the plates of Column LC2-WF (Legend 2 of Figure 13(a,b)) are more activated than those of Column LC1-WF (Legend 3 of Figure 13(a,b)). Activation of strains of both steels and rebars for specimen stiffened with interior stiffeners can be triggered similarly to that of monolithic Specimen LC3 at design level. Activation of steels and rebars became retarded (refer to Legend 3 of Figure 13(a,b)) due to large deformations which occurred in metal plates without interior stiffeners (refer to Legends 1 and 3 of Figure 13(c,d)). As indicated in Legend 1 of Figure 13(a, b), the largest activation of rebars and steels is observed with monolithic specimen. Deformation of metal plates of the column without interior stiffeners was initiated at a stroke of 81 mm (refer to Figure 13(e)). Nonlinear finite element analysis considering damaged concrete plasticity exhibited discrepancies with test data, to some extent. However, a similar
trend of strain evolution between the two studies was observed, as depicted in Figure 13(c, d).

3. Influence of axial loads on behavior of mechanical joints

3.1. Pre-stressing effect offered by initial axial loads on flexural capacity of columns

Microscopic strains in irregular hybrid mechanical joints were presented based on non-linear finite element analysis considering concrete-damaged plasticity (Figure 14(a)). Rate of strain increase of structural components was then calculated as lateral load progressed, contributing to the understanding of structural behavior of beam-column frames with L-type mechanical joints. Flexural capacities of the columns with 5,000 kN axial loads (refer to Curves 8, 5, and 2 in Figure 14(b)) are greater than those with 10,000 kN axial loads (refer to Curves 7, 4, and 1 in Figure 14(b)) because of large axial compressive strains (refer to Table 5) exerted by initial axial loads. Flexural capacities of the columns with the 5,000 kN axial loads are also greater than those with no axial loads (refer to Curves 9, 6, and 3 in Figure 14(b)) due to the pre-stressing effects. Pre-existing axial strains can contribute to flexural capacity of columns when axial loads are small or moderate. Large compressive strains (refer to Table 5) which pre-exited in columns result in concrete strains, quickly reaching design limit state corresponding to a strain of 0.003. As shown in Table 5, 28%, 23%, and 24% of axial strains (0.003) at design limit state pre-exist in columns when...
influence of axial loads on interior bolts

strains evolution shows that degradation of mechanical joints was retarded by interior bolts to a level similar to monolithic steel-concrete column, resulting in contribution of structural elements composing mechanical joint to flexural capacity of columns when axial loads are moderate. however, the stiffness degradations of joint plates were observed with mechanical joints having no interior bolts when no axial loads were exerted (refer to curve 3 of figure 14(b) and curve 3 of figure 14). flexural capacity with mechanical joints having no interior bolts similar to that of monolithic column was identified when a 10,000 kN axial load was applied, as shown in curves 7 and 1 in figure 14(b), whereas flexural capacity of mechanical joints with interior bolts (refer to curve 4 in figure 14(b)) under a 10,000 kN axial load is less than that with mechanical joints without interior bolts. this is because that pre-exiting compressive strains of columns with joint plates having interior stiffeners result in concrete strains quickly reaching ultimate limit state as shown in table 5 when axial loads are substantially large, whereas pre-existing axial strains can efficiently contribute to stiffness of joint plates when axial loads are small or moderate as shown with curves 5, 2 in figure 14(b) and curves 6, 3 in figure 14(b).

3.3. Rate of strain increase at concrete section with axial loads

in figure 15, rate of strain increase of concrete sections with 10,000 kN axial loads rapidly increased from 8 mm as shown in curve 4, and from 11 mm as shown in curves 7 and 1 while that with the 5,000 kN axial loads started to increase from 19 mm as shown in curves 2, 5 and 8. strain rate increases at around 20 mm when no axial loads were applied (refer to as shown in curves 3, 6 and 9). it is noted that strain rate with 10,000 kN axial loads increases earlier than columns with less axial loads.

![Stroke-strain curve (concrete)](image)

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**Table 5.** Average compressive concrete strains at the end of the application of the axial loads (percentage of a strain of 0.003) (Kim 2017).

| Column width of 200 mm Dilation angle of 30°, full damage | Axial load (kN) | Average concrete strains at the end of the application of axial loads | Percentage of a strain of 0.003 | \( M_{\text{u}} \) (kN·m) |
|---|---|---|---|---|
| Mechanical joint without interior stiffeners | 10,000 | 0.00072 | 24% | 1088 |
| 5,000 | 0.00027 | 9% | 1232 |
| 0 | - | - | 868 |
| Mechanical joint with interior stiffeners | 10,000 | 0.00069 | 23% | 898 |
| 5,000 | 0.0003 | 10% | 1342 |
| 0 | - | - | 1170 |
| Monolithic joints | 10,000 | 0.00083 | 27.7% | 1105 |
| 5,000 | 0.00037 | 12.3% | 1420 |
| 0 | - | - | 1340 |

axial loads of 10,000, 5,000, and 0 kN, respectively, are applied before lateral loads are exerted. the lower the axial load exerted, the more the ductile behavior is demonstrated, as shown in curves 9, 6, and 3 (10,000 kN), curves 8, 5, and 2 (5,000 kN), and curves 7, 4, and 1 (0 kN) in figure 14(b). flexural capacities of monolithic specimen were the largest followed by that with mechanical joints with and without interior bolts when 5,000 and 0 kN axial loads were applied, as shown in curves 8, 5, and 2, and curves 9, 6, and 3, where concrete strains of 0.003 are indicated by the red dots.

**Figure 15.** Rate of strain increase at the concrete section of all columns (Kim 2017).
due to stress concentration which occurred in plates. Strains which pre-occurred in column due to gravity loads initiated an early strain increase. Strain increases of column concrete denoted by Curves 7 and 1 are similar to moment-stroke relationships as explained by Curves 7 and 1 in Figure 14(b) when interior bolts are absent. However, Curves 5 and 8 show that interior stiffeners make joint plates more efficient, bringing flexural capacity to a level of monolithic columns when moderate axial loads (5000 kN) are applied. Column with a load of 10,000 kN reached design limit state (concrete strain of 0.003) in the vicinity of 13 mm (refer to Curves 1, 4 and 7) as indicated by red dots which is much earlier than columns with an axial load of 5,000 kN (refer to Curves 2, 5 and 8) and no axial load (refer to Curves 3, 6 and 9) reaching a design limit at 25–30 mm. Rates of strain increases of columns with 5,000 kN axial loads shown in Curves 2, 5, and 8 are similar to those of columns with no axial loads shown in Curves 3, 6, and 9. However, rates of strain increase of all three columns (monolithic columns and columns having mechanical joints with and without the interior stiffeners) under 10,000 kN show stroke at which strain rates increase earlier than columns under 0 and 5,000 kN axial loads. Much higher ductility is expected with columns with lower axial loads.

3.4. Rate of strain increase at rebars with axial loads

As shown in Curves 1 (0.0005), 4 (0.0004), and 7 (0.0008) of Figure 16, flexural design limit state was reached quickly at small tensile rebar strains (0.0004–0.0008) or at a deflection of 12–13 mm due to large initial axial strains (initial axial strain of 0.00075, as shown in Table 5) when a 10,000 kN axial load was applied.

Flexural design limit state is reached at larger tensile rebar strains (0.00278–0.00344) due to reduced initial axial strain of 0.00031 (refer to Table 5) when 5,000 kN axial loads are exerted on columns, as shown in Curves 2 (0.00344), 5 (0.00278), and 8 (0.00268).

Rebars in tension benefit from initial compressive strains occurred due to pre-stressing effects of columns under large axial loads whereas large initial compressive strains lead to early fracture of concrete section in compression. Curve 3 shows that flexural design limit state with no axial loads is reached at largest tensile rebar strain of 0.00596 when no interior stiffeners are provided. Rates of strain increase of tensile rebars without interior stiffeners similar to those with interior stiffeners and monolithic joints are observed when axial loads of 10,000 kN are applied, as shown in Curves 1, 4, and 7. However, for moderate axial loads (5,000 kN or no axial load), rates of strain increase of tensile rebars without interior stiffeners are different from those with interior stiffeners and with monolithic joints. Rates similar to those of monolithic joints are demonstrated for all axial loads when interior stiffeners are installed, whereas strain increase rates of rebars with no interior stiffeners increase rapidly for small axial loads.

3.5. Rate of strain increase at steel sections with axial loads

Pre-stressing effect was also observed with steel sections, as shown with Curves 1, 4, and 7 of Figure 17 where flexural design limit state was reached quickly at small tensile steel strains (0.000296–0.000566) or at deflections

![Figure 16. Rate of strain increase at the rebars of all columns (Kim 2017).](image-url)
of 12–13 mm due to large initial axial strains (average initial strain of 0.00075, as shown in Table 5) when 10,000 kN axial loads were applied. However, flexural design limit state was reached at deflections of 27–32 mm or at large tensile steel strains (0.0024–0.0026) due to reduced average initial strain of 0.00031 (refer to Table 5) when 5,000 kN axial loads were exerted on columns, as shown in Curves 2, 5, and 8. Curve 3 shows that flexural design limit state was reached at largest tensile steel strain of 0.0044 when no interior stiffeners were provided with no axial loads. Rates of tensile steel strain increase of all three types of the joints (with/without interior stiffeners and monolithic joint) with the 10,000 kN axial loads are similar to one another up to design limit state (12–13 mm), as shown in Curves 7, 4, and 1. Similarly for 5,000 kN axial loads, rates of steel strain increase of all three types of joints (with/without interior stiffeners and monolithic joint) similar to one another up to design limit state (26–30 mm) were obtained, as shown in Curves 2, 5, and 8 of Figure 17(b). This indicates, again, that interior
stiffeners are not necessary for activation of steel sections when large axial loads are applied. When axial loads are not applied, the largest steel strain of 0.0044 is reached at design limit state when interior stiffeners are not provided, as shown in Curve 3. Rates without interior stiffeners increased from a stroke of 26 mm (Curve 2 of Figure 17(b)) when a 5,000 kN axial load was applied whereas rate increased from deflection of 16 mm when no axial load was applied (Curve 3 of Figure 17(a)), indicating how axial loads affect structural behavior of mechanical plates, eliciting that interior stiffeners are necessary when moderate axial loads are applied. Notably, flexural design limit state for rebars was reached at higher strains (0.002–0.006) than those for steel sections, indicating that rebars were activated more than steel sections (0.002–0.0045).

3.6. Rate of strain increase at exterior bolts with axial loads

In Curves 4 and 1 of Figure 18, rates of tensile strain increase of exterior bolts with/without interior stiffeners with 10,000 kN axial loads show similar trends shown in Curves 5 and 2 with 5,000 kN axial loads, except for a slight increase at a stroke of 11 mm. When no axial loads are applied, rate of strain increase of exterior bolts (without interior
stiffeners) shown in Curve 3 is much greater than rate with interior stiffeners shown in Curve 6, where tensile strains of 0.022 and 0.00481 were obtained, respectively, at design limit state. Rates with a substantial difference indicate that interior stiffeners are required when small axial loads are applied. Tensile strain of exterior bolts remained at 0.000891 (Curves 1 and 4) regardless of an installation of interior bolts due to initial compressive strains exerted by axial load of 10,000 kN axial load. However, tensile strains of exterior bolts increase rapidly without interior stiffeners when axial loads on columns decrease as shown in Curves 3, 2, 1 of Figure 18.

3.7. Rate of strain increase at interior bolts with axial loads

As shown in Curves 4 (10,000 kN axial loads) and 5 and (5,000 kN axial loads) of Figure 19, rates of tensile strain increase at interior bolts with axial loads. However, tensile strains of exterior bolts increase rapidly without interior stiffeners when axial loads on columns decrease as shown in Curves 3, 2, 1 of Figure 18.

**Figure 20.** Rate of the strain increase at the lower plate of all columns (Kim 2017).

**Figure 21.** Rate of the strain increase at the upper plate of all columns (Kim 2017).
increase of interior bolts do not show rapid changes, reaching strains less than 0.000761 while those (refer to Curve 6 for no axial load) with no axial loads demonstrate rapid changes, reaching a strain of 0.084, requiring installation of interior stiffeners. Rates of tensile strain increase of interior bolts resemble those of exterior bolts calculated without interior stiffeners (refer to Curves 1, 2 and 3 of Figure 18).

3.8. Rate of strain increase at plates with axial loads

Tendency of tensile strain increase of plates relative to deflections of columns is shown in Figures 20 and 21. Rates of strain increase develop rapidly for small axial loads when interior stiffeners are not provided (refer to Curves 3 and 2 of Figure 20) while slower increases of strain rates are observed with large axial loads (refer to Curve 1 of Figure 20). However, rates of strain increase for columns having interior stiffeners develop slowly for all axial loads when interior stiffeners are provided (refer to Curves 6, 5 and 4 of Figure 20). Strain levels of 0.00287 (refer to Curve 3 of Figure 20) and 0.00269 (refer to Curve 3 of Figure 20) were reached for lower and upper plates without interior stiffeners, respectively, when no axial load was applied, whereas strains were much smaller which is less than 0.001 when interior stiffeners were installed (refer to Curve 6 of Figure 20 and Curve 6 of Figure 21, respectively). Plate uplift relative to lateral deflections of columns shown in Curves 1 and 4 of Figures 20 and 21 is not noticeable, only demonstrating a value of 0.000226 at lower plate (Curve 1 of Figure 20) and 0.0000843 at upper plate (Curve 1 of Figure 21) even when interior stiffeners are not installed. This behavior occurs because large axial loads (10,000 kN) suppress plates, preventing plates from deforming upwards. Tensile strains due to uplift of plates increase linearly when an axial load of 5,000 kN was applied, as shown in Curves 2 and 5 of Figures 20 and 21 in which strains of 0.00156 (Curve 2 of Figure 20) and 0.00170 (Curve 2 of Figure 21) were reached by lower plate and upper plate, respectively, when interior stiffeners were not installed. Differences in rates of strain increase between columns with and without interior stiffeners become noticeable as axial loads on columns decrease, demonstrating that there is a significant discrepancy, as shown in Curves 3 and 6 of Figures 20 and 21. Interior stiffeners contribute more to flexural stiffness when magnitude of axial load decreases.

4. Conclusions

It was shown, from the column test with L shape, use of thicker plates can contribute to increasing stiffness of the proposed irregular mechanical joint, helping create a rigid joint, however, seeking flexural capacity similar to that of monolithic columns through plate thickness was not a practical option. The structural behavior of mechanical joints used for L-shaped precast steel-concrete composite columns was numerically characterized by investigating microscopic strains and strain evolutions of structural component elements. Design verification of L-shaped column frames, design recommendations, and collateral benefits based on strain analysis are summarized below.

4.1. Without axial loads

Flexural strength of irregular columns spliced with mechanical plates can be improved to levels similar to those of monolithic steel-concrete columns when interior bolts are additionally used. Load-displacement relationships identified from joint having interior bolts (LC2-WF) are shown in Legends 2 and 5 in Figure 7(d) as well as Table 1 (Nzabonimpa and Hong 2019).

Structural performance of mechanical joint was greatly enhanced by interior bolts at a concrete compressive strain of 0.003 (Nzabonimpa and Hong 2019). Tensile strains observed at metal plates without interior stiffeners (3εp and 5εp) at the upper and lower plates, respectively, refer to Figure 8(a) (Nzabonimpa and Hong 2019) for Specimen LC1 were greater than those for Specimen LC2 (0.7εp) at both the upper and lower plates with interior stiffeners, refer to Figure 8(b) (Nzabonimpa and Hong 2019) during an application of a lateral load. Metal plate stiffened with interior bolts exhibited a smaller deformation of 0.4 mm compared to deformation of 1.5 mm observed for metal plate without interior bolts. Laminated plates interconnected by high-strength bolts offered a partially or fully restrained moment connection depending on stiffness of metal plates which activated rebars and steel sections attached to metal plates, accordingly, contributing to flexural capacity of spliced columns. Low rebar tensile strains of between 0.001 and 0.0024 (0.8εp) in rebars attached to metal plates without interior bolts were reached whereas higher rebar tensile strains reached as much as 0.0068 in monolithic specimen. High tensile rebar strains were also reached by use of interior bolts, leading to increased stiffness of mechanical joint, and recommending an efficient way to obtain a flexural capacity as large as possible. In Figure 8(b), it was observed that tensile strains of lower plates were substantially reduced to 0.0013 (0.7εp) in specimens with interior bolts while those without interior bolts demonstrated large tensile strains of 0.007 (5εp) as shown in Figure 8(a). It is also demonstrated in Figure 8(a)
that strains at exterior bolts (0.028, 6ey) decreased significantly to 0.002 (0.4ey) when internal bolts were added (Figure 8(b)). Steel section attached to plates in Specimen LC2 was more activated than in Specimen LC1, in which interior bolts were absent. Specimen LC2, which had interior bolts, exhibited a greater flexural capacity than Specimen LC1.

4.2. Influence of the axial loads on the interior bolts

L-shape columns behave quite differently when axial loads pre-exited. Extensive non-linear finite element models were developed and calibrated using experimental data, identifying influence of pre-exiting compressive strains of L-shaped columns on fractures and strength degradations of structural elements composing mechanical joint when axial loads are substantially large. Structural behavior of laminated plate connections and structural elements attached to plates including concrete section, rebars, steel sections, exterior bolts, and interior bolts were explored with design recommendations when axial loads were imposed. Failure modes of mechanical joints made of irregular laminated plates were identified and compared with measured test data, identifying how axial loads and interior bolts influenced ductility and strength of mechanical joints when columns were under axial loads. Pre-exiting compressive strains of columns with joint plates having interior stiffeners result in concrete strains quickly reaching ultimate limit state when axial loads are substantially large, whereas pre-existing axial strains can efficiently contribute to stiffness of joint plates when axial loads are small or moderate. Rates of tensile strain increase of interior bolts do not show rapid changes, when considerable axial loads exist whereas those with no axial loads show rapid changes in interior stiffeners, recommending a use of interior stiffeners when axial loads are insignificant.

4.3. Construction benefits

It can be most benefitted from this study that conventional use of grouted sleeve for irregular precast connections can be replaced by mechanical joints proposed in this study, contributing to rapid erection of precast concrete-based frames of irregular shapes (Hong 2019).

Disclosure statement

The authors declare that they have no conflict of interest.

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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.2013.02.017.

D’Aniello, M., D. Cassiano, and R. Landolfio. 2016. “Monotonic and Cyclic Inelastic Tensile Response of European Pre-loadable 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. Landolfio. 2017. “Simplified Criteria for Finite Element Modelling of European Pre-loadable Bolts.” Steel and Composite Structures 24 (6): 643–658. doi:10.12989/scs.2017.24.6.643.

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, 105–109. United Kingdom: Woodhead Publishing, Elsevier.

Hu, J. Y., W. K. Hong, and S. C. Park. 2017. “Experimental Investigation of Precast Concrete Based Dry Mechanical Column–column Joints for Precast Concrete Frames.” The Structural Design of Tall and Special Buildings 26 (5): e1337. doi:10.1002/tal.1337.

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

Kuang, Z., and G. Zheng. 2018. “Computational and Experimental Mechanical Modelling of a Composite Grouted Splice Sleeve Connector System.” Materials 11 (2): 306. doi:10.3390/ma11020306.

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. 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. 2017. “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.

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

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

Roy, S. C., S. Goyal, R. Sandhya, and S. K. Ray. 2012. “Low Cycle Fatigue Life Prediction of 316 L (N) Stainless Steel Based on Cyclic Elasto-plastic Response.” Nuclear Engineering and Design 253: 219–225. doi:10.1016/j.nucengdes.2012.08.024.

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. Weiijian. 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.