A novel erection technique of the L-shaped precast frames utilizing laminated metal plates

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ABSTRACT
This study aimed to demonstrate an erection of the irregular L-shaped precast frames utilizing rigid mechanical joints. Precast columns of L-shaped sections were introduced to replace rectangular columns, which do not fit inside corners; sections are preferred by architects because of their architectural flexibility at the corners of walls. The mechanical connections of steel-concrete composite precast columns were developed using irregular shaped pair of steel plates; each plate was installed at the bottom of the upper columns and on the top of the lower columns. These were then inter-connected via bolts, providing a flexural capacity that was similar to that of conventional monolithic column connections. Column rebars were spliced through metal plates. The columns and beams were also assembled into frames by extended endplates, which were, subsequently, prefabricated with precast beams. An extensive erection test of a full-scale precast frame was performed, demonstrating the efficient and effortless erection of the proposed method. The construction time compared with conventional monolithic assembly was substantially reduced, leading to a corresponding reduction in construction costs. Numerical investigation of the proposed frames was also conducted to verify the structural performance, demonstrating predictable and stable nonlinear structural behavior of the mechanical joints.

1. Introduction

1.1. Previous related studies and significance of this study

Precast concrete components were preferable to conventional cast-in-place concretes since they were cost-efficient structural systems that offered rapid and facile erection. Previous researchers (Guan et al. 2016; Proverbs, Holt, and Olomolaiye 1999; Elliott 2016; Henin and Morcous 2015) contributed to the development of precast concrete members. It was concluded that the use of prefabricated concrete components was preferred in order to reduce both the construction periods and the overall construction cost. Some researchers recently proposed fully restrained moment joints for column-to-column (Elliott 2016; Turan, Ozden, and Ertas 2006; Ou et al. 2009). The authors, in their previous study, proposed the interlocked mechanical joints to assemble precast concrete frames (Nzabonimpa and Hong 2018). The interlocking mechanical joints with fully restrained moment connections were developed to provide rapid and easy
connections for reinforced concrete precast columns. Columns were spliced via vertical column rebars, providing flexural capacity similar to that of conventional monolithic column connections. In the present study, conventional steel joints with fully restrained moment connections were modified to be implemented in mechanical joints for both irregular steel-concrete composite precast frames and reinforced concrete precast frames, as shown in Figure 1. The authors performed full scale test assembly of precast frames utilizing irregular-shaped laminated metal plates. The joints introduced by the authors for beam-to-column joints (Nzabonimpa, Hong, and Park 2017a; Nzabonimpa, Hong, and Kim 2017b) and column-to-column joints (Nzabonimpa, Hong, and Kim 2017c; Hu, Hong, and Park 2017) eliminated the use of concrete pour forms at joints which have typically been used in conventional concrete practices. These joints did not require concrete to be cast at the joints. Joints of these studies were able to transfer moments, creating fully rigid moment connections that could withstand seismic loads (Nzabonimpa, Hong, and Park 2017a;)

![Figure 1. The column and beam-column connection details proposed in this study.]
Nzabonimpa, Hong, and Kim 2017b, 2017c; Hu, Hong, and Park 2017).

Nonlinear finite element analyses of mechanical column-to-column joints with metal filler plates under static loadings were performed by (Nzabonimpa and Hong 2018b) to determine their failure modes and deformations in the column plates. Damages to the rigid mechanical connections for composite precast columns using metal plates were identified. The FEA parameters for the concrete damaged plasticity that were suitable for the prediction of the proposed precast columns were also explored. (Nzabonimpa, Hong, and Kim 2018) performed nonlinear numerical investigation to understand the structural behavior of the mechanical connections with laminated steel and concrete filler plates which transfer axial loads and moments across the column joints for precast concrete frames. They identified a rate of strain increase of the mechanical beam-column joints consisting of extended beam endplates, rebar, and steel flanges.

Conventionally, cast-in-place concrete and sleeve connections have been widely used for the construction of precast concrete members (Elliott 2016; Turan, Ozden, and Ertas 2006; Ou et al. 2009; Johnson 1969). A significant number of studies were performed to ameliorate the application of precast members (PCI committee on Parking structures 2014; Choi, Choi, and Choi 2013). Some of these studies suggested that temporary connections should be provided to stabilize partially completed structures. Researchers suggested that temporary connection details (PCI committee on Parking structures 2014) should be carefully considered. Conventional practices used diagonal cables to brace columns, stairs, and elevator shafts to stabilize the structure during the erection of precast members. However, some of these practices seemed to obstruct the ongoing erection process.

It was also reported that the erection process depended on the size and shape of the members, the joint design, and the complexity of the overall structure (Seeley 1996; Martin and Perry 2004; Elliott and Jolly 2013; Panganti 2018). In the present study, a mechanical joint with metal plates was introduced to assemblage irregular columns (L-type, see Figure 1), replacing conventional precast concrete frames. Importantly, the L-shaped column was intended to replace rectangular columns, which do not fit at the corners (Hong 2019). The L-shaped sections were preferred by architects due to their architectural flexibility at the corners of the walls in residential buildings. The joints with bolted metal plates reduced the construction time by eliminating the time required to cure concrete at the joints. However, the application of conventional precast joints required the use of cast concrete. The test erection of a full-scale precast frame that was assembled using irregularly shaped bolted metal plates demonstrated efficient and rapid assembly. The assembly time decreased substantially when the use of pour forms and curing times (which are required for conventional concrete frames) were eliminated. The L-shaped bolted mechanical plates also represented a cost-saving alternative to conventional monolithic cast-in-place joints. The proposed assembly method was also proven to significantly reduce the time required to assemble precast frames; this time was similar to that needed to assemble steel frames (Hong 2019).

2. Installation mechanism

2.1. Manufacturing of the frames (columns and girders) with the proposed joints

The conventional monolithic cast-in-place joints used to assemble precast concrete frames can be replaced by the precast frame having mechanical joints when a pair of column plates with high-strength bolts was used to splice precast concrete columns with an L shape, as illustrated in Figure 1(a). The spliced precast concrete column with an irregular shape transfers axial loads and moments throughout the joints. The joint of the L-shaped columns consisted of two endplates (lower and upper column plates), nuts, and high-strength bolts. For the full-scale erection test of this study, the steel-concrete columns were manufactured with 35-mm-thick column plates (upper and lower) that had metal filler plates between them. The nuts connecting column rebars to metal plates were completely anchored and hidden in counterbores, as shown in Figure 2(a). The axial loads and the moments were similar to the nominal capacity offered by a monolithic steel-concrete composite column, and were directly transferred by the column plates.

2.2. Joint details for erections

The joint details of the specimens for the test erection were developed for the column connections based on the geometry described in Figure 1(a). Here, the proposed mechanical joint was designed to fully transfer moments through the inter-connected components; this can be applied for both precast steel-concrete composite frames and precast concrete frames. The joint of the proposed connection consisted of two endplates (lower and upper column plates), nuts, and high-strength bolts used to transfer moments through both the lower and upper column plates. Columns were fabricated with 35-mm-thick column plates with a metal filler plate between them, as shown in Figure 1(b) in which 25-mm vertical rebars were used. Three columns were manufactured for the erection test, including two columns with mechanical joints and one monolithically cured column (refer to Figure 1(a)). Nuts were incorporated to connect the threaded end of
the vertical reinforcing bars at the rear part of the endplates, as illustrated in Figure 2(a). Nuts were threaded onto the rebar ends, which were located in the counterbores in the plates; these were thick enough to completely accommodate the nuts while also offering anchorage for the nuts. The steel sections shown in Figure 2(b) were encased in concrete, making steel-concrete hybrid composite precast column sections. Figure 2(b) also exhibits the full-scale columns used for the assembly test, demonstrating the efficiency of assembling of precast frames compared to that of the conventional precast construction. In this study, fully restrained moment steel joints were modified for the use in mechanical joints for both steel-concrete composite precast frames and reinforced concrete precast frames, as shown in Figure 1(b–1,2). The primary application of the extended endplates shown in Figure 1(b–1) was to provide a fully restrained moment capacity between the column-to-beam joint. Conventional extended endplates of the steel structures were used for transferring moments. The horizontal beam rebars at the top were directly connected to columns by couplers embedded in the column face, and not by the extended endplate, as shown in Figure 1(b–1). The

Figure 2. Manufacturing columns.
axial loads and moment were directly transferred across the column-to-beam joint assembly. The locations of the vertical and horizontal couplers were precisely placed during fabrication of this assembly.

2.3. Assembly using mechanical joints

A significant number of experimental and numerical studies have been performed to examine the behavior of extended endplate connections for steel structures that were subjected to monotonic and cyclic loads (Tahir and Hussein 2008; Mureșan and Bălc 2017; Ismail et al. 2016; Sumner and Murray 2002; Sofas, Kalfas, and Pachoumis 2014). These studies showed that steel connections can act as either fully rigid or semi-rigid connections, depending on the endplate thickness, bolt diameter, number of bolt rows and columns, bolt spacing, bolt grade, stiffeners, column and beam sizes, and yield strength of the steel. In this study, mechanical joints with fully restrained moment connections (which were developed to provide rapid and facile connections for composite precast columns) were employed. Here, a pair of steel plates was connected, offering monolithic column joints. Extensive full-scale assembly testing of the large columns that were designed to resist gravity and lateral loads, was performed to investigate how efficiently the vertical columns can form monolithic joints. The observed time required for this assembly using mechanical joints was less than 30 min. The proposed method can be used as an alternative for modular offsite construction for buildings and industrial plants subjected to heavy loads, thereby reducing the cost of construction compared to using steel structures.

2.4. Erection test for column assembly

In the full-scale erection test, columns with the joint details are lifted as shown in Figure 3(a), which depicts the new column joints that consisted of L-shaped metal plates at the column connection (bolt holes were prepared in both the upper and lower metal plates). Interior bolts were also installed in the lower plate, which was fixed to the upper plate, providing additional flexural strength to the mechanical joint during the application of overturning moments. The two plates were inter-connected by high-strength exterior and interior bolts. Interior bolts preinstalled on the lower plate were anchored to the nuts in the recessed area prepared inside the upper column as illustrated in Figure 3(a,d), making the plate connection more ductile. These bolts were embedded with concrete mortar. Figure 3(b,c) shows the plates after they have been put in position and were ready to be bolted, and the observed time for this placement was less than 1 min. Finally, the two plates were inter-connected, as demonstrated in Figure 3(d). The plates and bolts were designed to transfer force couples (tension and compression) to create moment connections.

2.5. Assembly of the column-to-beam connection via mechanical joints

Figure 4 shows the full-scale erection test for the assembly of the column-to-beam connection, demonstrating the efficient assemblage of precast frames compared to that obtained by conventional precast construction. The beams and columns are connected by metal beam endplates at the ends of the beams and plates embedded in the face of the columns, which were, then, inter-connected to assemble the joint. Couplers in both the precast columns and beams were used to anchor rebars to plates. The erection of the proposed frame connection included the following procedures: (1) lifting beams (Figure 4(a)), (2) placing girders between columns with filler plates (Figure 4(b–f)), and (3) connecting girder and column plates by bolts (Figure 4(g,h)). Figure 4(c,d) show the column-to-girder joint assembly method involving bolting extended endplates borrowed from the traditional steel construction. It is worth noting that a girder-to-beam assembly (Figure 4(e)) was also assembled using connections with extended endplates. For the composite beams and girders, the steel sections were welded to the endplates. In Figures 1(b,1) and 4(h), the beam endplates and column plates inter-connected by high-strength bolts were anchored into couplers embedded in the column unit. Rigid moment connections were, then, formed between beams and the column plates. The torque in the bolts should be measured to ensure that the required pre-tension force in the bolt shank was successfully introduced. Top girder rebars are anchored into the embedded couplers prepared in the column face as shown in Figure 4(i). There are always some spaces available above ceiling that can accommodate the protruded steel plates as shown in Figure 4(j). When the laminated metal plate connections are used to splice columns exposed to the exterior of a building, the interior bolts can be installed in the recessed area shown in Figure 4(j). The recessed area should be grouted with non-shrinking high strength mortar. Frame assembly was completed as shown in Figure 4(k,l). Metal deck plates or pour forms for the construction of slabs were then put in place graphically. Figure 4(m) summarizes the assembly procedure with the specific time spent for the assembly of the two-bays frame (Figure 4(k,l)) when the proposed precast erection method was implemented in the erection test of this study. The entire time observed during the test erection of the frame was around 30 minutes from the lifting of the columns to the assembly of the beam-column joints. It is well
expected to take more than a week when the concrete is cast and cured with conventional cast-in-place construction method. The construction efficiency with rapid assembly was also obvious in which 100% dry assembly was demonstrated using irregular metal plates for the irregular column shapes. Cost saving also looks obvious when installing pour forms and construction delays due to the curing of the cast-in-place concrete is not necessary. However, the erection test in this study may differ from a real-world frame construction because more sophisticated preparations may be required in a real-world scenario.

2.6. Tolerance management

Erection tolerances between the precast concrete components should be introduced to account for the dimensional variations of thicknesses in the plates and the lengths of precast members. Metal filler plates shown in Figure 4(c,e) were used to compensate for dimensional variations between precast members for this erection test. The tolerances are very important when fitting beams between the preinstalled columns. The 10 mm tolerance was provided for the test assembly, installing beams was as fast as 5 minutes. The tolerance was then filled with metal plates. However, the wider the tolerances are, the thicker the metal filler plates become, affecting the cost of the frame assembly. The tolerances of 5 to 10 mm are recommended depending on the construction cost and workmanships. Tolerances between plates and bolts in bolt holes must be provided based on different reasons. The installation of the bolts in the bolt holes is not as difficult as that of filler plates. But problems occur when plates deform due to the loads, colliding into bolt shafts, which then displaces nuts from their positions (Nzabonimpa, Hong, and Park 2017a). The unexpected distortions between nuts and tread of re-bars can be prevented by maintaining plate deformation minimum or by providing sufficient gaps between nuts and the holes prepared in plates. The least tolerance of 5 mm is
recommended for this tolerance. The clearance for tolerance between precast units and structure should be managed at the cost of metal filler plates.

### 2.7. Application of the proposed L-shape frames with mechanical joints

Figure 5(a) describes a 20-story apartment building having L-type composite precast frames of multiple floors with mechanical joints. A case study based on non-linear finite element analysis has been performed to replace the conventional walls at the corner of the 20-story apartment building shown in Figure 5(b). The L-shaped precast frame was used in the erection test. A total axial load of 11,680 kN was applied to the columns in this case study; dead (2 kN/m²) and live (4 kN/m²) loads were used to design the plans shown in Figure 5(a,b). The wall frames at the corners were similar to those provided by the L-shaped columns, as compared in Figure 5(a,b). Substantial amounts of concrete and construction time can be saved by replacing structural walls with dry partitions; this also enhances the architectural flexibility at the corners.

### 3. Description of the finite element model

#### 3.1. Discretization of the proposed mechanical joints

Discretization of the proposed mechanical joints was performed using ABAQUS (Dassault Systèmes 2017). Elements of types C3D8R and R3D4 were chosen to represent the FE model of the proposed column-to-column mechanical joint, as shown in Figures 6(a) and 7. C3D8R and R3D4 elements were used to model the concrete column, rebar, bolts, embedded H-steels, hoops, and extended endplates. The cross section of the model is illustrated in Figure 6(b), showing the total number of
elements used in this column-to-column model was 220,087. Global mesh size is 20 mm. The constitutive relationships for the concrete, rebar, steel sections and the loading protocol used in finite element models are presented in Figure 6(c,d), respectively. More information can be found in the previous study of the authors (Nzabonimpa and Hong 2019).

3.2. Activations and evolutions of the strains of the L-shaped metal connection

In Figure 7, the strains and stresses at concrete, rebar, the interior, and the exterior) of the L-shaped column connection used in Figure 5(a,b) are identified at $M_0$ (350 kN-m). The frame dimensions of the columns having mechanical joints were determined based on a strain analysis provided by the non-linear finite element analysis. The load–displacement relationships (Figure 8) of the Specimens, LC1-WF, LC2-WF, and LC3-WF are presented in the previous study of the authors (Nzabonimpa and Hong 2019) in which the legends can be found. A nonlinear finite element investigation, based on concrete plasticity, was used to explore the strains and stresses exerted on the joints at the design load demand ($M_0$). The influences of the mechanical joint (utilizing bolted laminated metal plates), the interior bolts, and the thickness of the metal plates on the flexural capacity of the proposed connections were investigated as shown in Figure 9. Stress–strain relationships of the selected structural elements were retrieved with axial loads of 5,000 kN as shown in Figure 9 where the stress–strain relationships for the selected elements in the column-to-column mechanical joint were demonstrated. The stresses and strains are indicated by the red dots in Figure 9 when the concrete compressive strain reached its design limit value of 0.003. Concrete area reaching ultimate concrete compression strain of 0.003 for design value is illustrated in Figs (a) to (g), showing formation of early plastic hinge in the column section. Concrete area reaching compression strain of 0.003 was not wide, however, this area will spread as concrete column gets degraded, forming complete formation of the plastic hinge. The strains observed in the column rebars and steel sections were 0.0026 and 0.0024, respectively, as shown in Figure 9(a,b). At a concrete compressive strain of 0.003, the rebar (which has a yield strain of 0.0029) was about to yield, while the strains found in the steel section were 1.5 times larger than its yield strain ($\varepsilon_y = 0.0015$). It should be noted that the embedded steel section made a significant contribution to the flexural capacity of the proposed mechanical joint. In Figure 9(c,d), the metal plates (upper and lower) did not experience large deformations; strains in the upper and lower plates were found to be 0.00176 and 0.00124, respectively. Small strains were observed in the exterior bolts, whereas large strains were found at the interior bolts (see Figure 9(e,f), respectively. This is because the interior bolts were located near the embedded steel section, which exerted substantial tensions on the metal plate. A large portion of the forces exerted by the metal plates was transferred directly to the interior bolts with large strains. The strain in the interior bolts was 0.0076, which was twice that of the exterior bolts, as shown in Figure 9(e,f). The maximum compressive strength of concrete reached 37 MPa with a corresponding strain of 0.002, as shown in Figure 9(g). The structural elements of the joint demonstrated good structural performance at the design load limit state corresponding to a concrete strain of 0.003, demonstrating that the structural safety was verified by the experimental and numerical investigations. In Figure 9(a,b), rebars were less activated reaching 0.93$\varepsilon_y$ (0.0027) than L-shaped steel section which reached a strain of 1.51$\varepsilon_y$ (0.0024) when concrete reached a strain of 0.003. Rebars did not yield whereas L-shaped steel section yielded, recommending that use of rebars be reduced to activate rebars more, reducing rebar quantities. As shown in Figure 9(e,f), strains of exterior bolts reached only 0.7$\varepsilon_y$ (0.0034) whereas strains of interior bolts reached 1.51$\varepsilon_y$ (0.0074) when concrete reached a strain of 0.003, resulting in strains of 1.07$\varepsilon_y$ (0.0017) and 0.75$\varepsilon_y$ (0.0012) reached by
upper and lower plates, respectively. Use of interior bolts efficiently contributes to the stable behavior of the connection plates.

4. Construction efficiency

4.1. Structural quantities

Construction scenario for an evaluation of construction schedule and cost for the proposed precast building frames with mechanical joints is presented in Figure 10 where monolithic cast-in-place wall frames of 16 stories were replaced by precast frames utilizing mechanical plate joints. Table 1 provides the structural quantities which were slightly saved by 2% of material quantities compared with conventional monolithic cast-in-place wall frames.

4.2. Construction schedule

As shown in the erection test of the multi-bay frames with mechanical connections shown in Figure 4(k,l),
the proposed assembly of precast columns was quick and easy, proving constructability. Bolt holes were accurately aligned for bolt installations, improving the assembly efficiency. A construction schedule of the hybrid frames is compared with that of the reinforced concrete frames in Figures 11 and 12 where the construction durations of the precast frames and cast-in-place reinforced concrete frames were estimated. The total construction time for the frame erections of the proposed frames was expected to be shortened by the 2 months (10%) when compared with the cast-in-place concrete frames.

5. Architectural flexibility

L-shaped columns are preferred by architects, rather than rectangular ones because they have better architectural flexibility at the corners of the walls. Figure 13 shows protruded steel plates with an enlarged dimension of extended edge, showing the plates are wider than the L-shaped column by 100 mm. The extended edge is small enough to be accommodated in spaces available above ceiling. When the laminated metal plate connections are used to splice columns exposed to the exterior of a building, interior bolts can be installed in the recessed area shown in Figure 4(j). The recessed area should be grouted with non-shrinking high strength mortar. Consequently, the extended edge will not interfere with any architectural aspect.

6. Conclusions

The efficient and effortless assembly of the precast columns with irregular sections was shown to replace rectangular columns. The L-shaped wall column sections are
Figure 9. Stress-strain relationships of the selected structural elements with axial loads of 5,000 kN.

preferred by architects because of their architectural flexibility at the corners of walls. The following conclusions can be drawn from this study to be implemented by users.

1. A test erection using the full-scale precast columns with irregular sections that were interconnected by mechanical plates was performed to demonstrate that the construction time relative to conventional monolithic assembly could be reduced. The erection test eliminated the use of pour forms and curing times, which are required for conventional concrete.

2. For the precast designs of connections in this erection test, the dimensional variations between precast members were accounted for by providing erection tolerances between the precast concrete components. The test erection showed that the dimensional variations caused by thicknesses of the plates and the lengths of precast members were sufficiently addressed using metal filler plates. The clearance for the tolerance between

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precast units and structure was adjusted by modifying the metal filler plates. (3) The proposed joints were able to deliver an adequate flexural strength to provide connections with sufficient strength and resistance for the construction and service loads. The finite element analysis described in the present study presents strain activations and strain evolutions of the structural components consisting of the L-shaped mechanical connections. Figures 7, 8, and 9 can be useful when the hybrid joints are designed, showing how effectively the structural components of the joints are activated and evolving. An extensive finite element analysis was previously performed by the authors (Nzabonimpa and Hong 2019) where significant information can be found about the numerical investigation of the structural behavior of the L-shaped precast columns. (4) Formation of early plastic hinge in the column section was numerically observed in Figs (a) to (g) where concrete area reaching ultimate concrete compression strain of 0.003 is depicted for column design. The area formed by plastic hinge will spread as concrete column gets degraded, forming complete plastic hinges.

| Table 1. Cost comparison of construction materials. |
|------------------------------------------------------|
| **Case-in-plate wall frame (Won)** | **Precast frames (Won)** |
|-------------------------------------|--------------------------|
| 1. Temporary support                | 180,687,889              | 0                         |
| 2. Cast-in-place concrete           | 1,505,330,194            | 525,403,571               |
| 3. Precast concrete + Plates        | 0                        | 1,095,714,850             |
| and bolts                           |                          |                           |
| 4. Partitions                       | 0                        | 88,825,948                |
| 5. Finishing                        | 52,081,224               | 0                         |
| 6. Foundations                      | 49,390,959               | 44,359,959                |
| Total comparison                    | 1,787,690,266            | 1,754,304,328             |
| Saving by precast frame (%)         |                          | 2% saved                  |

Figure 10. Erection of precast columns of 2 stories implementing mechanical plates.

Figure 11. Construction schedule based on cast-in-place frames.
even if concrete area reaching compression strain of 0.003 is not wide.

(5) Nominal flexural strength of the proposed columns corresponding to concrete compression strain of 0.003 was found greater than the design moment ($M_d = 350$ kN·m) as shown by stress and strain levels of the structural components at the design moment (Figure 9). Resistance of the structure against load demand was reserved sufficiently.

(6) It can be design based on the resistance by calculating nominal strength and design strength (by multiplying nominal strength by strength reduction factor) of the sections which, then, should be greater than load demands.

(7) Strains of interior bolts reached $1.51\varepsilon_y$ (0.0074) whereas those of exterior bolts reached only $0.7\varepsilon_y$ (0.0034) when concrete reached a strain of 0.003, indicating that use of interior bolts efficiently contributes to the stable behavior of the connection plates which reached only $1.07\varepsilon_y$ (0.0017) and $0.75\varepsilon_y$ (0.0012) for upper and lower plates, respectively.

(8) L-shaped steel section reaching a strain of $1.51\varepsilon_y$ (0.0024) was activated more than rebars which reached $0.93\varepsilon_y$ (0.0027) when concrete reached a strain of 0.003. L-shaped steel section yielded whereas rebars did not, recommending that use of rebars be reduced to activate rebars more, reducing rebar quantities.
(9) The conventional buildings with walls can be replaced by the proposed L-type hybrid composite precast frames having mechanical connection details. The assembly of precast columns with irregular sections to replace rectangular columns is efficient and effortless, significantly reducing the construction time required to assemble precast frames relative to the conventional monolithic assembly. Novel erection method alternative to the conventional monolithic cast-in-place joints was provided.

Acknowledgments

This work was supported by the National Research Foundation of Korea (NRF) grant funded by the Korean government (MSIT 2019R1A2C2004965).

Disclosure statement

No potential conflict of interest was reported by the authors.

Funding

This work was supported by the National Research Foundation of Korea (NRF) grant funded by the Korean government (MSIT 2019R1A2C2004965).

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