

Finite Element Analysis of Precast Concrete Connections under Incremental load

¹Masoud Davari, ²Mahyar Ramezani, ²Aliakbar Hayatdavoodi, ²Mohammad Nazari

¹Department of Strength of Materials and Structural Engineering, Universitat Politècnica de Catalunya (UPC) Jordi Girona 1-3, Edifici C1, 08034 Barcelona, Spain

²Department of Civil and Structural Engineering, Faculty of Engineering and Built Environment, Universiti Kebangsaan Malaysia

Abstract: Beam-to-column connections affect the rigidity and strength of the overall precast concrete structures. Even though many experimental researches have been carried out on beam-to-column connections, the behavior and failure mode of the connection in precast concrete is often difficult to assess through experimental program. The finite element analysis, on the other hand, can be an option to properly evaluate the condition of the connections. Nevertheless, the finite element analysis on the beam-to-column connections is quite limited. Thus, there is a need to study and explore the behavior of the connection system based on the finite element data. In this research, the finite element analysis was performed to study the performance of different types of beam-to-column connections in precast concrete frames. A total of four specimens were modeled and analyzed to study the connection behavior involving load-displacement relationship under static incremental load. Different connection details were considered and the behavior of various beam-to-column connections were investigated.

Key words: Precast concrete frame, Pinned Beam-to-column Connection, Finite element method

INTRODUCTION

Precast concrete construction has been getting popularity and being widely applied in construction industry (Ahmad Baharuddin Abd. Rahman and Dennis Chan Paul Leong, 2006). The most important advantage of using precast concrete is the opportunity to achieve consistent end products. In recent years, the increasing shortage of skilled workers in Malaysia has lowered the standard of workmanship in many projects. In precast construction, factory controlled conditions will enable the desired quality, dimension, as well as color and texture of precast concrete to be easily achieved (Construction Industry Development Board, 1997). The history of precast concrete dates back to few decades ago in which several factors such as rising steel costs, material shortages during the Korean conflict, the expanded highway construction program, and the development of mass production methods to minimize labor costs led to the use of precast concrete in the United States (Sheppard and Philips, 1989). The first precast concrete skeletal frame in United Kingdom was Weaver's Mill in Swansea which was constructed in 1897-98 (Elliot K.S *et al.*, 1998). The success of precast concrete buildings depends on the connections between the components, particularly beam-to-column connections. Furthermore, the behavior and failure mode of the connection in precast concrete is often difficult to predict due to various types of joint and modifications in connection. The behavior of the connection can therefore be properly assessed using finite element method. However, there is a lack of finite element data and analytical proof to determine the behaviour of the ductile connection in precast concrete. Therefore, more research needs to be done.

Simulation of Connections:

This study investigated a total of four specimens which have been tested in the laboratory using experimental test by Ahmad Baharuddin and Dennis Chan (Ahmad Baharuddin Abd. Rahman and Dennis Chan Paul Leong, 2006). Each specimen basically consisted of a precast beam of 200 × 300 mm cross-section with 1000 mm in length. The column size was 200 × 200 mm cross-section and 2000 mm in total height, with a corbel of 200 mm wide and 220 mm in height.

Specimen 1 was a simple connection with 16 mm diameter dowel bar projecting from corbel in the precast column. The precast beam was inserted into the projecting dowel and supported by a bearing pad with dimension of 150 × 80 × 10 mm. While for specimen 2, an additional top fixing angle cleat of 150 × 90 × 10 mm thick and 80 mm width was placed on top of the precast beam. The projecting dowel bar was bolted through the seating angle cleat. A 16 mm diameter threaded bolt was then inserted to the seating cleat to pass through the column, bolted with 80 × 80 × 10 mm thick steel plate located at the other end. Similar to specimen 2, specimen 3 was connected using the same method, except that the angle cleat of 150 × 90 × 10 mm thick and 80 mm width was used and stiffened by a single bolt of 16 mm diameter. Specimen 4 was modeled with stiffened angle cleat of 150 × 90 × 10 mm thick and 150 mm width. The connection consisted of two 16 mm diameter threaded bolts which passed through the column. Table 1 shows the details of the reinforcements used in precast beams, columns, and corbels. The concrete strength was 40 N/mm² at 28 days.

The minimum cover provided for all precast components was 20 mm. The details of all specimens are shown in Figure 1.

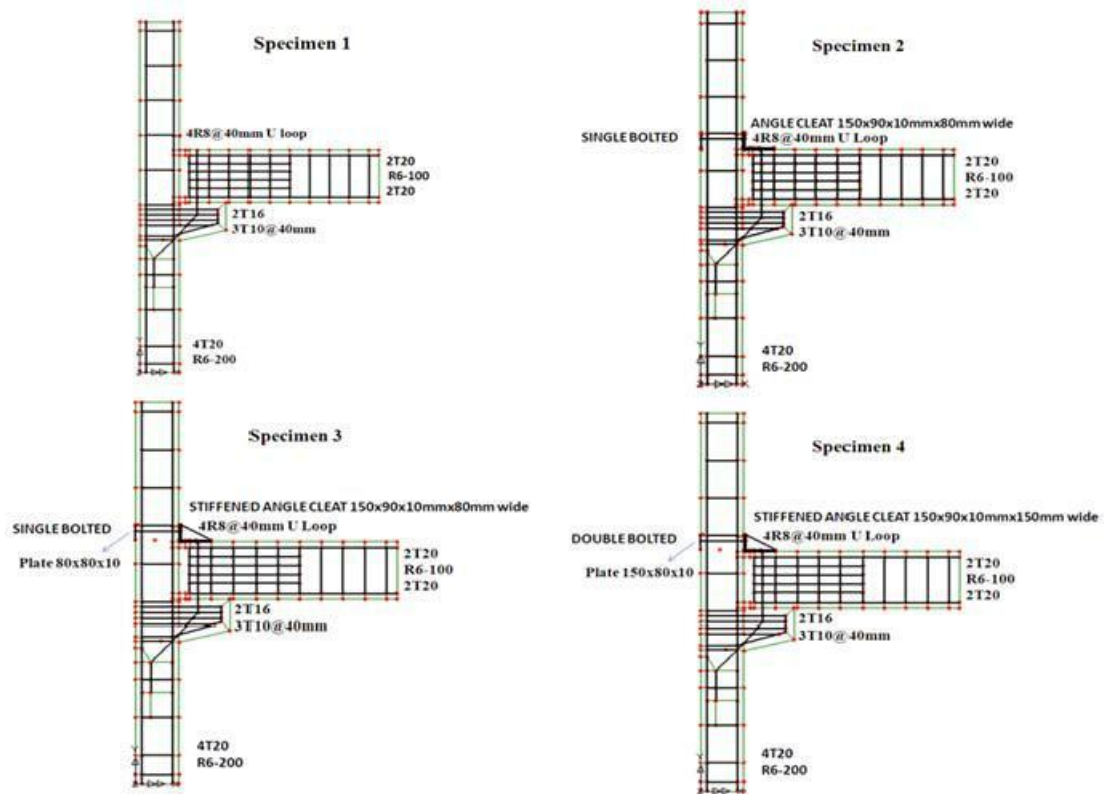


Fig. 1: Geometry of the specimens in LUSAS modeler.

Table 1: Detail of all Reinforcements used in specimens.

Reinforcements	Diameter (mm)	Area (mm ²)
U loop	16	200.96
Column reinforcement	20	314
Corbel reinforcement	16	200.96
Shear links/column and beam links	12	113.04
Shear links/corbel links	10	78.5

MATERIAL AND METHOD

Material Properties:

There are 3 groups of material properties for the finite element model (MacNeal-Schwendler, 1992). Table 2 shows the material properties of concrete components. The material properties for the steel components are shown in Table 3 (Thomas Telford Ltd, 1986).

Table 2: Material properties for concrete components.

Material properties	concrete
Elastic	
Young's modulus, E_c	18×10^3 N/mm ²
Poisson's ratio	0.2
Plastic (Cracking & Crushing model)	
Tensile strength, f_t	4 N/mm ²
Compressive strength, f_c	40 N/mm ²
Strain at peak compressive stress, ϵ_{cp}	2.7×10^{-3}
Strain at end of compressive softening curve, ϵ_{co}	0.0136
Strain at end of tensile softening curve, ϵ_{to}	0.004

Table 3: Material properties for steel components.

Material Properties	Steel Reinforcement & shear links
Elastic	
Young's modulus, E_c	$2.09 \times 10^5 \text{ N/mm}^2$
Poisson's ratio	0.3
Plastic (Stress potential model)	
Initial uniaxial yield stress	250/460 N/mm^2
Ultimate yield stress	275/490 N/mm^2
Strain at end of yield curve, ϵ_{s0}	-
Hardening gradient, E_{rp}	2121 N/mm^2

In non-linear finite element analysis (NLFEA), the material non-linearity stress-strain curve for the concrete in compression, concrete in tension, and steel are shown in Figures 2, 3, and 4, respectively (Thomas Telford Ltd, 1986 and MacNeal-Schwendler, 1992).

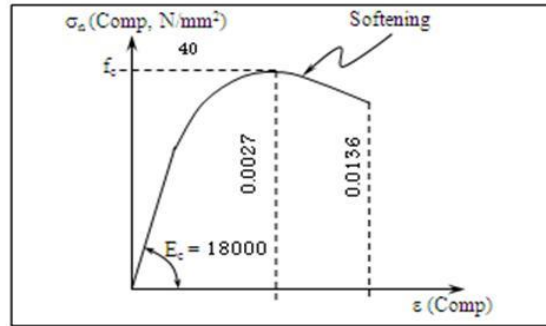


Fig. 2: Compressive behavior of concrete.

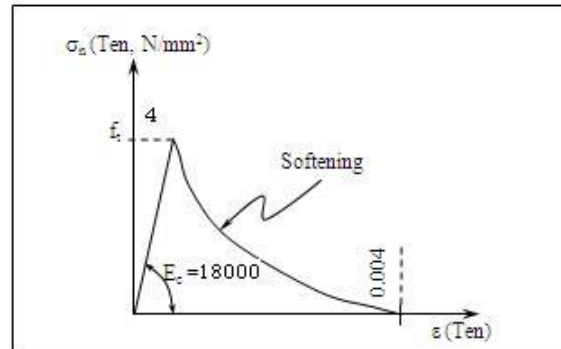


Fig. 3: Tensile behavior of concrete.

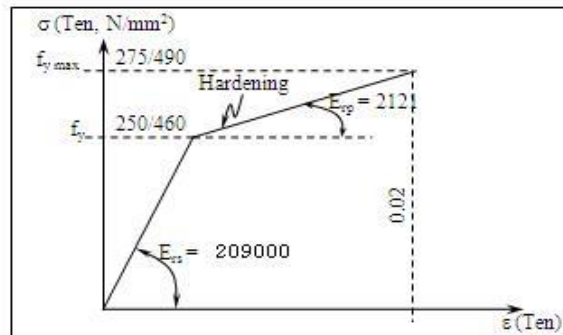


Fig. 4: Tensile behavior of steel reinforcement bar.

Meshing:

The 50 mm × 50 mm mesh size was chosen. The 3D finite element model was discretized into finite element with surfaces and lines and volumes one to one meshing in LUSAS Modeler (Lusas help Solver Reference Model 2001). The overall view of the finite element after meshing is shown in Figure 5.

Boundary Condition Loading:

LUSAS Modeler allows all input of restraints or loads at individual nodes and elements to be done directly to the selected entities. Restraints were applied to the top and bottom of the support as to present fully fixed. A point load is applied at the top of the beam which is 900 mm from the column face to produce moment at the connections. The point load is applied incrementally. The locations of the displacement measurement are shown in Figures 5 and 6. The distance between point 1 and point 2 was 900 mm, measured from the centre line of precast beam. While for point 3, it was located at 300 mm from precast column face. Point 4 was at distance of 200 mm from point 3 followed by point 5 at a distance of 200 mm from point 4.

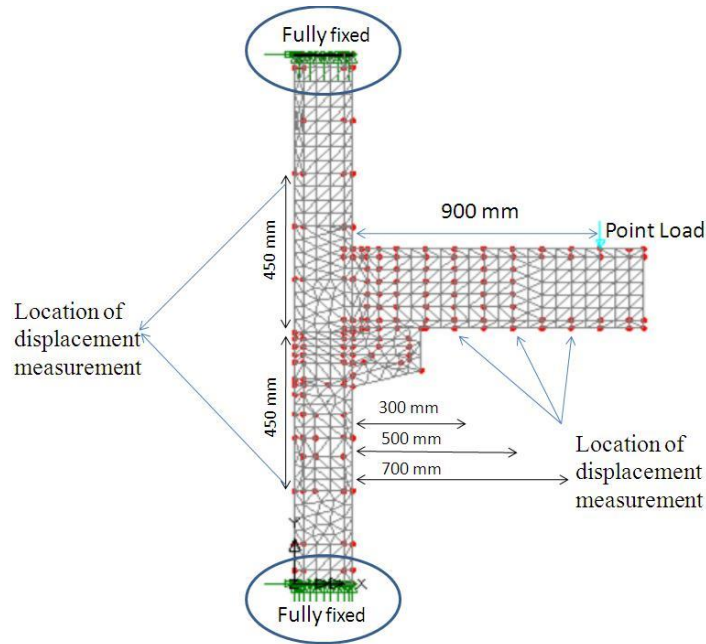


Fig. 5: Meshing, boundary condition and load of the finite element model.

Results:**Load-displacement Results:**

Figure 6 shows the various important points that involved in the calculation of displacements.

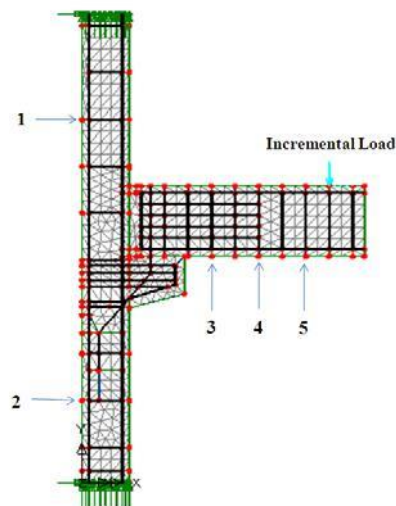


Fig. 6: location of displacement measurement.

Specimen 1:

The final load-displacement curves of specimen 1 for each point are presented in Figure 7. The total applied load included self-weight of precast beam (1.44 kN) and incremental load (0.0659 kN). In the beginning of the connection analysis, the precast beam was found to rotate and deflect with initial values of 0.184 mm, 14.0058 mm, 26.172 mm, and 40.42 mm at Point 1, Point 3, Point 4, and Point 5, respectively, due to self-weight of precast beam and incremental load. However, no initial deflection was found at Point 2. After the precast beam has touched the corbel (All specimens were modeled with bearing pad 80 mm $\times$ 80 mm in cross-section with 10 mm thickness between corbel and beam), the load resistance of specimen 1 reached a maximum value of 17.33 kN, with maximum displacements of 4.9761 mm (Point 1), 0.0368 mm (Point 2), 19.453 mm (Point 3), 36.5754 mm (Point 4), and 52.128 mm (Point 5).

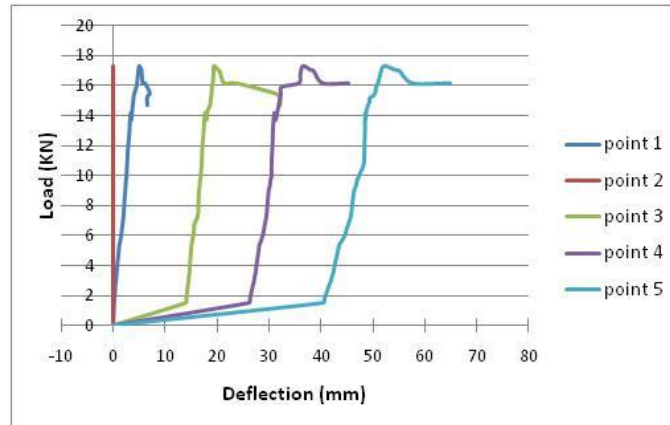


Fig. 7: Load-displacement curves of specimen 1.

Specimen 2:

The load-displacement curves of specimen 2 are illustrated in Figure 8. Like specimen 1, the total applied load of specimen 2 included self-weight of a precast beam and incremental load. As shown in Figure 8, the displacements increased steadily until it attained maximum value of 15.414 kN (before the precast beam touched the corbel). In excess of 15.414 kN, the displacements were found to increase drastically (when precast beam was supported by corbel edge). The displacements were 3.726 mm (Point 1), 0.441 mm (Point 2), 9.0336 mm (Point 3), 16.5816 mm (Point 4), and 22.85806 mm (Point 5). The load resistance of specimen 2 reached a maximum value of 40.4871 kN, with maximum displacements of 9.64 mm (Point 1), 0.441 mm (Point 2), 22.32 mm (Point 3), 43.5264 mm (Point 4), and 61.0428 mm (Point 5).

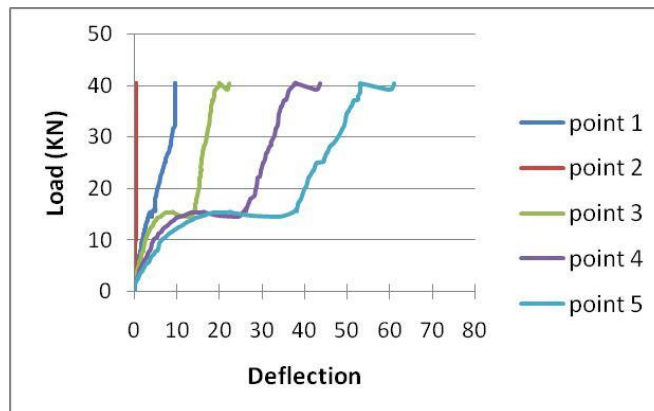


Fig. 8: Load-displacement curves of specimen 2.

Specimen 3:

The load-displacement relationships of specimen 3 were plotted as shown in Figure 9. The total applied load includes of self-weight of precast beam and incremental load which was applied on top of the precast beam. The load-displacement curves of specimen 3 also showed two stages of increment like specimen 2. At first stage (before the precast beam touched the corbel), the displacements increased steadily until it attained maximum value of 14.175 kN. The displacements were 4.082 mm (Point 1), 0.352 mm (Point 2), 7.98 mm (Point 3), 14.994 mm (Point 4), and 19.542 mm (Point 5). When precast beam was supported by corbel edge, it led to a rapid increment of displacement values while the loads were found decreasing.

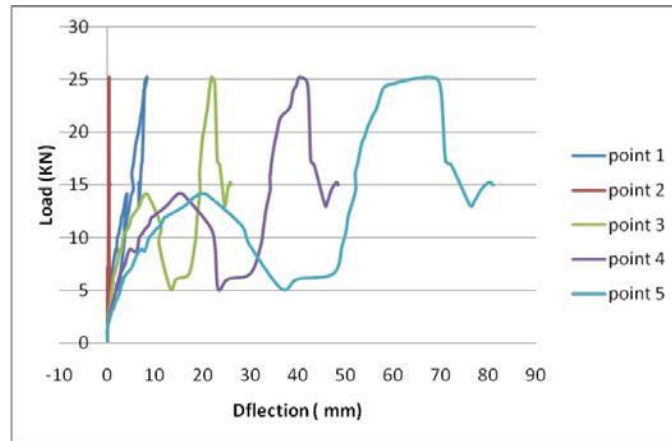


Fig. 9: Load-displacement curves of specimen 3.

Specimen 4:

The load-displacement curves for specimen 4 are as illustrated in Figure 10. The total applied load of specimen 4 was the sum of the precast beam self-weight and incremental load. Specimen 4 showed a steady increment of displacements with corresponding applied loads when applied loads were less than 16.849 kN. The maximum displacements recorded at this stage were 4.065 mm (Point 1), 0.156 mm (Point 2), 8.883 mm (Point 3), 13.824 mm (Point 4) and 21.554 mm (Point 5). Subsequently, the vertical displacements started to increase rapidly after this value had been exceeded. Finally, the specimen resisted a total applied load of 36.052 kN as well as maximum displacement values of 8.914 mm (Point 1), 0.156 mm (Point 2), 20.885 mm (Point 3), 32.796 mm (Point 4) and 51.587 mm (Point 5).

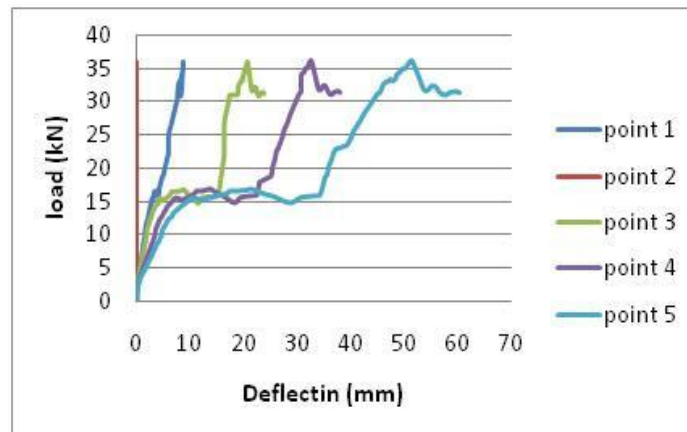


Fig. 10: Load-displacement curves of specimen 4.

Discussion:

For comparison purpose, load-displacement curves of specimens at every single point were plotted into the same graph, as shown in Figure 11 through Figure 15. Furthermore, maximum load and maximum displacement values attained by each specimen were summarized in Table 4 through Table 6.

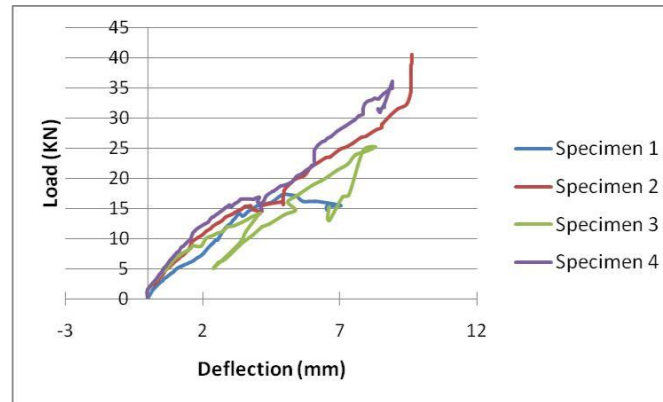


Fig. 11: Load-displacement curves at Point 1.

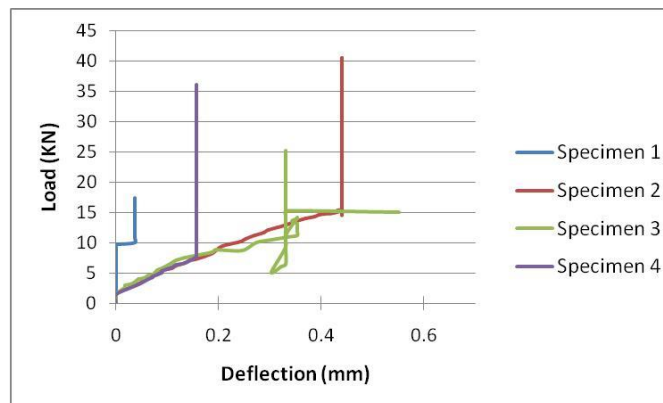


Fig. 12: Load-displacement curves at Point 2.

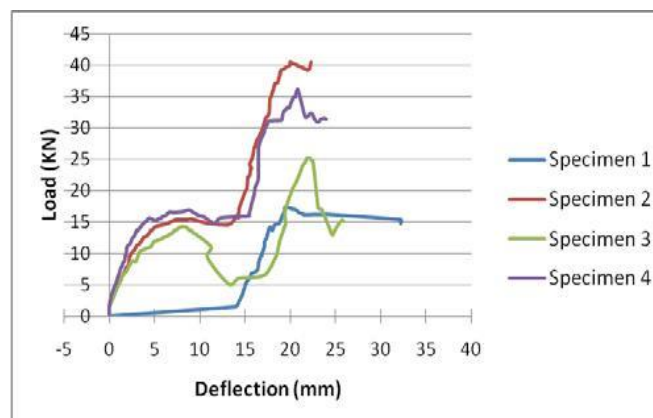


Fig. 13: Load-displacement curves at Point 3.

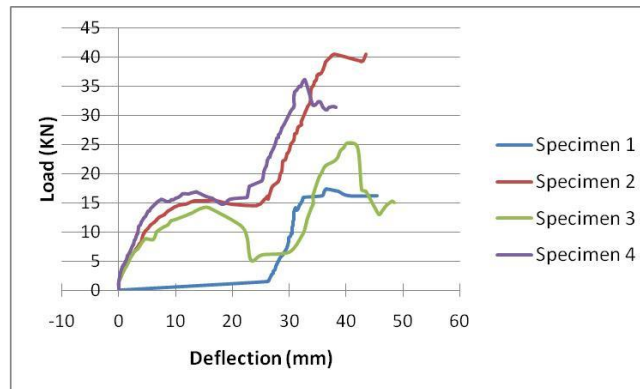


Fig. 14: Load-displacement curves at Point 4.

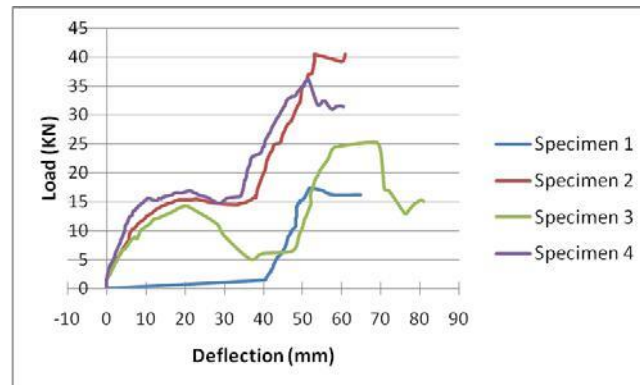


Fig. 15: Load-displacement curves at Point 5.

Table 4: Load resistances of specimens.

Type	Maximum Load (KN)	
	First stage	Final stage
Specimen 1	-	17.329
Specimen 2	15.414	40.487
Specimen 3	14.175	25.253
Specimen 4	16.849	36.052

Table 5: Maximum displacements of specimens at first stage.

Specimen	Maximum displacement (mm) at first stage				
	Point				
	1	2	3	4	5
1	-	-	-	-	-
2	3.726	0.441	9.033	16.581	22.858
3	4.082	0.352	7.98	14.994	19.542
4	4.065	0.156	8.883	13.824	21.554

Table 6: Maximum displacements of specimens at final stage.

Specimen	Maximum displacement (mm) at final stage				
	Point				
	1	2	3	4	5
1	4.976	0.036	19.453	36.575	52.128
2	3.726	0.441	9.033	16.581	22.858
3	4.082	0.352	7.98	14.994	19.542
4	4.065	0.156	8.883	13.824	21.554

From the above figures and tables, specimen 2, specimen 3, and specimen 4 achieved load resistances of 15.414 kN, 14.175 kN, and 16.849 kN, respectively, at the first stage or before the precast beam had touched the corbel. While for specimen 1, the load resistance was almost zero before it touched the corbel. At the second stage or after the precast beam has touched the corbel, specimen 2, specimen 3, and specimen 4 attained higher ultimate load resistances, which were 133%, 45%, and 108% higher than specimen 1, respectively. At this stage, specimen 2 achieved the highest ultimate load resistance which was then followed by specimen 4. Meanwhile, specimen 3 had the weakest load resistance when compared with specimens 2 and 4. In addition, the use of the angle cleat in the precast concrete simple beam-to-column connection increased the load resistance of the entire connection in this study.

The use of ordinary angle cleat without stiffener as connector performed better in this study. When loads were applied to the specimen, the connection part (particularly at connecting cleat) was subjected to shear forces as it tended to restrain the precast beam from rotation movements. In this case, ordinary angle cleat used in specimen 2 was more flexible in distortion process when it was subjected to shear forces. It tended to follow the shear forces direction when distorted. Therefore, shear forces were distributed over the cleat, whereas forces induced on dowel at connecting cleat were lesser due to distortion actions. Indirectly, the initial yielding of dowel (bolting part) had been slowed down, causing the precast beam deflected and cracked slowly at connection part before touching the corbel to start the second stage of increment. Consequently, it attained higher load resistance at first stage and maintained constant loads before touching the corbel. Eventually, it exhibited the highest load resistance amongst all.

While for specimen 3 stiffened type angle cleat was used. The distortion process was hardly to occur with stiffener. Thus, rotation movements of precast beam had induced greater shear forces to be resisted by dowel. Consequently, it yielded at a faster rate and caused the precast beam to deflect faster before touching the corbel, resulting in declinations of resisting loads and rapid increments in deflections after first yield. As a result, it attained lesser ultimate load resistance at final stage.

In the case of specimen 4, two steel bolts were used as the connector, connecting the stiffened angle cleat ($150 \times 90 \times 150$ mm) and passing through the precast column. Although the stiffened angle cleat did not resist the induced shear forces, the shear forces had been distributed to the bolts. Thus, the dowel (bolting part) could yield slower and was able to attain a higher load resistance at the first stage. After the first yield, specimen 4 managed to maintain its maximum load resistance prior to the beginning of second stage increment as deflection mechanisms of precast beam was much slower.

Furthermore, another significant observation can be made. The use of an angle cleat had controlled the initial vertical displacement of precast beam due to its self-weight. With additional top fixing cleat, specimens could also reach the maximum displacement values prior to final failure like controlled specimen. In other words, the use of angle cleats did not significantly reduce the ductility of connections.

Conclusion:

The load capacity of specimen 4 reached 16.849 kN followed by specimen 2 with 15.414 kN, specimen 3 with 14.175 kN, and specimen 1 with zero load resistance. Obviously, specimen 4 that consisted of stiffened angle cleat with two bolts connecting the precast column performed well in this study. Nevertheless, it is found that adding steel angles in specimen 2, specimen 3, and specimen 4 did not significantly reduce the ductility of connection. Thus, adding steel angle cleat in the simple connection improved the performance of the entire connection.

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