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Finite Element Analysis of Precast Concrete Connections under Incremental load

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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, due to lack of finite element testing which can inhibit further improvement on the beam-to-column connections, there is a need to study and explore the behavior of the connection system based on finite element data is quite limited Therefore, in this research, an idealization to test through finite element method was formed. The test was to study the performance of beam-to-column with different type of connections in precast concrete frames. A total of 4 specimens were modeled and analyzed to study the connection behavior involving load-displacement relationship, under static incremental load. In this research, some modifications were introduced to the specimens and then connection behavior of various beam-column connections were investigated

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

INTRODUCTION

Precast concrete construction have been getting popular and being widely applied in construction industry today (Ahmad BaharuddinAbd. Rahman and Dennis Chan Paul Leong, 2006). The most important advantage of using precast is the opportunity to achieve consistently 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, and colournd 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 have all been factors leading to the use of precast concrete in 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 of the components in 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. Therefore, more research works need to be done and there is lacking of finite element data and analytical proof accounts to determine the behaviour of the ductile connection in precast concrete. The behaviour of the connection can only be properly assessed by finite element method.

Simulation of Connections:

This study involved a total of four specimens which have been tested in the laboratory using experimental test by Ahmad Baharuddin and Dennis Chan (Ahmad BaharuddinAbd. Rahman and Dennis Chan Paul Leong, 2006) . Each specimen basically consisted of a precast beam of 200 x 300 mm cross-section with 1000 mm in length. The column size was 200 x 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 x 80 x 10 mm. While for specimen 2 an additional top fixing angle cleat of 150 x 90 x 10 mm thick and 80 mm wide 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 x 80 x 10 mm thick steel plate located at other end.

Similarly with specimen 2, specimen 3 was connected using the same method, except the angle cleat of 150 x 90 x 10 mm thick and 80 mm wide and stiffened by single bolt of 16 mm diameter. Specimen 4 was modeled with stiffened angle cleat of 150 x 90 x 10 mm thick and 150 mm wide. The connection involved the bolting of two 16 mm diameter threaded bolts which passed through the column. Table 1 shows the detail of all reinforcement used in precast beams, columns and corbels. The concrete strength used was 40 N/mm2 in 28

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days. The minimum covers provided for all precast components were 20 mm. The details of all specimens are as shown in Figure 1.

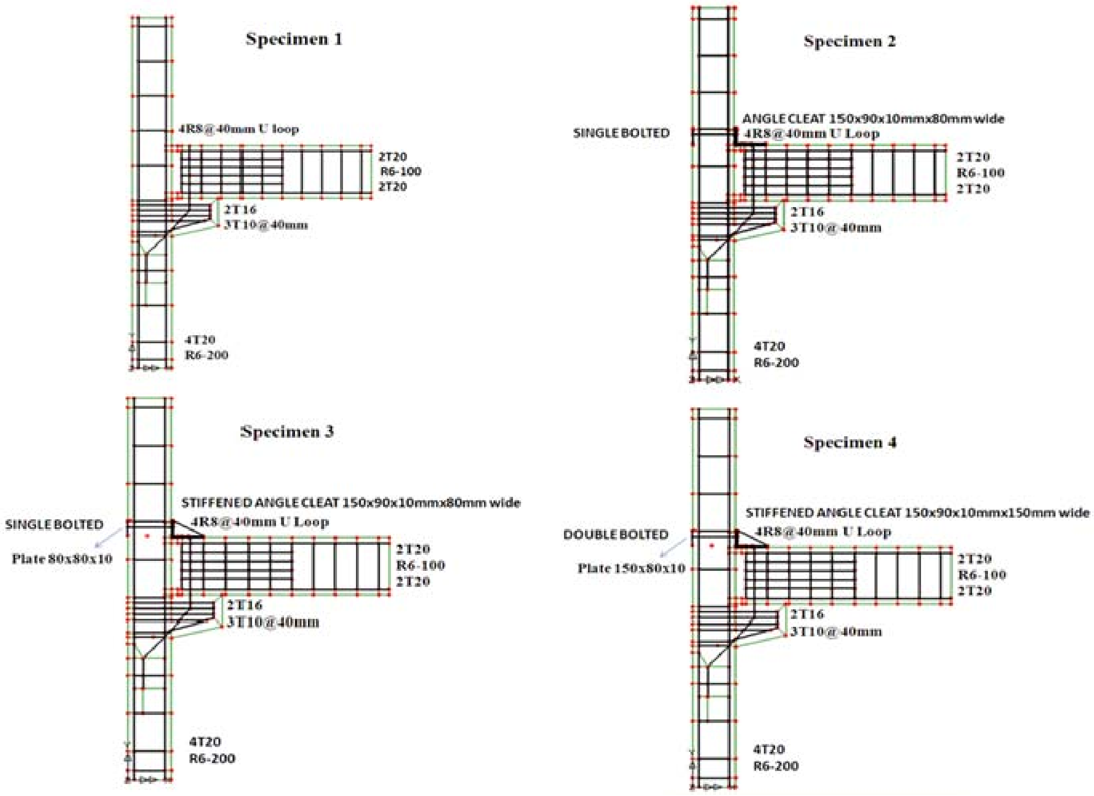


Fig. 1: Geometry of the specimens in LUSAS modeler.

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. 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,Ec	18x103 N/mm2
Poisson's ratio	0.2
Plastic (Cracking & Crushing model):	
Tensile strength, ft	4 N/mm2
Compressive strength, fc	40 N/mm2
Strain at peak compressive stress, ecp	2.7x10-3
Strain at end of compressive softening curve, eco	0.0136
Strain at end of tensile softening curve, eto	0.004

Table 3: Material properties for steel components.

Material Properties	Steel Reinforcement & shear links
Elastic:	
Young's modulus,Ec Poisson's ratio	2.09x105 N/mm2 0.3
Plastic (Stress potential model):	
Initial uniaxial yield stress Ultimate yield stress Strain at end of yield curve, eso Hardening gradient, Erp	250/460 N/mm2 275/490 N/mm2 - 2121 N/mm2

In non-linear finite element analysis (NLFEA), the material non-linearity stress-strain curve for the concrete and steel are shown respectively in Figure 2, 3 and 4 (Thomas Telford Ltd, 1986 and MacNeal-Schwendler, 1992) (Reza Masoudnia, *et al*, 2011).

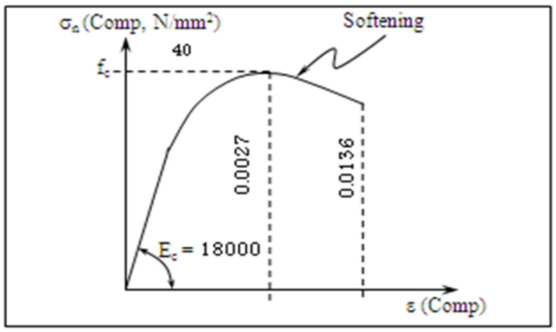


Fig. 2: Compressive behavior of concrete.

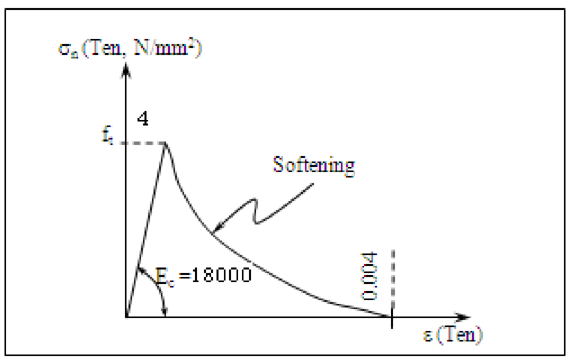


Fig. 3: Tensile behavior of concrete.

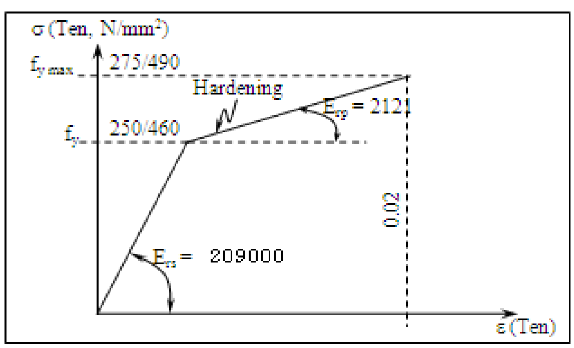


Fig. 4: Tensile behavior of steel reinforcement bar.

Meshing:

The 50mmx50mm 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) (Reza Masoudnia, *et al*, 2011). 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 900mm from the column face to produce moment at the connections. The point load is applied incrementally. The location of displacement measurement is as illustrated in Figure 5. 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; then followed by point 5 at a distance of 200 mm from point 4.

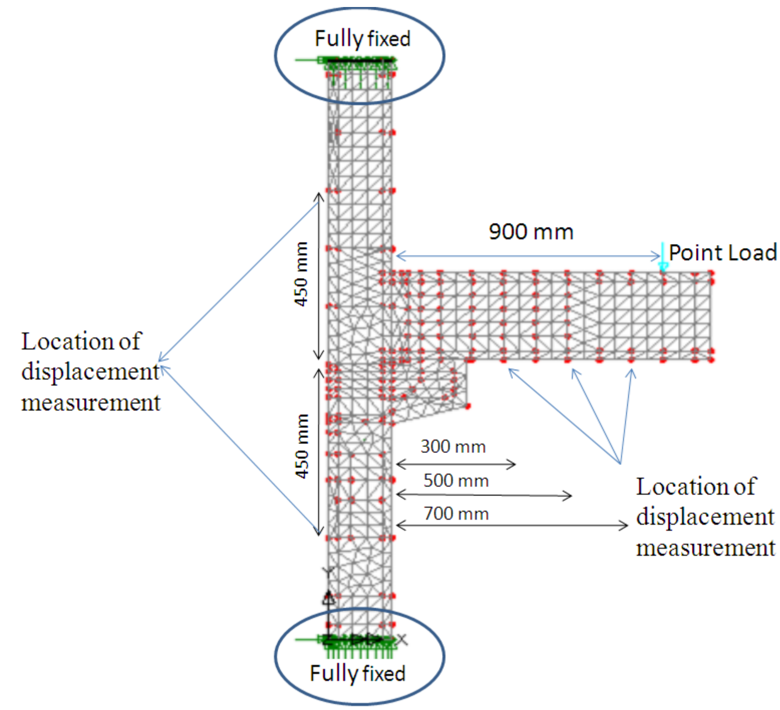
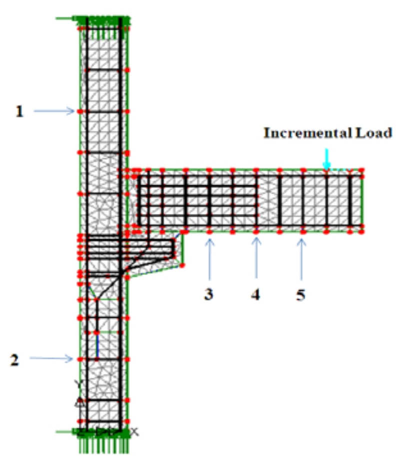


Fig. 5: Meshing, boundary condition and load of the finite element model.

Results:
Load-displacement Results:



In this case, P is the incremental point load applied at the beam end, and δ is the corresponding deflection under the incremental applied load. Figure 6 shows the various important points that involved in the calculation of displacement.

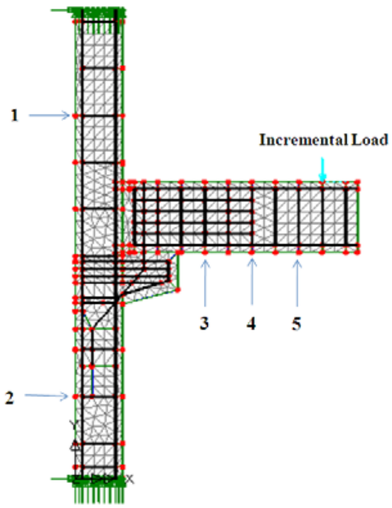


Fig. 6: location of displacement measurement.

Specimen 1:

The final load-displacement curves of specimen 1 for each point are as 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 80mmx80mm 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).

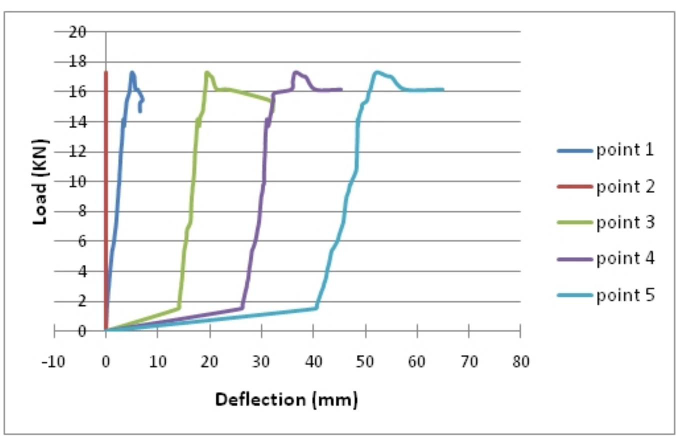


Fig. 7: Load-displacement curves of specimen 1.

Specimen 2:

The load-displacement curves of specimen 2 are as illustrated in Figure 8. Like specimen 1, the total applied load of specimen 2 included self weight of 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 (Point5). The load resistance of specimen 1 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).

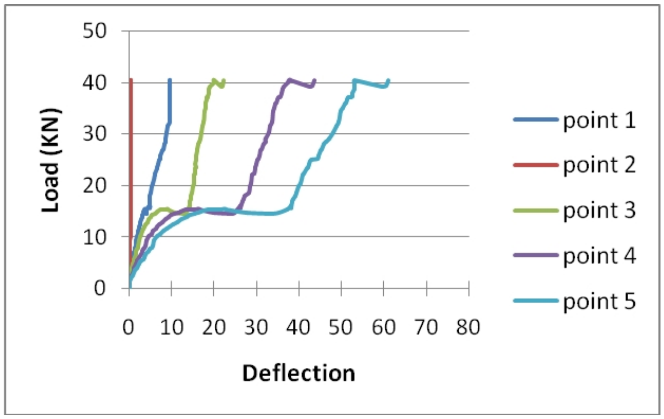


Fig. 8: Load-displacement curves of specimen 2.

Specimen 3:

The load-displacement relationships of specimen 3 were plotted and are as shown as in Figure 9. The total applied load includes of self weight of precast beam and incremental load which was applied on top of 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 had led to rapid increment of displacement values while the loads were found decreasing.

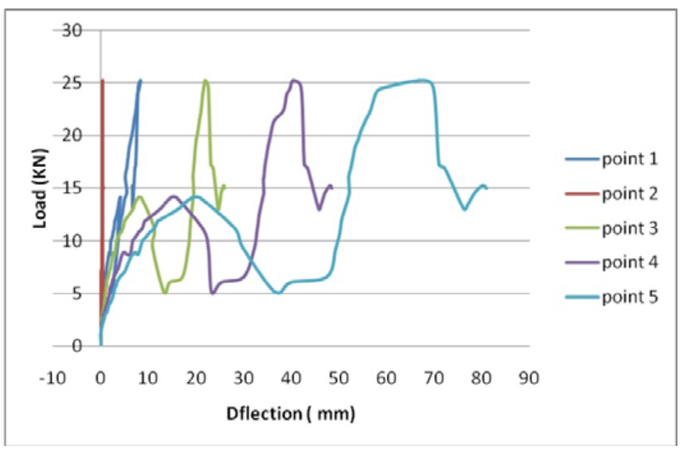


Fig. 9: Load-displacement curves of specimen 3.

Specimen 4:

The load-displacement curves are as illustrated in Figure 10. The total applied load of specimen 4 was the sum of 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 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).

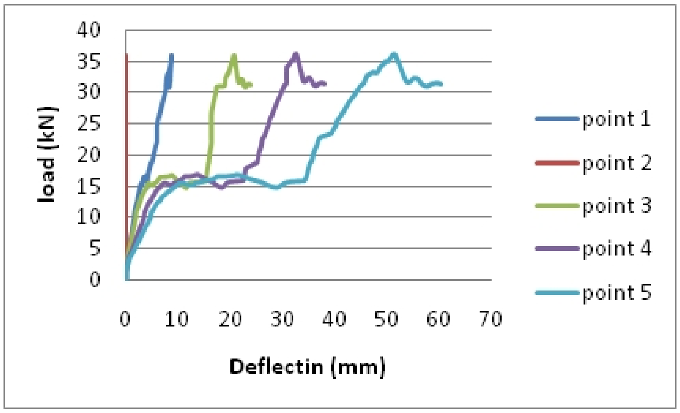


Fig. 10: Load-displacement curves of specimen 4.

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 16. Furthermore, maximum load and maximum displacement values attained by each specimen were summarized in Table 4.1 through Table 4.3.

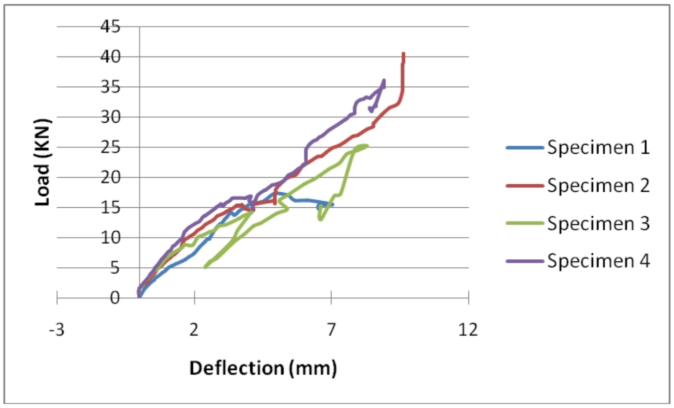


Fig. 11: Load-displacement curves at Point 1.

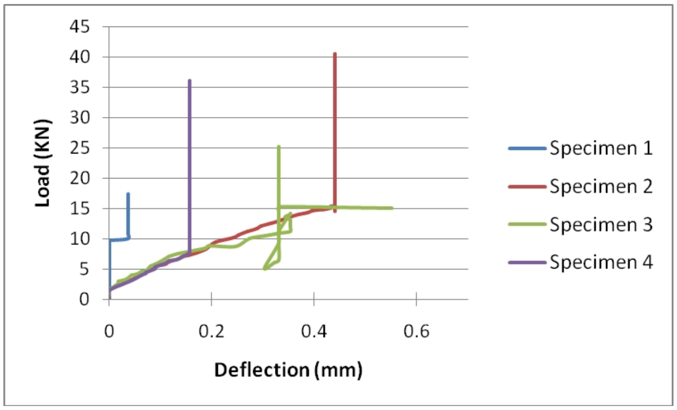


Fig. 12: Load-displacement curves at Point 2.

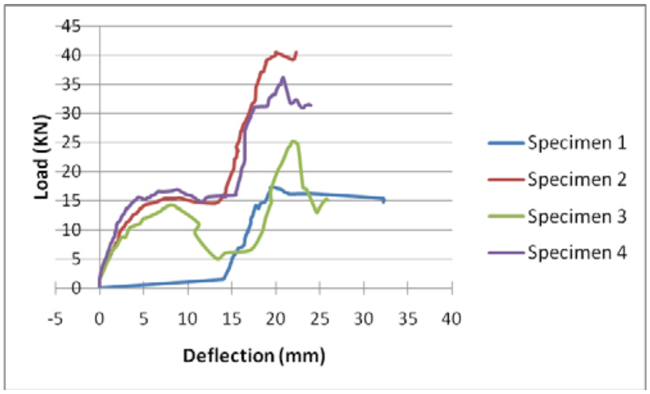


Fig. 13: Load-displacement curves at Point 3.

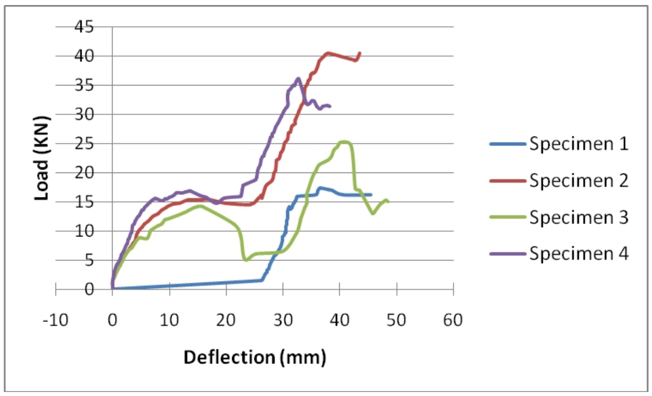


Fig. 14: Load-displacement curves at Point 4.

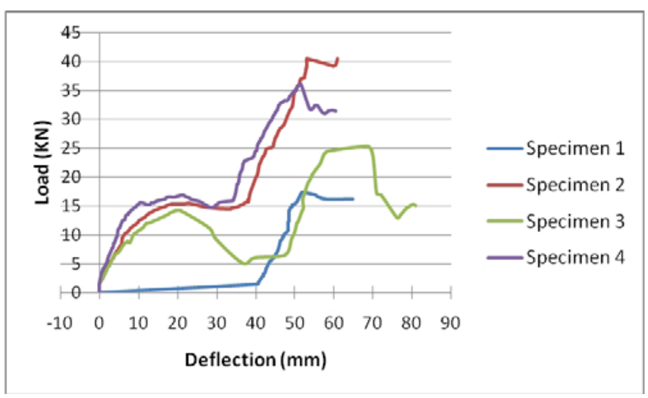


Fig. 15: Load-displacement curves at Point 5.

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	7.98	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 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 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; it was then followed by specimen 4. Meanwhile, specimen 3 had the weakest load resistance if compared to specimen 2 and specimen 4. In addition, the use of angle cleat in precast concrete simple beam-to-column connection had 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 recorded the highest load resistance among all.

While for specimen 3, stiffened type angle cleat was used. The distortion process was hardly to occur with stiffener. Hence, 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, thereby 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 x 90 x 150 mm wide) and passing through the precast column. Although the stiffened angle cleat did not resist the induced shear forces, however, these forces had been distributed to the bolts. Hence, the dowel could (bolting part) yield slower and thereby able to attain higher load resistance at 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 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 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; it was then 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. Hence, adding steel angle cleat in the simple connection had improved the performance of the entire connection.

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