

New Technique for Partial Continuity of Simple Support Beam-Column Connection

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Abstract: *This paper presents new technique to develop the precast concrete beam column connection to be capable of mobilizing the beam's negative moment and relieve percent of the direct reaction from column corbel to be direct transfer to the column core.*

This was achieved by providing twice additional new detail; the first new reinforcement details in the structural topping above the beam column connection to increase the capability of the beam section in negative zone. The second new modification is steel base plate between the column corbel and beam to shift the component of the beam reaction to the column core in trial to relief the direct column corbel reaction.

In the study; experimental and numerical approaches were adopted to predict deflections, inclination angles and bending moment distribution for the Mockup structure at different loading stages.

The new details success to create partial continuity of simply supported precast beam column connection by 28 to 33% of the theoretical fixed support case at the mid support of Beam, and partial fixity of 14 to 25% of theoretical fixed support case at column.

Keywords: *Precast concrete, Beam column connection, Partial continuity, Simple support.*

I. Introduction

In one of the biggest educational project in kingdom of Saudi Arabia a precast concrete framed buildings are those for which it is intended to carry out remedial measures to the partially completed buildings to address know shortfalls in the original design, detailing of reinforcement, and in the shop drawing detailing, and to address defects in manufacture and construction of the structure so far.

One of the fundamental shortfalls in the original design, detailing, manufacture, and construction is that of the connections between pre-cast concrete beams and pre-cast concrete corbels of columns. This study has developed remedial proposals for modifying these connections such that they will be able to transfer the forces required.

There are two main component parts to the proposed remedial works for these typical junctions:

- 1- To introduce partial continuity along beam-lines within the completed structures- by enhancing the structural topping reinforcement details, and by infilling the gaps between beam ends and column faces with grout.
- 2- To effectively extend the beams closer to their supporting columns by introducing extension steel plates to the bottom of the beams by creating a shear key within each beam end, as shown in Fig. 1.

The study has carried out numerical analysis to show the adequacy of each component part of the system. Additionally it was agreed that the experimental test results (Mock-Up sample) would be constructed and load tested. The mock-Up included exaggerated versions of some of the known defects.

The mock-Up had instrumentation applied to its own weight and planned loading regime as the original design criteria of structure.

II. Existing Shortfalls In Precast Beam To Column Corbel Connections

In the above mentioned project; the design review identified that, although the original structural design of the precast concrete corbels was numerically adequate, the original design detailing did not take account of the design requirements with the result that the beam to corbel junctions did not comply with recognized standards PCI(2007).

This shortfall of the design detailing means that, even for cases in which the reinforcement had been fixed in accordance with the design details, the actual capacity of the corbel might be lower than that derived numerically.

In fact, the situation is known to be worse than that. The following deviations from the approved shop drawings are known to exist in various locations:

- 1- Cover between corbel-face and down-turned primary corbel bars greater than shown on drawings
- 2- Gap between end of beam and face of column larger than intended.
- 3- Cover between beam-end and up-turned beam bars greater than shown on drawings.
- 4- Cover between top of corbel and primary reinforcement greater than intended.
- 5- Cover between sides of corbel and primary corbel reinforcement greater than intended.

These factors, or combinations of them, could reduce the actual capacities of some of the corbels further. They could also result in premature local spalling of concrete of the outer edges of some corbels. Critical to the overall load-carrying performance of the corbels are the above points (1, 2, 3); as shown in Figure.2.

A set of measurable criteria have been defined for each of these:

- X= Cover between corbel-face and down-turned primary corbel bars. [measurable by Ferro scan]
- Y= Gap between end of beam and face of column. [measurable by tape measure]
- Z= Cover between beam-end and upturned primary beam bars. [measurable by Ferro scan]

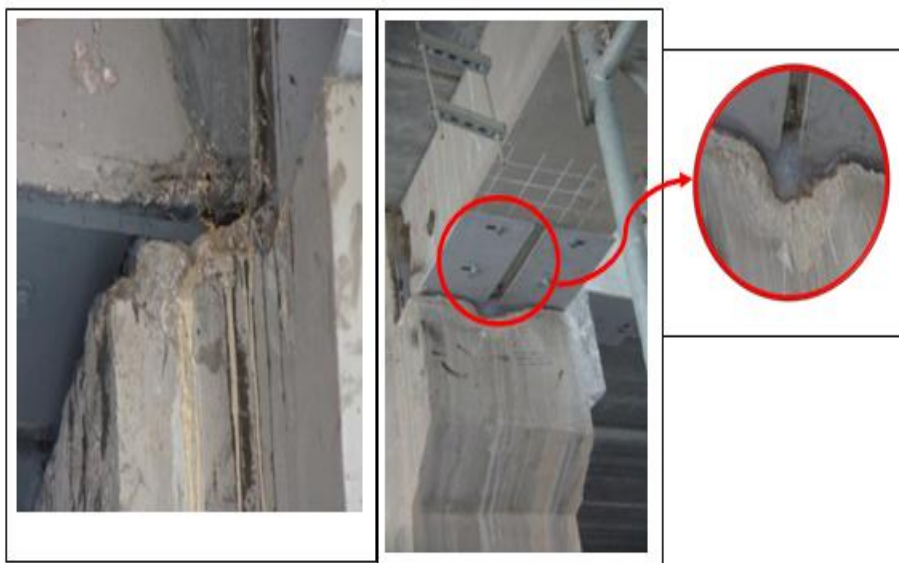


Fig.1 Corbel shortfall reinforcement detailing.

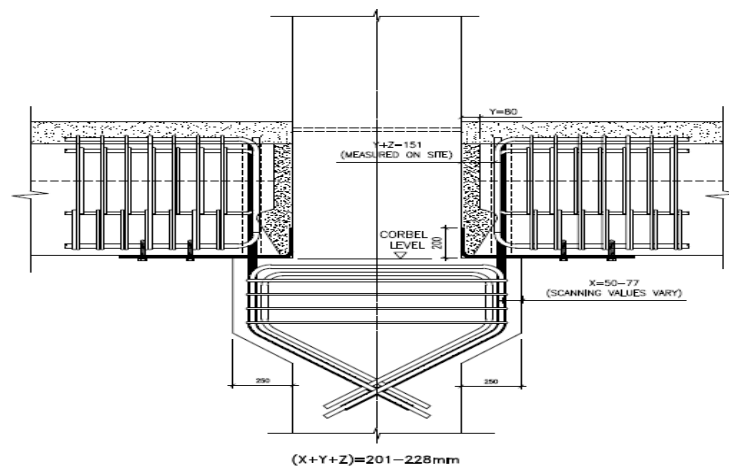


Fig. 2 Corbel deviations' parameters

III. New Concept To Create Partial Continuity At Beam Column Connections

In all precast concrete beams were effectively were designed as simply-supported at centre-point of their bearings onto their supporting corbels. This meant that the strength of the junctions was very dependent on the configuration of concrete profile and of reinforcement within the corbel and the beam-end all as described above.

It is proposed that the required partial continuity be introduced by a combination of the following:

- 1- Increasing the thickness of structure topping.
- 2- Adding dedicated additional reinforcement within the topping around internal columns.
- 3- Introducing grout between ends of beams and column faces.
- 4- Temporarily propping the precast concrete beams until the topping has achieved adequate strength.

The amount of partial continuity is therefore partly a function of the status of construction at the time that the partial continuity devices are introduced. For example:

- 1 -The minimum amount of partial continuity occurs where only the beams and hollow core have been installed prior to implementing partial continuity devices, (during construction stage, partial loaded case)
- 2 -The maximum amount of partial continuity occurs where precast beams, hollow core, topping, and perimeter cladding have been installed prior to implementing partial continuity devices, (Final construction stage; full loaded case).

A series of analysis of individual precast beams and associated beam to column junctions have been assessed for ranges described at above. Individual bending moment diagrams for simply supported condition loads, and for continuous condition loads, were derived and then non-elastically superimposed in order to arrive at series of overall partial continuity diagrams. From these main items were assessed:

- The magnitude of the value of the horizontal compression towards the bottom of the section- acting between the beam end and the column face.
- The length of compression zone established by determining the distance from column face at which the overall bending moment is zero.
- The new reinforcement details are presented clear as shown in Fig.3 to show the new proposed steel reinforcement details in the structural topping for corner and edge column, respectively.

As shown in the Figure 4, with shear key and imbed steel plate, It can be consider that a beam is able to deliver its vertical shear to appoint that is slightly short of the intended support. The aim of the additional measure is therefore to transfer this shear from the point of action of shear to the intended point of support. The distances by which this shear needs to be extended is only of the order of 100mm to 150mm. it is proposed that the additional force transfer be achieved by creating a mini strut and tie system. The main components of this mini strut and tie system are as follows:

- Thrust key formed in the end of the installed beam to allow the strut to direct the compression into the body of the beam at an angle to the vertical.
- Tie constructed from bent steel plate.
- Bolts or bonding connecting bent late to underside of beam.

The analogy that was developed as part of the development of this proposal was for that of abeam that was still simply supported. For this condition the tie forces would be simply the horizontal component of inclined strut force.

In practice, because this additional measure will be implemented in conjunction with the overall partial continuity, much of this horizontal component actually ends up as a part of the horizontal compression between the beam end and column face.

Figure 5 viewed real photos for the additional measure can be considered as a simple extension of the bottom-reinforcement towards the intended support-point of the beam, and an extension of the shear strength of the beam by means of a strut, working in conjunction with the plate-extension.

The sequence of installation and implementation of the additional measure and partial-continuity will generally be:

1. Prop beams [plus locally jack load out of beams in cases where it is necessary to do so to achieve the partial-continuity].
2. Remove existing Neoprene pads.
3. Grind / cut off chamfer to leading-edges of corbels – as safeguard against local damage.
4. Cut thrust-keys at beam-ends, by stitch-drilling [using non-diamond drill bits], and by then removing the cut-portions of concrete.
5. Insert plates [Generally from both sides so as to leave corbel-dowel intact. For more lightly loaded perimeter beams a one-sided plate solution is also being developed].
6. Grout compression struts and the gaps between beam-ends and column-faces.
7. Install dedicated additional top reinforcement around columns / into columns
8. Cast structural topping
9. Once topping has achieved the required strength – de-prop beams.

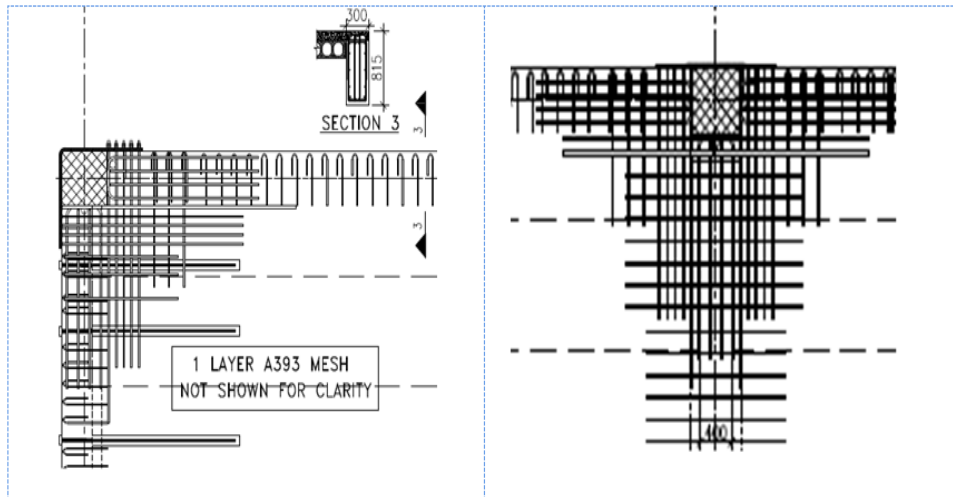


Fig. 3 the new proposed steel reinforcement details in structural topping for corner and internal column for partial continuity, respectively.

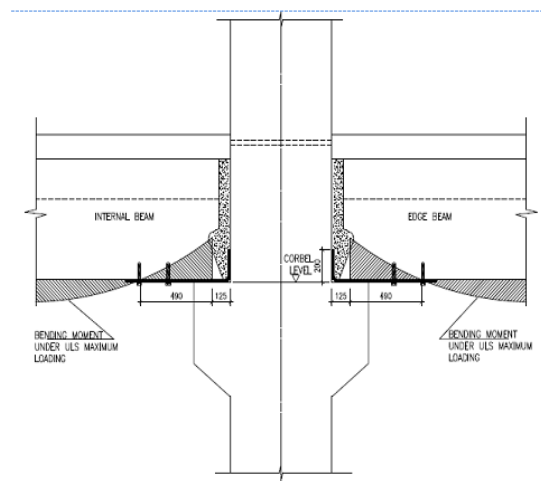


Fig. 4 Section elevation of new corbel beam connection to create partial continuity.

IV. Experimental Test Setup

In this study, analytical and numerical approaches were adopted to predict deflections, inclination angles and bending moment distribution for the Mockup structure at different loading stages which was defined before. The second part of the study describes briefly observations gathered throughout the duration of the tests at different loading conditions; Figure 6 is a real photo for the mockup sample loaded up to the final Loading Condition.

It should be noted that the majority of the analysis presented in this study is based on the collected displacement data gathered from the mockup test. This is due to the fact that strain gage data collected from the mockup test were generally very small and in many cases had fluctuations in measured values. The probable reason for these small strain values is that the beams used in building the mockup were very stiff. Also, as it is mentioned in the body of the study, the analytical procedures used in predicting the moment and the actual stiffness of the flexural members is an approximate, however, the validity of the analytical procedures is confirmed through backward calculations of the flexural stiffness. The strain measurements were used the analysis only to approximately locate a reasonable positions of the inflection points.



Fig. 5 Installation of steel plate with shear key shape at end of beam.



Fig. 6 Loading of the Mockup Structure

Figure 7 presents plan and section elevation of tested mockup sample, all precast elements columns and beams that was used in the construction of this mockup sample was selected from the rejected elements that has defects on corbel reinforcement details and beams. Hollow core slabs, selected from the same supplier but its not erected before to be sure it has the efficiency to resist applied load with the design pre camber value as per the original design.

Figures 8 and 9 present locations and real photos for the fixation of displacement and inclinometers for interior beams and non edge beams, respectively. The inclinometers was used at beam column connection to measured any rotation angle where it is very important to inspect the negative moment by mathematical integration to

The scope of the expert's work comprises of three main tasks:

- i.** Identification of locations and type of sensors needed for the verification process of the Mockup test and for assessment of the efficiency of the new proposed retrofitting system,
- ii.** Monitoring and evaluating the reliability of sensors data and determination of data collection frequency,
- iii.** Analysis of data acquired at different loadings and unloading stages and ensuring that the range of readings falls within the two extreme boundaries limited by the simply supported condition and

hypothetical fixed-end boundary scenario. This included the analysis of strain, deflection of both beams and Hollow Core Slabs (HCS), beam rotations as well as crack size and crack propagation.

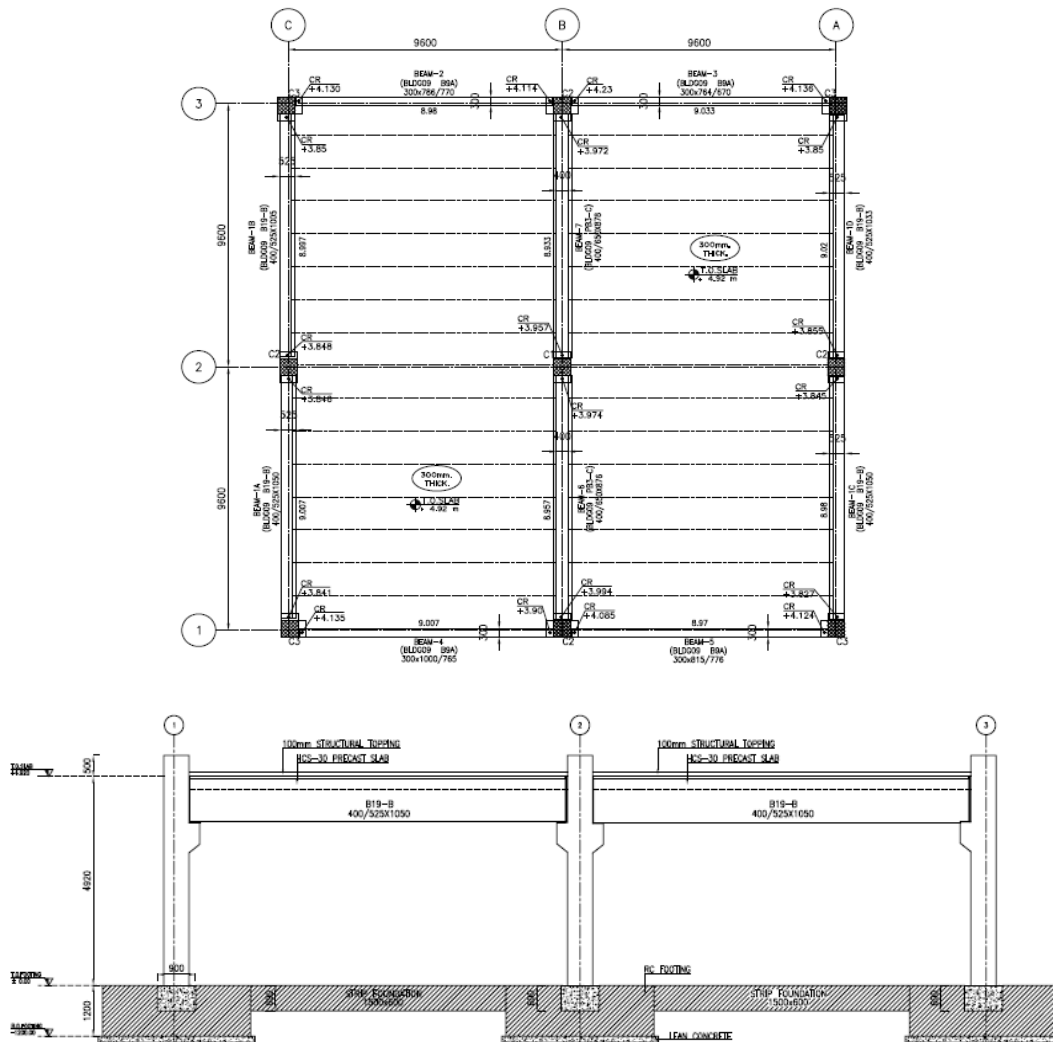


Fig. 7 Plan and elevation cross section of tested Mockup sample, respectively.

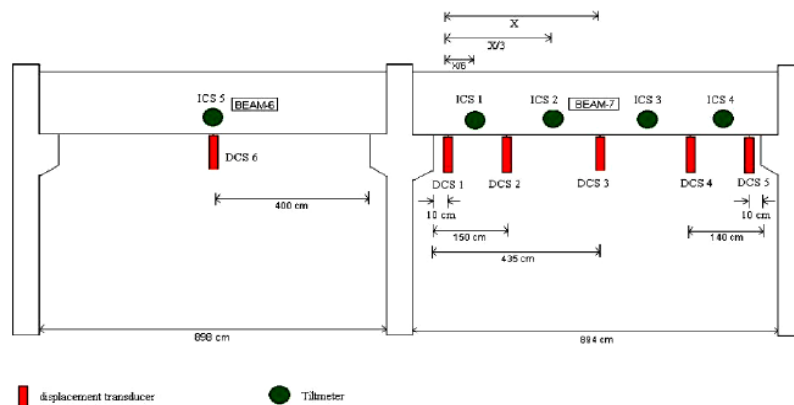


Fig. 8 Showing location of displacement and inclinometers for e interior beams.



Fig. 9 photo of fixation displacement and inclinometers for interior and external beams.

V. Mockup Test Observations And Crack Progression

In order to compare the actual performance of the retrofitted mockup structure that is expected to provide partial continuity to the as-built simply supported details, two extreme cases, with regards to the girders' boundary conditions, were analyzed using elastic beam theory. In the analysis, deflections and curvatures were calculated for the two extreme end support cases; namely, (i) The simply-supported beams scenario, and (ii) the fully continuous beams scenario. The concrete strength values used in the analysis were based on the data provided by contractor and the designer. These values included the following:

- (i) Compressive strength of existing precast members is $f_c'=60$ MPa, and
- (ii) Compressive strength of the concrete topping is $f_c'=40$ Mpa

Based on mockup test observations, where minimum or no flexural cracks were observed, especially for loading cases prior to the final case, it was decided to carry on the analysis using the Gross Moment of Inertia of the beams including the FEA (Finite Element Analysis) models. The use of gross moment of inertia is justifiable, since this is not a theoretical design problem, and in our case it was proven, by inspection, that no or minor flexural cracks were developed until the maximum loading case.

After unloading of the majority of loading stage, three types of cracks were visually observed; namely (i) transversal (parallel to the beams axes as shown in Figure 10), (ii) longitudinal (parallel to the HCS axes – Figure 10) and (iii) diagonal cracks around the column perimeters (refer to Fig.11). The transversal cracks were developed along the junctions between the hollow cores and the main girders. When the floor is loaded, both the HC slabs and the beams deflect. The mid-span deflection of the slabs depends on stiffness of both the supporting beams and slabs (see Figure 10). It is anticipated that the formation of the transversal cracks may result in a decrease in composite action. These transversal cracks usually developed due a combined effect of shrinkage of the topping materials (at these junction locations) and end rotations of the hollow cores under loading.

As shown in Figure 11, two crack lines at the girder boundaries initiated at the column edges were observed with sizes ranged from 0.1 to 0.4 mm in different locations.

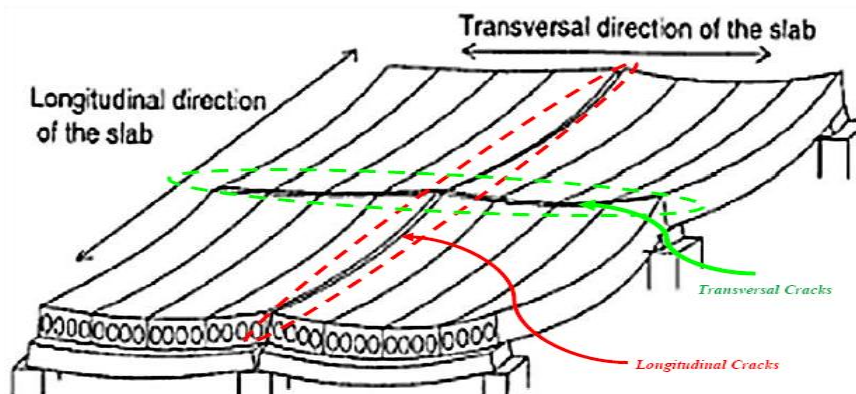


Fig. 10 Longitudinal and Transversal Cracks Formation at the HCS and Beam Interface.

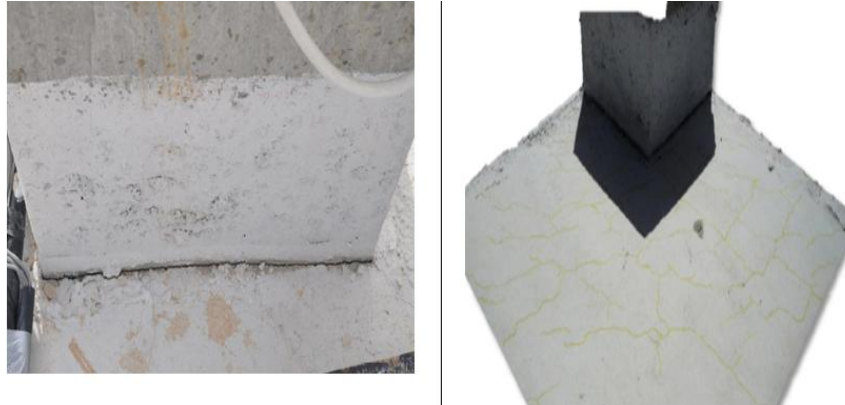


Fig. 11 Example of Crack Pattern of the Mockup Top Floor around the Column

VI. Interpretation Of Experimental And Numerical Analysis Results

In this section, the measured test data acquired at different loading stages (after the composite action is achieved and due to topping load and applied additional load on the partially continuous beams) and different locations for both interior and edge beams is analyzed and compared to the results obtained when to the extreme conditions described in the preceding sections. As shown in Figures 12 and 13, a total of twelve displacement transducers and ten inclinometers were used to capture data along the interior and exterior beams spans. However, only critical locations with relatively large deflections are considered in the analysis in order to describe the global performance of the mockup structure.

Figures 12 and 13 show the strain distribution for both the interior beams and edge beams. Unlike deflection and rotation recorded measurements, strain values gathered from the mockup tests were fluctuating and in some instants inconsistent. In order to evaluate the strain sensors used in the mockup test, tests were performed on standard steel rebar that were instrumented by both vibrating wire strain sensors as well as electrical strain sensors to check if the vibrating wire sensors were malfunctioning. Laboratory test results indicated that both sensors readings were similar that verified that the vibrating wire sensors used in the mockup structure tests had no technical problems. So, the variation can be attributed to several issues including the relatively very low strain level of both the beams and corbels with high stiffness design sections, temperature effects, and the influence of ambient vibration from cranes, equipment generated at the site on the performance of the vibrating wires sensors.

Deflection and Rotation Calculations for Continuous Beam Assumption: In order to evaluate the deflection data for the continuous beam case, the finite element models were developed and analyzed. In this model, the beam is assumed continuous at the middle support (interior corbels locations), while the exterior supports (edge corbels) were assumed to act as hinged supports. The total deflection and rotation values were determined as a sum of the deflection and rotation values generated from the two models for both non-composite and composite sections, respectively. Summary of both calculated and mockup test measured data for mid-span deflections and edge rotations are presented in Table 1.

In estimating edge beams moment distribution, it was assumed that the vertical displacement is restricted at the edges (i.e. at the corbel edges), while it is partially restricted along the corbels length (refer to Figure 14a). In this scenario and due to the lack of vertical displacement information at the hogging zone (no deflection) transducers were installed at the top surface of the hogging zone. Even if that was done, it would have been also difficult to effectively interpret these values due to the existence of the columns preventing the physical continuity of the beam creating a complicated load path.

This applies to both interior and exterior cases as shown in Figure 14b, extrapolation was used to predict the hogging moment based on the aforementioned assumptions. The partial physical continuity is provided by a two-dimensional (2D) edge tie plates. This edge tie plates are these portion of the specially reinforced concrete topping. The assumed statistical system of this complex system is illustrated in Fig.14b.

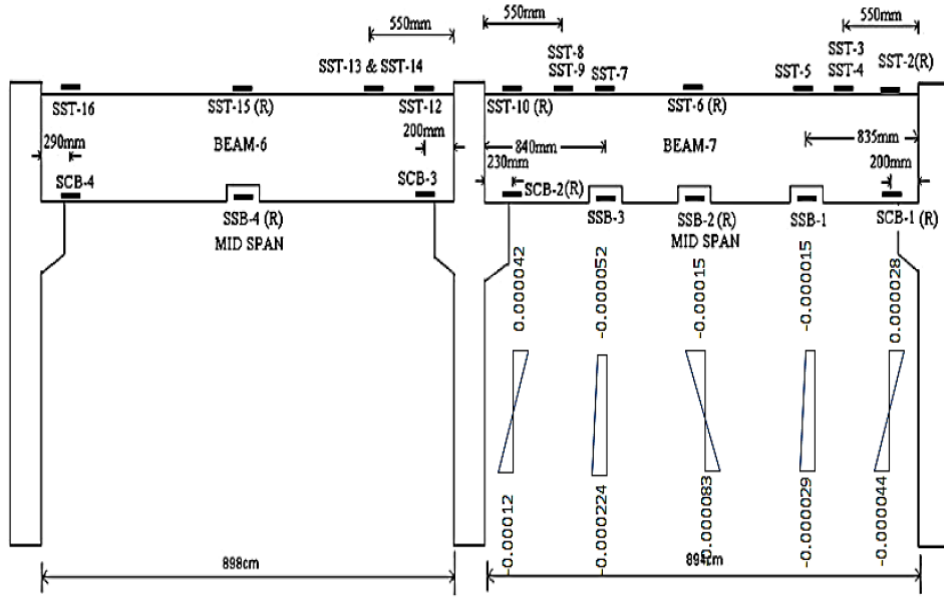


Fig.12 Strain distribution for external beam at different locations along beam length.

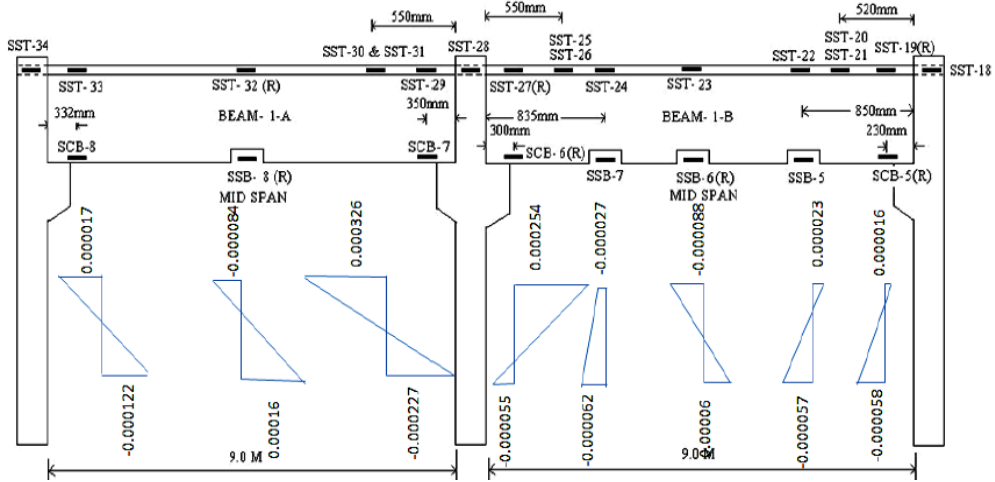


Fig. 13 Strain distribution for internal beam along beam length.

Table 1 Calculated and Measured Edge Beams Displacements and Rotations.

Loading case	Deformation	Finite element results *		Experimental Measure value
		Simply supported	Full continuity of two span	
Loading Case 1 (D+L) (9.2 kn/m ²)	Maximum deflection, (mm)	4.87	2.1	3.54
	Rotation value near beam edge support (degree)	0.092	0.031	0.059
Loading Case 2 (1.2D+1.6L) (15 kn/m ²)	Maximum deflection, (mm)	6.6	2.8	4.9
	Rotation value near beam edge support (degree)	0.126	0.043	0.175
Loading Case 3 [1.1(1.2D+1.6L)] (18 kn/m ²)	Maximum deflection, (mm)	7.4	3.2	5.53
	Rotation value near beam edge support (degree)	0.137	0.049	0.21

*calculated using the finite element model.

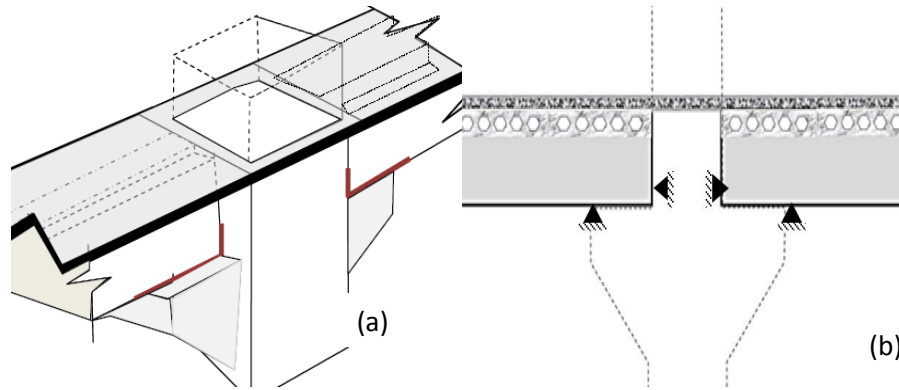


Fig.14 (a, b) An Isometric Showing the Edge Tie Plates (Surrounding the Column to create Partial Continuity Load Path), that Connect the two adjacent Beams

VII. Analysis Of Mockup Test Results

Deflection, Rotation and Curvature: Figure 15 shows the elastic curve (deflection profile) of internal Beam at the Loading case1 Condition. Using regression analysis, a best fit equation was derived for the deflection elastic curve as shown in the Figure 15. By differentiating this equation once, one can obtain the equation for the slope angle, i.e.:

$$y' = dy/dx \tag{1}$$

Now, differentiating the deflection equation twice, the curvature equation is generated, i.e.:

$$y'' = d^2y/dx^2 \tag{2}$$

Table (3) presents the different values of y' , y'' , y''' , and moment value for internal beam, while the predicted moment values are illustrated graphically in Fig.16. Now, differentiating the elastic curve (deflection) equation four times, y'''' is determined (refer to Equation 3). Knowing the value of actual applied load on the mockup structure at this loading case, a realistic (as-built) value of the beam flexural stiffness (EI) can be estimated.

$$y'''' = \frac{d^4y}{dx^4} = \frac{-w(x)}{EI} \tag{3}$$

Where: $w(x)$ is the applied load. Table 2 presents a summary of y'''' and M values for internal Beam for loading case1.

Analytical Estimation of the As-built Flexural Stiffness: The estimated value of flexural stiffness obtained using Equation (3) is used to estimate the corresponding moment values using Equation (4):

$$M = -EI y'' \tag{4}$$

Loading Case 1: The actual applied load on the mockup test for internal beams under loading case 1 is 80.16 kN/m. Using Equation (3), the value of the EI (As-built) was calculated to be 1,539,170.51 kN-m². It should be noted that the EI(As-built) value obtained from the Mockup test data is EI, after concrete topping curing during all stages of loading (composite action).

$$EI_{\text{theoretical}} = 36,406,043 \times 0.05 = 1,820,302.15 \text{ kN/m}^2$$

Comparing this value to that calculated from the mockup data analysis (1,539,170.51 kN/m² versus (1, 820,302.15 kN-m²), predicted stiffness value is about 85% of the theoretical stiffness.

Table 2 Values for y' , y'' , y''' , and moment value of internal beam under loading case 1

Distance along span (meters)	0.0	1.73	4.53	7.05	9.00
y'	-0.0012	0.0000	0.0010	0.0013	-0.0011
y''	-0.0001	0.0005	0.0003	0.0000	0.0002
y'''	-0.00005	-0.00005	-0.00005	-0.00005	-0.00005
M (kN.m)	-153.92	365.19	696.99	458.28	-75.79

Figure17 is bar columns presents the fixed end moment for full continuous beam compared to the predicted end moment measured from mockup sample (partial continuous beam), of mid and external beams

respectively. The moment value for mid beam is higher than the edge beam where the partial continuity for the mid beam is more improved than the edge one; which is expected. As shown in Figure 17 Fixity ratio was about 28% up to 37% depend on the load cases for mid beam where the fixity ratio of edge beam for the same load cases about 14% to 25%.

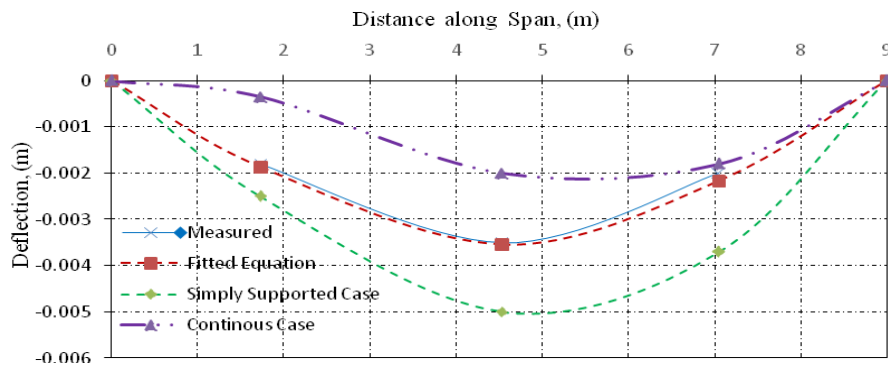


Fig. 15 Deflection beam value of continuous, simply, numerical, and measured beam value of mockup sample.

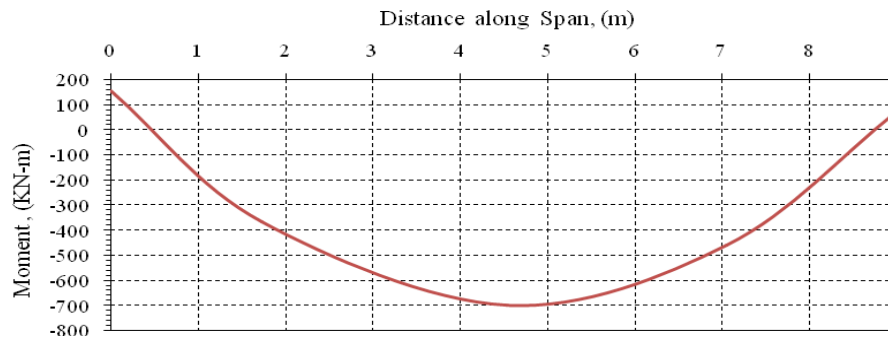


Fig. 16 Predicted moment of internal beam under loading case 1.

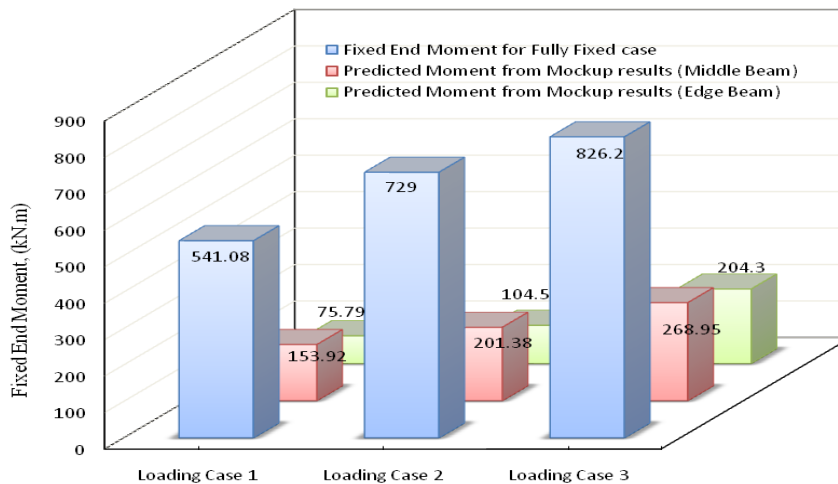


Fig. 17 Comparison between fully fixed end moment beam to middle, and edge beam, respectively; under different loading cases.

VIII. Conclusion

Based on mockup test measured and numerical analysis results data, the following conclusions and recommendations are drawn:

1. No significant visible local damages were detected up to 110% of the ultimate design load, in beams, corbels, hollow core slabs and columns,
2. However, cracks were observed at the topping as early as service load level. Four types of cracks were observed namely: (i) Transversal cracks parallel to the beams' axes, (ii) Longitudinal cracks parallel to the hollow core slabs axes, (iii) Diagonal cracks around the columns zones, and (iv) Separation cracks around

the columns perimeters. Reasoning of these cracks was discussed earlier in this report and suggested remedial procedures were also provided. This includes the addition of steel wire mesh at the top surface of the topping at the column zone where longitudinal cracks likely to initiate and on along the twice the width of the beam where transversal cracks are initiated or developing other additional structural details to prevent the initiation and growth of such cracks,

3. Cracks were also observed in some locations at the vertical grout/column faces that can be attributed to both shrinkage of “controlled-shrinkage” grout used in building the mockup structure as well as possible minor losses of partial continuity at higher applied loads.
4. Based on the mockup structure test results and data analysis performed in this paper, it was found that the retrofit system was capable to provide partial continuity ranging from 28 to 33% of the theoretical fixed support case at the mid support of Beam 1B, and partial fixity of 14 to 25% of theoretical fixed support case depending on the loading level as described before.
5. The deflection values recorded are far less than the values permitted by the Building Code Requirements for Structural Concrete and Commentary (ACI 318-08).
6. For durability assurance, galvanized steel plates and stainless steel bolts should be used. Care should be taken to paint the drilled holes in the horizontal leg of the steel angles.

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