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Investigation on an Innovative Way to Connect RC Beam and Steel Column

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Abstract**—**An experimental study was performed to investigate the behavior and strength of proposed technique to connect reinforced concrete (RC) beam to steel or composite columns. This approach can practically be used in several types of building construction. In this technique, the main beam of the frame consists of a transfer part (part of beam; Tr.P) and a common reinforcement concrete beam. The transfer part of the beam is connected to the column, whereas the rest of the beam is connected to the transfer part from each side. Four full-scale beam-column connections were tested under static loading. The test parameters were the length of the transfer part and the column properties. The test results show that using of the transfer part technique leads to modify the deformation capabilities for the RC beam and hence it increases its resistance against failure. Increase in length of the transfer part did not necessarily indicate an enhanced behavior. The test results contribute to the characterization of the connection behavior between RC beam - steel column and can be used to calibrate numerical models for the simulation of this type of connection.

Keywords**—**Composite column, reinforced concrete beam, Steel Column, Transfer Part.

I. Introduction

TRANSFERRING the load from beam to the column by a safe way is considered one of the critical issues which have been studied by many researchers in many fields. The failure of the connection between the column and the beam is one of the major reasons which causes structures failure and has a great effect on the acceleration of collapse. It can be stated that conventional building construction depends mainly (at two type of materials; concrete and steel which enter as a basic compound) on the most three common types of structural elements: reinforced concrete, steel and composite elements. Many researchers have investigated different types of connection between beam and column [7]. Their techniques to connect beam to column changed according to the material, method of construction and the expected loads. Parra-Montesinos, Dasgupta and Goel [2] presented and developed a new connection design that would allow using of FRC-encased steel truss members in earthquake-resistance RC framed construction. Elremaily, Azizinamini [3], [4] presented and developed an economical connection detail for connecting steel beams to concrete filled tube (CFT) columns. The results indicated that the capability of this detail to develop the full plastic flexural capacity of the beam can be achieved when the strong column–weak beam criterion is followed. Seismic performance of the concrete filled U-shaped steel beam (TSC beam)–RC column connection has been studied by [6] and special detail using diagonal re-bars and welded re-bar connections was used to strength the beam-column joint. Chen, Lin and Tsai [8] elucidated the cyclic behavior of connection between a steel beam and a welded box column. The results indicated that brittle fracture occurs at the beam flange complete joint penetration weld and in the weld access whole region, because the stresses are concentrated in these regions.

The study in this research investigates a proposal to keep the failure location inside the structure and far from joint and columns area. The proposed technique for connecting bare steel or composite column to the RC beam [1] would try to achieve two targets; first one is to avoid the collapse at the joint or the column and second one is to propose an easy innovative construction system (CAAP) that has capabilities to give high performance without a significant increase in cost. Fig. 1 gives a general idea about CAAP construction system.

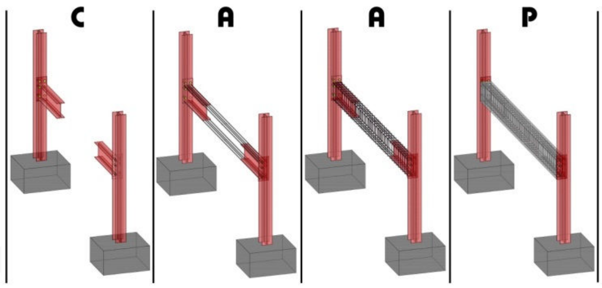

Fig. 1 Proposed Technique

II. Experimental Investigation

A. Test Specimens

The specimens were designed to represent an exterior beam-column connection. Each column element represents a half-story column in a building, and each beam element represents a part of full beam length up to contra-flexural point. Four specimens (SP1, SP2, SP3 and SP4) were built with the same dimension as shown in Fig. 2. SP1 is a control beam whereas the reinforcement bars are connected directly to the column without transfer part. SP2, SP3 and SP4 use a transfer part to transfer the load from the common reinforced concrete beam to the column. The geometry, dimensions, and reinforcement detailing of the test specimens are depicted in

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Fig. 2. The net height of all columns is 2220mm each. The beam length is 1200mm. Test program includes four specimen frames; all of them have a composite beam connected to a hinged-to-hinged column. The concrete cross section of all beams is (160mm) in width and (320mm) in depth. Overall length of each beam is (1200mm). The reinforcement of all beams is same; running along the span, there are four bars at the top and four bars at the bottom, each with a diameter of 12mm. For shear reinforcement, 8mm diameter stirrups are often placed 100mm apart along the entire length of each beam.

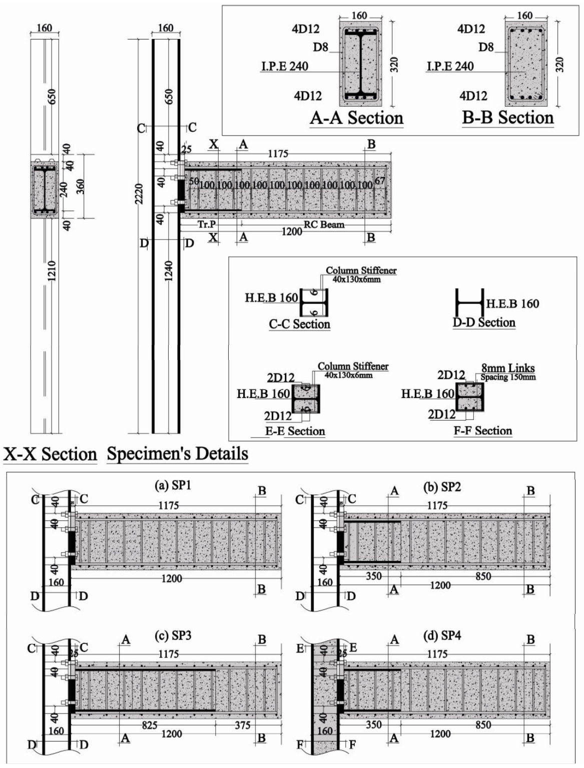

Fig. 2 All specimens details

All steel beams (IPE 240) connected to the column (HEB 160) in the same way; the steel beam is welded to an end plate which is connected to the column by using bolts [5]. This part of the beam is considered a transfer part; the remaining part is reinforced concrete one with top and bottom reinforcement. It is important to state that the reinforcement covers the whole span of the beam including the transfer part. All specimens have the same connection details which are shown in Fig. 3.

B. Materials

All specimens were cast on the same day and with the same concrete mixture. Compressive tests on concrete cubic samples (measuring 0.15 x 0.15 x 0.15 m), cast together with the specimens, were conducted to determine the concrete compressive strength. A mean compressive strength equal to 50 MPa was obtained. Steel bars are made of high tensile steel (360/520), stirrups of normal steel 240/350, steel beams and columns material are ST37.

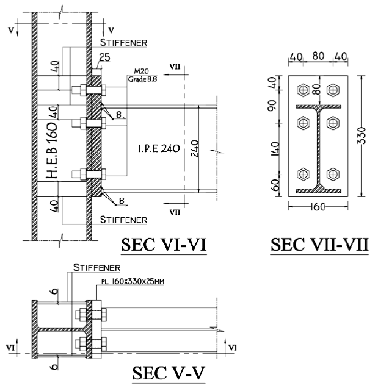

Fig. 3 Typical connection detail

C. Test Setup and Loading Pattern

Fig. 4 illustrates the test setup that was employed, indicating the ideal support and loading conditions. The column was supported by the top and bottom hinge. Static load was applied vertically at the end of the beam.

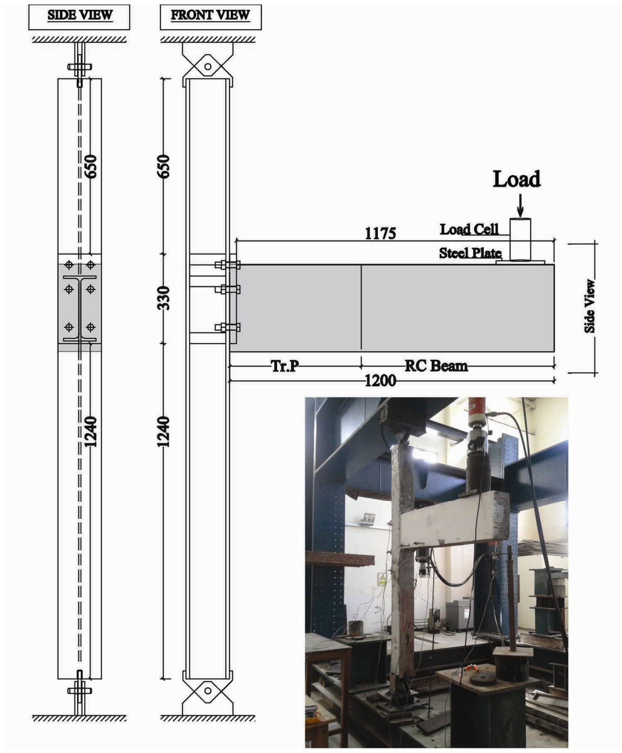

Fig. 4 Testing frame setup

D. Instrumentation

The displacement is measured by using LVDT's for both the column and the beam. LVDT's for all specimens assigned at the same location. Strain gauges of 5mm length are used to measure steel strain that are installed at several locations while concrete strains are recorded by Bi-Shape installed at two points; the first one at a distance of 50mm from the steel end plate and the second one at a distance of 350mm from the column face. Both of the Bi-Shapes are at the same horizontal line parallel to the bottom steel reinforcement.

III. Experimental Results and Discussion

Results and discussions of the experimental work are illustrated below for all specimens. Tables I and II show a summary of the straining actions at the maximum loads for all specimens.

TABLE I
ULTIMATE LOAD, BEAM CORRESPONDING DEFLECTION AND STRAIN

Name	Ultimate Load (kN)	Maximum Beam Deflection (mm)	ε_s Of ST Beam	ε_c	ε_c
			At 350mm	At 0.0mm	At 350mm
SP1	39.67	18.52	--------	0.00166	0.0
SP2	109.5	52.95	0.000682	0.00448	0.00002
SP3	121.28	52.81	0.000742	0.00356	0.00102
SP4	83.43	19.91	0.0000941	0.00016	0.00088

TABLE II
ULTIMATE LOAD, COLUMN CORRESPONDING DEFLECTION AND STRAIN

Name	Ultimate Load (kN)	Upper Chord		Lower Chord	
		Maximum Deflection (mm)	Maximum Strain	Maximum Deflection (mm)	Maximum Strain
SP1	39.67	3.06	0.000063	3.84	0.0000345
SP2	109.5	6.74	0.000435	1.256	0.0021470
SP3	121.28	3.15	0.000156	5.74	0.0039007
SP4	83.43	1.60	N.A	0.41	N.A

A. Load-Deflection Relationship

Fig. 5 shows the load-deflection curve for the all specimens' beams; it is noticeable that all specimens have a gradual deformation increasing under the increase of loading. Both of SP2 and SP3 scored high vertical displacement values, the highest of them was 52.95mm and 52.81mm respectively, which are almost the same under the effect of the Ultimate load of each one. The maximum deflection value of SP1 and SP4 is 18.42mm and 19.91mm respectively under the Ultimate load of each one. It is necessary to mention that the maximum load of SP4 is higher than the ultimate load of SP1 by 2.1 times. It can be said that the use of the proposed technique as in SP4 can double the Ultimate load without a significant increase at the deflection value.

As shown in Fig. 5 SP1, SP2 and SP3 have approximately the same behavior up to 20kN because all of them have the same column properties, the changes in behavior which are observed later is the result of variation in the stiffness of the beams. SP4 shows a higher stiffness than the other samples because of the use of composite column instead of bare steel column.

It can be clearly shown that the effect of increasing the length of Tr.P is followed by increase in the ultimate load and decrease in the deflection value. RC beam has a very low loading capacity and sudden failure can be noticed. Increasing the length of transfer part from 30% at SP2 to 70% in SP3 raised the ultimate load by 9.7% and had a minor effect on the maximum deflection values while in SP4 keeping the transfer part length at 30% of whole span and changing the type of column to a composite column instead of bare steel column recorded a ultimate load (23.8% less than SP2) and opposite deflection values (62.3% less than SP2) but the great advantage with SP4 is that it keeps the failure location at the beam side far from column joint and not at the column side as has occurred with SP2 and SP3. SP2, SP3 and SP4 achieved a good and better loading capacity than SP1.

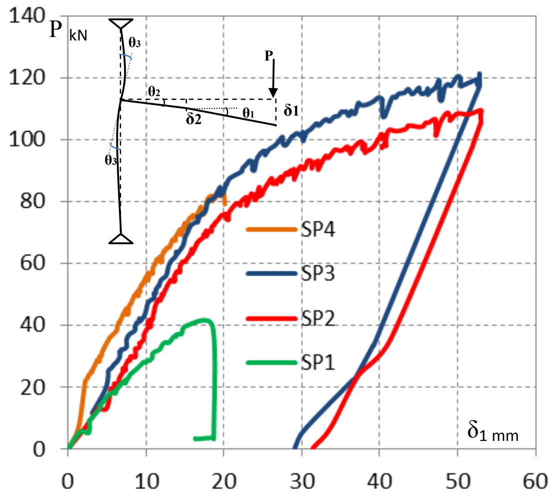

Fig. 5 Load-deflection relationship

Deflection of the beam has been measured at two points; first one at the end of the beam, second one at 350mm measured from column flange. Comparison of all specimens' deformation line at the maximum load of SP1 is shown in Fig. 6. It is observed that the use of the proposed technique at SP2, SP3 and SP4 decreases the deflection values along the beam. Using 70% of the length of the whole span as the length of the transfer part at SP3 results in a 23% decrease in deflection value as compared to SP2 where the length of the transfer part is only 30% of the entire length of the beam. Although same specifications and dimensions of transfer part are used in SP2 and SP4, recorded deflection values of SP4 are the least of all specimens because of the use of the composite column instead of bare steel column at this specimen.

B. Load-Rotation Relationship

Results of three rotation angles which are obtained through deflection values are studied for having more knowledge about the specimens' behavior. These angles are named θ_1, θ_2 and θ_3. θ_2 is measured at 0.0mm and θ_1 at 350mm, measured from column flange.

Fig. 7 shows that all specimens have more stiffness than SP1. SP4 has the highest initial stiffness but once the loading is recorded 47.5% of the SP4' Ultimate load (39.64kN), its stiffness is considered almost identical with the SP3 stiffness up to 90% of the SP4' maximum load (75 kN). This behavior gives a general idea about the mutual influence of both of beam and column components of the full system.

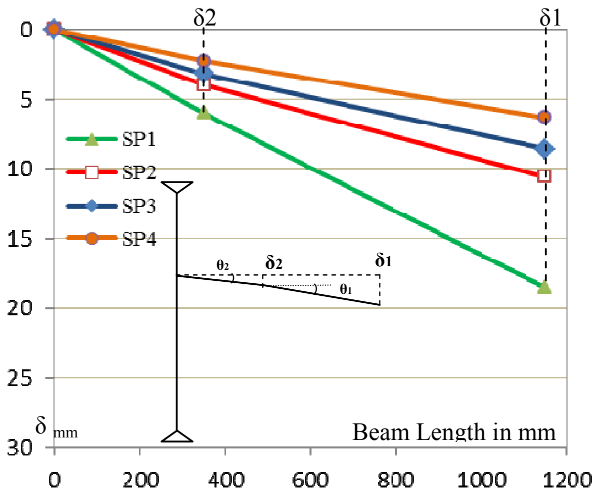

Fig. 6 Beams Deformation Line for All Specimens at the Ultimate load of SP1

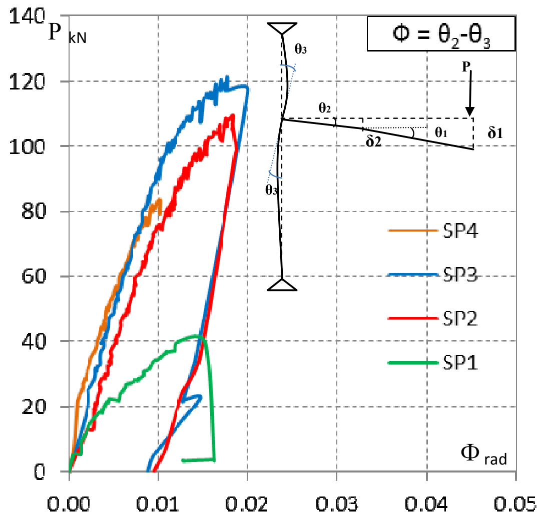

Fig. 7 Load- Rotation relationships

It is noticeable that the relation between the angle at the joint (θ2) and the angle at 30% of the entire length (θ1) which defined by θ2/θ1 decreases incrementally after 40% of the ultimate load of each specimen and continue at decreasing up to the failure. Column angle (θ3) of SP2, SP3 and SP4 record an increase up to failure, see Fig. 8. The ratio of θ_2/θ_1 for all specimens is limited between 1.0 and 1.5; the lowest ratios are recorded for SP4. SP2 and SP3 show approximately the same value of θ_2/θ_1 at 80% of the Ultimate load of each one; afterwards this ratio keeps decreasing up to the Ultimate load for each one. SP3 is the specimen which has the lowest response in changing the ratio of θ_2/θ_1 because of the continuity of the same transfer part section for about 70% of the whole span.

SP2 shows a big decrease in θ_2/θ_1 ratio; the maximum recorded ratio was 1.49 at 40% of the specimen's ultimate load ($0.4P_{2u}$) and the lowest one was 1.17 at the specimen's ultimate load (P_{2u}) which means that at the time which the load increase by 60% from 40% to 100%, the ratio of θ_2/θ_1 decrease by around 22%. The lowest ratio of θ_2/θ_1 recorded for SP4 was 1.07 at its ultimate load (P4u). It can be said that the use of the transfer part to transfer the load from RC beam to the column functioned successfully to work within the whole beam as one unit.

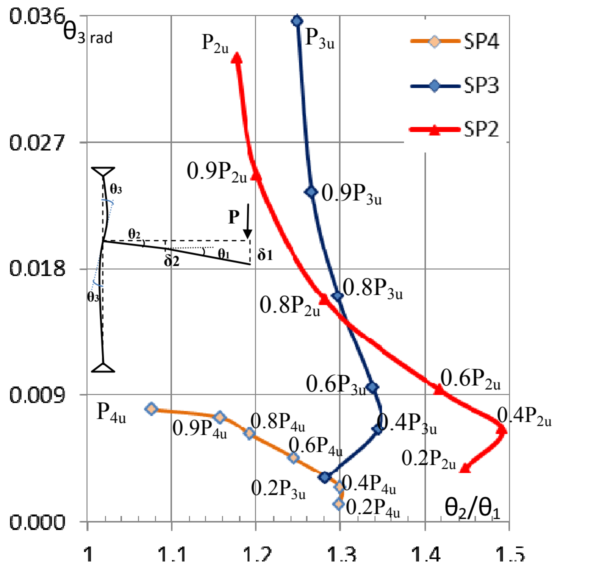

Fig. 8 Relation between the Beams Rotation Angles and the Column angle

IV. Failure Mode and Analytical Discussion

The failure criteria of any element can be projected according to the location of the plastic hinge. In this study, plastic hinge have two expected location; (1) first one is before the transfer part in the reinforced concrete area (RC) beam and (2) second one is located at the joint where the beam section is a composite one, see Fig. 9.

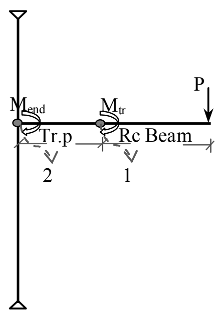

Fig. 9 Models static system

It has to be stated that the stress ability of the beams sections inside a frame is not the only factor for keeping the performance at its peak. The column is one of the highly affected parameters which have a great effect on the behavior of the full frame. All frame elements have to have the capability not only to carry the applied load but also to transfer it in a safe way.

The nominal moment of beam, both as the reinforcement concrete part and the composite part is provided in the next paragraph.

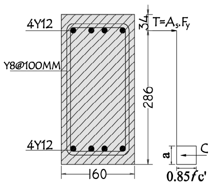

Fig. 10 Beams RC Section

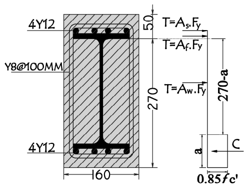

Fig. 11 Beams composite Section

A. Sections Capability

It was adopted by ACI in 1956 from the rules of equilibrium that Compression force (C) must be equal to tension force (T). Concrete stress of $(C = 0.85f_c')$ had been defined in ACI section 10.2.7.1. M_n using the equivalent rectangle is obtained from Fig. 10 as:

$$C = T$$
$$0.85 f_c' a\, b = A_s\, F_y$$
$$a = \frac{A_s\, F_y}{0.85 f_c' b} = 39.05mm$$
$$f_c' = 0.8 * 50 = 40N/mm^2$$
$$b = 160mm \;\; d = 286mm$$
$$M_n = 0.85 * f_c' * b * a * \left(d - \frac{a}{2}\right) = 56.60kN.m$$

A chance of slipping between steel bars and steel section is not being considered in this scenario of fully encased steel section and that is why balanced equation $C = T$ is provided to calculate the nominal moment, see Fig. 11.

$$C = 0.85 f_c' a\, b$$
$$T = A_s\, F_{ys} + A_f\, F_{ya} + A_w\, F_{ya}$$
$$A_s\, F_{ys} = 452 * 470$$
$$A_f\, F_{ya} = 120 * 9.8 * 370$$
$$A_w\, F_{ya} = (270 - a) t_w * 370$$

From the equilibrium equation $C = T \rightarrow a = 163.8mm$

$$M_n = 0.85 * f_c' * b * a * \left(d - \frac{a}{2}\right) - A_w.F_y \left(\frac{270-a}{2} + t_w + \frac{Rft\ Steel\ Dia}{2}\right) - A_f.F_y\left(\frac{t_w}{2} + \frac{Rft\ Steel\ Dia}{2}\right) = 162\text{kN.m}$$

As shown above the nominal moment of the reinforced concrete section is less than the nominal moment of the composite section by more than 50%.

Elastic and plastic moment of the bare steel column is calculated below where the Elastic Modulus (Z_x) equal to 312cm^3 and the Plastic Modulus (S_x) equal to 354 cm^3.

$$M_y = F_y.Z_x = 370 * 312000 * 10^{-6} = 115.44\ kN.m$$
$$M_p = F_y.S_x = 370 * 354000 * 10^{-6} = 130.980\ kN.m$$

As per calculation; the highest ability to carry moment is for the composite section while bare steel column is not able to carry the same amount of moment without going through plastic deformation. Table III shows moment and transfer moment values for all tested specimens.

TABLE III
MAXIMUM MOMENT AT THE END AND AT THE TRANSFER POINT

Name	M_{end} (kN.m)	M_{Tr} (kN.m)
SP1	47.60	--------
SP2	131.40	82.125
SP3	145.53	30.32
SP4	100.12	62.57

B. Failure Modes

There wasn't an occurrence of sudden failure in any specimen. All specimens show flexural cracks along the beam before failure. SP2 and SP3 have the same failure mode while both of SP1 and SP4 have a different mode of failure. The reason for the SP1 failure is the splitting of the steel bars as shown in Fig. 12. The split occurred at the moment value which was 15% less than the nominal value of the moment. Had the nuts been longer, the split could have been avoided.

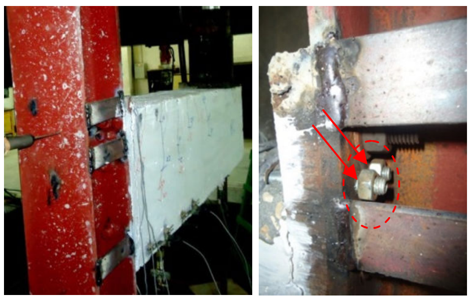

Fig. 12 Cracks in SP1 Beam and Splitting of the Steel Bars

The existing of the transfer part by 30% of the whole span as in SP2 arise the capacity of the reinforced concrete section, that the ultimate moment of the used concrete section is 56.60kN.m whiled the applied one before failure was 82.12kN.m and no failure is occurred at the RC section or the composite section which appeared a moment by a value of 131.40kN.m without failure.

Fig. 13 SP2; Beam flexural cracks and column deformed shape

The ability of the column to rotate under the effect of increasing applied load prevented formation of a plastic hinge at the transfer area between reinforced concrete and composite beam. If the column did not have an ability to rotate, failure would have occurred at the transfer joint. It is distinctly shown that the composition of beam succeeded in transferring the load to the column. In other words, the over lapping length of steel reinforcement along the steel beam and confinement it by stirrups enabled the two parts of the beam to work as a one unit; see Fig. 13 for beam flexural cracks and column plastic deformed. Same behavior of SP2 is repeated at SP3 that the plastic deformation of the SP3 column was the major defect. Flexural cracks which appeared along the beam were minor as although the column strain value is recorded 3900microstrain at the maximum applied load, see Fig 14.

It has to be mentioned that the extra loading on specimens SP2 and SP3 did not show mentionable response. The maximum applied moment at the SP3 specimen was 145.53kN.m which was less than the ultimate capacity of the composite section but higher than the plastic moment of the column.

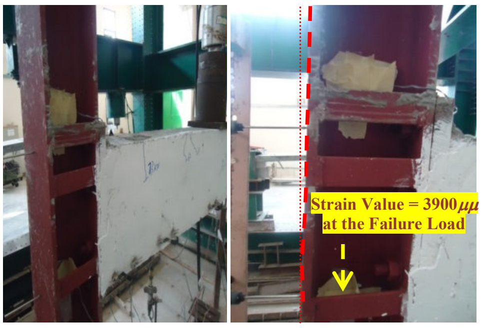

Fig. 14 SP3; Beam flexural cracks and column deformed shape

Column is deformed at both SP2 and SP3 in a way which shows that the stresses on column exceeded the elastic limit and it is running at the plastic stage. Once the load was released, column didn't back to its original shape. As mentioned before, M_p of column is equal to 130.98kN.m while the applied moment at the columns of SP2 and SP3 is 131.40 and 145.53kN.m respectively. Undoubtedly, failure of both samples happened because the applied moments on their columns was higher than their capacity in elastic stage.

Fig. 15 Cracks at the failure of SP4

SP4 failure, being "flexural failure", is the ideal one where the failure occurred at the beam side in the transfer area as shown in Fig. 15. Moment at failure was 62.57kN.m at the transfer area which was higher than the nominal moment of the beam reinforcement at this point "56.60kN.m" by 9.4%. It is worth mentioning that the use of composite column section has decreased the ability of column to rotate which can be considered the main reason for the failure at the beam side. As shown in Fig. 15, the major crack which started from top to bottom of the beam is occurred at the end of steel beam and all other cracks were minor until the appearance of the major crack. All other cracks were a natural result of increasing tension force on the top reinforcement of the beam. Changing the column type to a partially encased column instead of bare steel column prevented the failure of the column and kept it far from joint. It must be mentioned that although exaggerated increase of the transfer part could increase the failure load but by doing that, the failure mode may change to shear failure or the failure may shift at the column side.

V. Conclusion

The main conclusions of this investigation can be summarized as:

1. Use of the transfer beam (Tr.P) is considered a new promising technique to decrease the beam required depth and modifies the beam behavior.
2. Use of of Tr.P increases the beam capacity with minimum steel requirement.
3. The existence of Tr.P helps to avoid the sudden collapse of the structure.
4. Replacing bare steel column by a composite column give the advantage of shifting the collapse from the column.
5. Increasing the stiffness of the column either as a composite or bare steel is recommended for avoiding column buckling.

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