Experimental Investigation On Behaviour of 'Semi- Rigid' Connections Between Slabs and Composite Columns

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الملخص

في المبانى الشاهقة الارتفاع، تم استخدام الأعمدة من القطاعات المركبة على نطاق واسع لتقليل نسبة النحافة للعمود فضلا عن زيادة حمله. في المنشأت الهيكلية من الإطارات الخرسانية يمكن إستخدام الاعمدة و الكمرات من القطاعات المركبة و فى هذه الحالة يوجد عدة أساليب لتصميم الوصلات بينهما لتحقيق نقل العزوم من خلال هذه الوصلات من عدمه. و يجب أن تتمتع الوصلات بجساءة كافية لتقليل ترخيم الكمرات بشكل مفيد ، لذلك هناك اهتمام كبير حاليا بإجراء الأبحاث على أنواع تلك الوصلات لتطوير أساليب تصميمها بحيث أن معظم الكودات والبحوث الأخيرة تعتبر أن جميع البلاطات و الكمرات والأعمدة يمكن إستخدامها من القطاعات المركبة. فى هذا البحث تم إختبار أربعة وصلات مختلفة بين البلاطات الخرسانية و الأعمدة ذات القطاعات المركبة لدراسة تأثير شكل الوصلة و جساءتها على نقل العزوم من البلاطة للعمود و شكل الإنهيار. و قد أوضحت النتائج أن كلا من الحالة B (طول اللحام بين الأسياخ و ألواح الصلب بالعمود يساوى سبعة أضعاف قطر السيخ) والحالة D (تم تدعيم لوحة الصلب العمود باستخدام لوح صلب المستمر (600 × 50 × 6 ملم)، تم ثنى أسياخ حديد التسليح بطول يساوى سبعة أضعاف قطر السيخ ثم تم لحامها على كل من لوحى الصلب الأساسى والإضافى باستخدام اللحام) تم تحسين خصائص الاتصال بالمقارنة مع العينة القياسية ، فقد تم الإنهيار تحت تأثير العزوم بحيث زاد حمل الخضوع بنسبة 10٪ و 223٪، وزيادة قيمة التشكل عند الخضوع بنسبة 6.30٪ و 14.40٪، و زيادة حمل الإنهيار بنسبة 0.0٪ و 153.00٪، و زيادة الحد الأقصى للتشكل أيضا بنسبة 150٪ و 142٪ من قيم العينة القياسية. ولذلك فإنه يجب الاخذ فى الإعتبار تفاصيل الوصلات بين الأعمدة من القطاعات المركبة و البلاطات الخرسانية المسلحة عند التصميم.

ABSTRACT:

In high rise building, the composite column was widely used because of reducing the effective slenderness of the column, as well as, the increase of its buckling load. In framed structures, there may be composite beams, columns, or both. Design methods have to take account of the interaction between them, so that many types of beam-to-column connection must be considered. Their behaviour can range from 'nominally pinned' to 'rigid', and influences bending moments throughout the frame. The connections have sufficient stiffness to reduce deflections of beams to an extent that is useful, so there is much current interest in testing connections and developing design methods for frames with 'semi- rigid' connections. Both codes and recent researches considered that all slabs, beams, and columns are composite. Therefore, an experiment program was used to investigate behaviour of composite steel column and reinforced concrete connections. Four slab-to-composite column connections have been investigated to study the behaviour of connection type on transfer bending moment from slab to column, stiffness of the connection, and type of failure. The experimental results are compared with the values of reinforced concrete slab - column connection. It is concluded that both case B (fillet welding of bent cutting bars, upwards with length equals to seven times the bar diameter (7φ), to column steel plate) and case D (the column steel plate was strengthened by using a continuous steel plate (Pl.600×50×6mm), the cutting rebar were bent upward with length equals to seven times the bar diameter (7φ), then, they were welded to both column steel plate and the additional continuous steel plate (Pl.600×50×6mm) by using fillet welding) not only

achieve but improve the connection properties when compared to the control specimen, as well as, the govern failure is flexural type. That is, the yield applied load P yield is increased by 10% and 223.0%, the yield deformation Δ yield is increased by 6.30% and 14.40%, the ultimate applied load PU is increased by 0.0% and 153.00%, the maximum deformation Δ max is also increased by 150% and 142% than the values of the control sample. Therefore, it is recommended to use details of both connections for composite steel column and reinforced concrete slab intersections.

Keywords: 'Semi- Rigid' connections; Slabs and composite steel column; Slab flexure failure; Slab shear failure.

INTRODUCTION:

In buildings, it is expensive to make connections so stiff that they can be modelled as 'Rigid'. Even the simplest connections have sufficient stiffness to reduce defiexions of beams to an extent that is useful, so there is much current interest in testing connections and developing design methods for frames with 'semi- rigid' connections. The columns are usually continuous, and the beams are attached to their external faces by connections. These are usually assumed in design to act as pin joints, but they may be 'semi-rigid' or 'rigid'. The aim of this research is to study the behaviour of connection between composite steel column and reinforced concrete slab, since no such method is yet widely accepted. The Egyptian code (ECP-DCCS 2007& ECP-SCB 2006) Ref. [1 &2] did not mention any guidance to engineer to deal with this type of R.C slab to composite steel columns. Furthermore, no details or guidance for 'Semi-Rigid or Rigid' connections were given in international cods, Ref. [3 &4]. At each intersection between beams and columns, the slabs, beams, and columns are all assumed composite. In published books Ref. [5&6], illustrated only proposed methods for three types of connection between a steel beam and the flange of an H-section steel column, as shown in Fig. 1. As well as, in recent researches, Ref. [7], three types of connections between tubular composite columns and concrete slab are investigated. Therefore, the designer engineer is responsible for design, since it is outside the scope of codes.
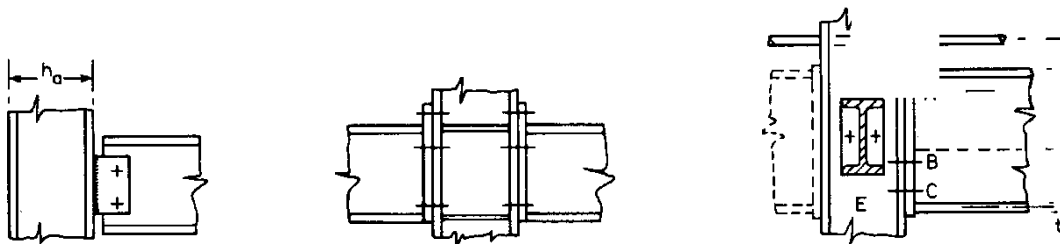

Fig. 1:   Elevations of beam-to-column connections, [6]

From above, It is clear that there is a lack of research in simulation the connectivity between composite steel column and reinforced concrete slab and moreover a concept to achieve the rigid connectivity between these two structural elements. In this paper, an experiment program was used to investigate the behaviour of frames connections 'semi-rigid, & Rigid'. Four slab-to-composite column connections have been investigated to study the behaviour of connection type on transfer bending moment from slab to column, stiffness of the connection, and type of failure.

EXPERIMENTAL PROGRAM:

An experiment program was used to investigate behaviour of these frames with 'semi-rigid' connections. Five slab-to-composite column connections have been investigated to study the behaviour of connection type on transfer bending moment from slab to column, stiffness of the connection, and type of failure, i.e. shear failure or flexible failure. The capacity of the four composite connections is compared with the values of reinforced concrete slab to column connection. The used samples of R.C. slab and Composite Steel Column consists of reinforced concrete slab of thickness 100mm with bottom and top meshes. The bottom mesh consists of main rft.10T12 @ 100mm and transverse rft. 5R8 @ 200mm. The top mesh consists of main and transverse rft. 5R8 @ 200mm. The column dimension is 200×600mm with steel plate (Pl.300×500×6mm), as shown in Figure (2). There are 6 steel bars cutting in the middle because they are intersecting with the column steel plate. In this study, four slab-to-composite column connections have been proposed to study the behaviour of each connection; their details are illustrated in Table (1).
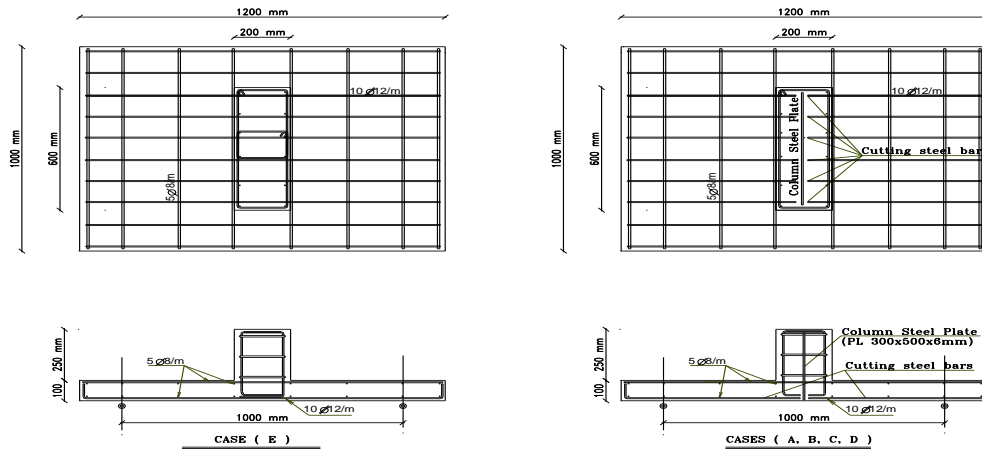

Fig. (2): Slab – composite column, Dimensions and Reinforcement

Table (1): Details of the Tested Slabs

Slab Id.	Reinforcement	Connection details	Comp. Strength fcu,(MPa)	Notes
A	+Main Rft. 10T12 @100mm	Butt welding of cutting rebar to column steel plate, with additional U shape bars equivalent to cutting bars.	25	
B		Bent cutting bars upwards with length = 7 ω, and welding to column steel plate		
C		Fillet welding of cutting rebar to steel plate (Pl.70×50×6mm), which welded to column steel plate.		
D	^Transverse Rft. 5R8@200mm	Bent cutting bars upwards with length = 7 ω, and welding to column steel plate, beside fillet welding to another steel plate (Pl.600×50×6mm) which welded to column steel plate		
E				Control

+T denotes high grade deformed bars, and the following number indicates the diameter in mm
^ R denotes mild steel, and the following number indicates the diameter in mm

MATERIALS

The used high grade reinforcing steel (45/52) had yield stress of 590 MPa and the corresponding ultimate strength is 690 MPa, was used for slabs longitudinal reinforcement. The Mild steel with fy=380 Mpa was used for slabs transverse reinforcement. A concrete grade was designed as shown in Table (1). Ordinary Portland cement (CIM 1), siliceous sand, coarse aggregates size 10 mm, was used with the quantities shown in Table (2). The average compressive strengths fcu measured at the time of testing the specimens are also shown in Table (2).

Table (2): Mix Proportions of Designed Concrete Mixes

| Slab Id.	Cement kg/m3	Dolomite kg/m3	Sand kg/m3	W/C	Actual Comp. Strength f_{cu}, MPa
A	300	1400	700	0.50	28
B	300	1400	700	0.50	26
C	300	1400	700	0.50	27
D	300	1400	700	0.50	28
E	300	1400	700	0.50	27

TEST PARAMETERS:

The main parameters examined in the experimental program are the following:

1. Direct Butt welding of cutting rebar to column steel plate, and adding U shape bars equivalent to cutting bars, Case A.
2. Fillet welding of bent cutting bars, upwards with length = 7 ɸ, to column steel plate, Case B.
3. Strengthening the column steel plate by using steel plates (Pl.70×50×6mm), which welded to column steel plate at location of cutting bars, for welding cutting rebar by using fillet welding, Case C.
4. Strengthening the column steel plate by welding a continuous steel plate (Pl.600×50×6mm). The cutting rebar were welded by using fillet welding to both column steel plate and the additional continuous steel plate (Pl.600×50×6mm), Case D.

A complete list of the selected specimens is shown in Figures (2 & 3). The cross section, dimension, and reinforcements, as well as, details of frame connections between slab and column, are shown in Figure (3).

CASE (A)

CASE (B)

CASE (C)

CASE (D)
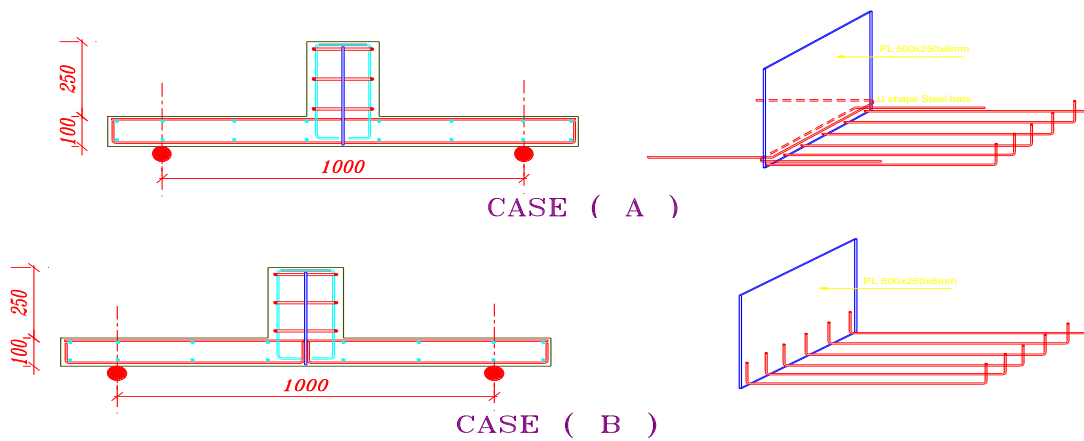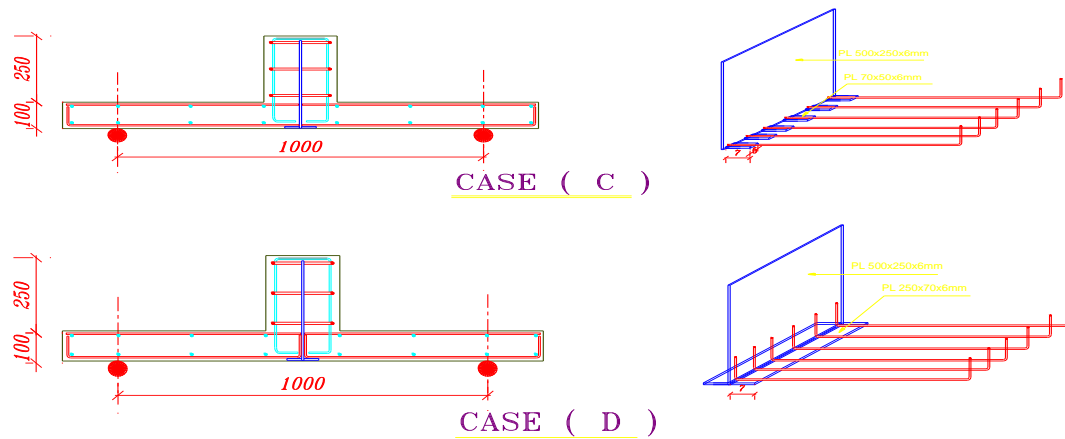
Fig. (3): Connection Details for Slabs and Composite Columns Specimens

EXPERIMENTAL SETUP

The reaction frame used in the present study is shown in Figure (4). The columns are fixed to the floor of the Laboratory of concrete structures. Resting on two of the columns is a horizontal beam to which the hydraulic actuators is attached. A Reinforced concrete block was used to support the specimen (Slab with Stud), through steel bars of diameter 80 mm, during the application of the vertical loads. Measured displacements due to deflection of the slab were detected through (four LVDT's). The first one is at slab middle and the other three LVDT are at equal distance from the middle and in perpendicular directions.
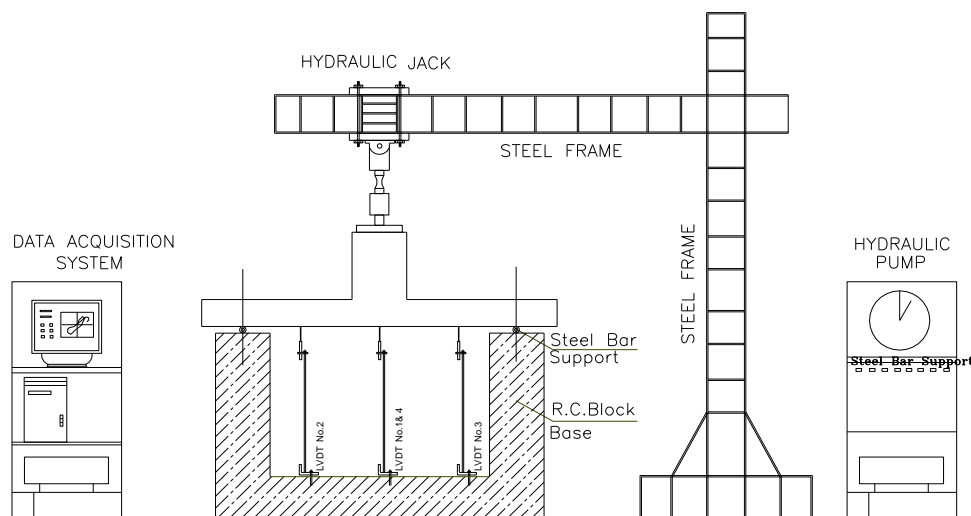

Fig. (4): Reaction Frame with Specimen Setup

INSTRUMENTATION

The data acquisition system consists of five internal control and recording channels for monitoring data from external instruments [linear variable displacement transducers (LVDTs)]. In addition to the load cells at the end of the hydraulic actuators, a series of LVDTs were used for measuring critical response quantities. As shown in Figure (5), four LVDTs were installed at the bottom of the specimen to monitor the bottom

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displacements. The foregoing system of measurements made it possible to estimate the flexural, and deformation line of the slab, as discussed in the following sections.
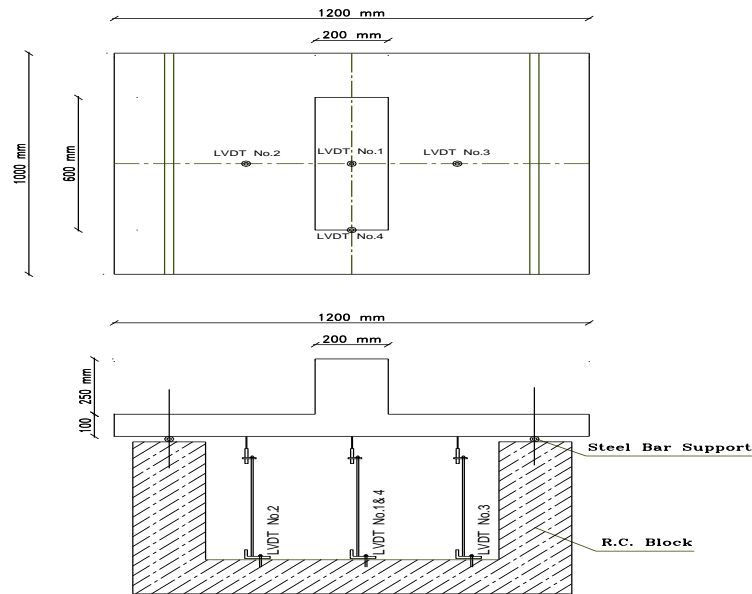
Fig. (5): Instrumentation of Typical Specimens.

APPLICATION OF LOADING:

The vertical loading was applied at the top of the columns specimens (Fig. 4). The use of a steel bars of diameter 80 mm to support slab specimens to prevent any deformation of axial loads in slab. Displacement control was used throughout the test, up to the failure point, defined as that corresponding to 70 present of the maximum strength.

TEST RESULTS:

In the following, the results of applied loads versus specimens deflections attached with picture will be presented for each case of the studied connections.

Case (A):

In this case, direct butt welding of cutting rebar to column steel plate, and adding U shape bars equivalent to cutting bars, is used in reinforcing the connection. As shown in figure (6), the ultimate load increases then suddenly reduces, with small pending deflections, not exceeding 4.50 mm. After that, it began to increases then starting to decrease sharply, with little deflections less than 10 mm. This behaviour of the first peak can be explained from figure (7), wherever, the sample has a continuous crack at the location of butt welding of cutting bars till complete separation, from column steel plate, occurs. Then, the U shapes additional bars starts to carry loads, therefore the second peak occurs, but due to both excessive flexural cracks at the middle and a diagonal shear crack occurs in the same time, a sudden drop of load is achieved. Because of snapping of loads, the maximum values of the peaks are neglected, and the P_u value is considered as the bottom datum of the two peaks. Finally, it is obvious that, the flexure failure starts at first of applying loads then at last the shear failure is the govern one.
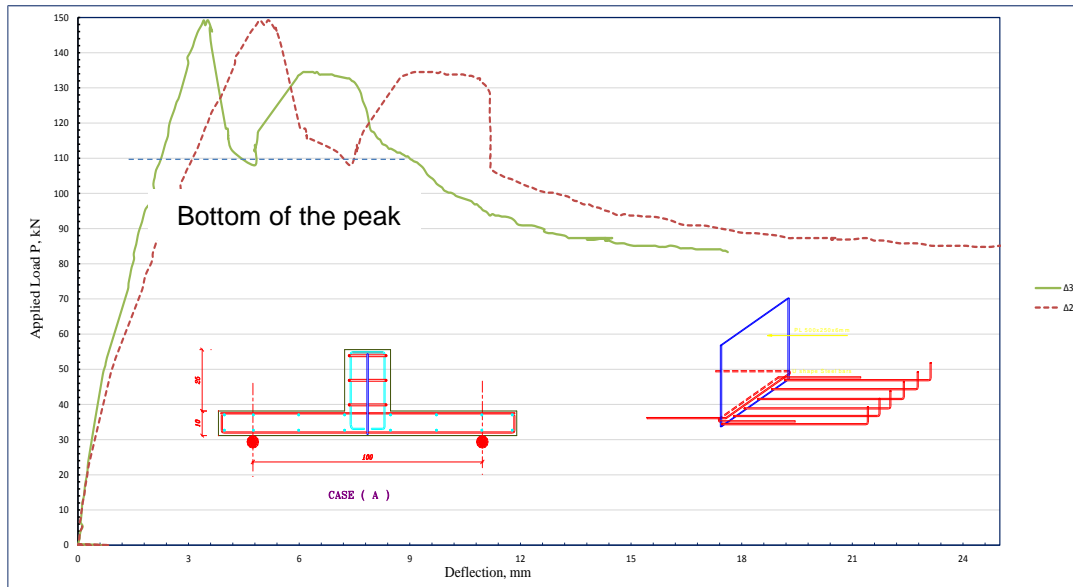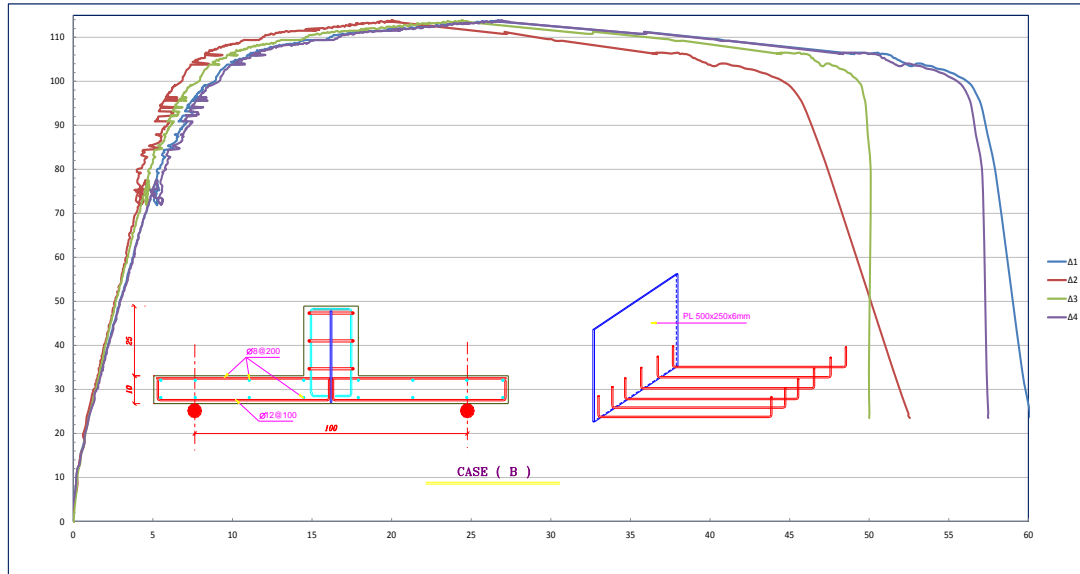
Fig. (6): Applied load versus Deflection for sample of Butt welding of cutting rebar to column steel plate, with additional U shape bars equivalent to cutting bars

Fig. (7): Flexure failure at start of loading, at last shear failure occurs

Case (B):

In this case, fillet welding of bent cutting bars, upwards with length equals to seven times the bar diameter (7φ), to column steel plate, is used in reinforcing the connection. From Figure (8), the ultimate load increases then slightly decreases, with large bending deflections, that equal to approximately 50 mm. It is recognized that, the collapse is mainly occurs due to the flexure failure but at final stage the shear failure occurs after excessive pending deformations.

Fig. (8): Applied load versus Deflection for sample of Fillet welding of bent cutting bars, upwards with length = 7 φ, to column steel plate

Case (C):

In this case, steel plates of dimensions (Pl.70×50×6mm) were welded to the column steel plate parallel to the direction of cutting bars. After that, the cutting rebar were welded to these additional plates by using fillet welding, with length equals to seven times the bar diameter (7φ), to form the connection. As shown in figure (9), the applied load increases till reaches its ultimate values with minor deflections, less than 6 mm. Then, it begins to reduce sharply with maximum deformation equal to 12 mm. It is realized that, the collapse is mainly occurs due to the shear failure accompanied with minor flexure cracks because of slippage of rebar, as shown in Figure (10).
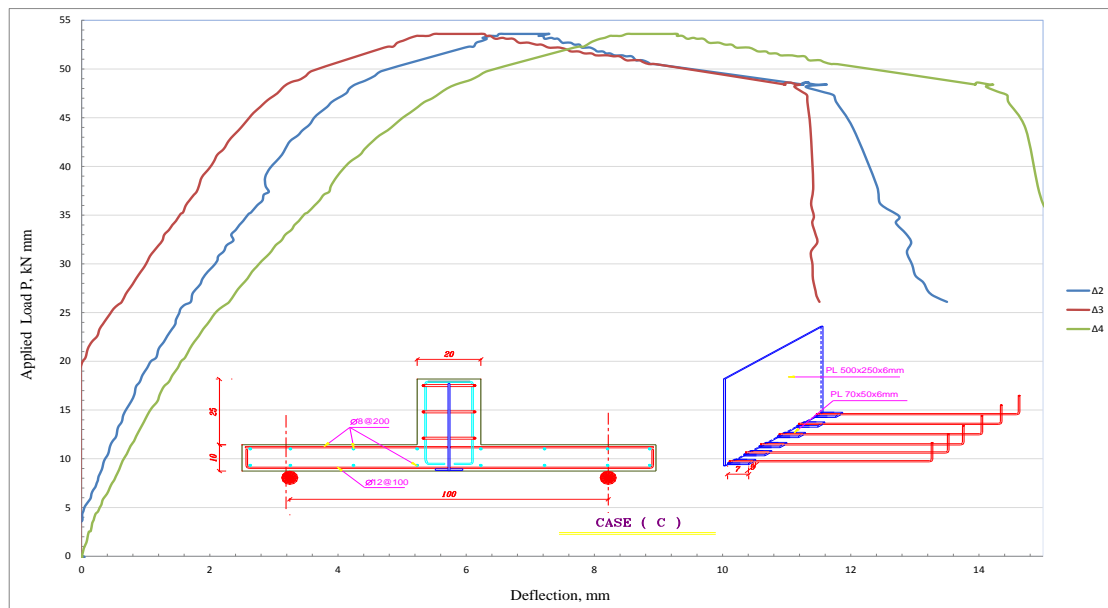
Fig. (9): Applied load versus Deflection for sample of welding cutting rebar by using fillet welding to steel plates (Pl.70×50×6mm)

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Fig. (10): Shear failure occurs accompanied with minor flexure cracks.

Case (D):

In this case, the column steel plate was strengthened by using a continuous steel plate (Pl.600×50×6mm). The cutting rebar were bent upward with length equals to seven times the bar diameter (7φ). Then, they were welded to both column steel plate and the additional continuous steel plate (Pl.600×50×6mm) by using fillet welding. As shown in figure (11), the applied load increases gradually till reaches its ultimate values with maximum deflection, greater than 50 mm. Then, it begins to reduce sharply with maximum deformation equal to 55 mm. It is obvious that, the collapse is mainly occurs due to the flexure failure, as shown in Figure (12).
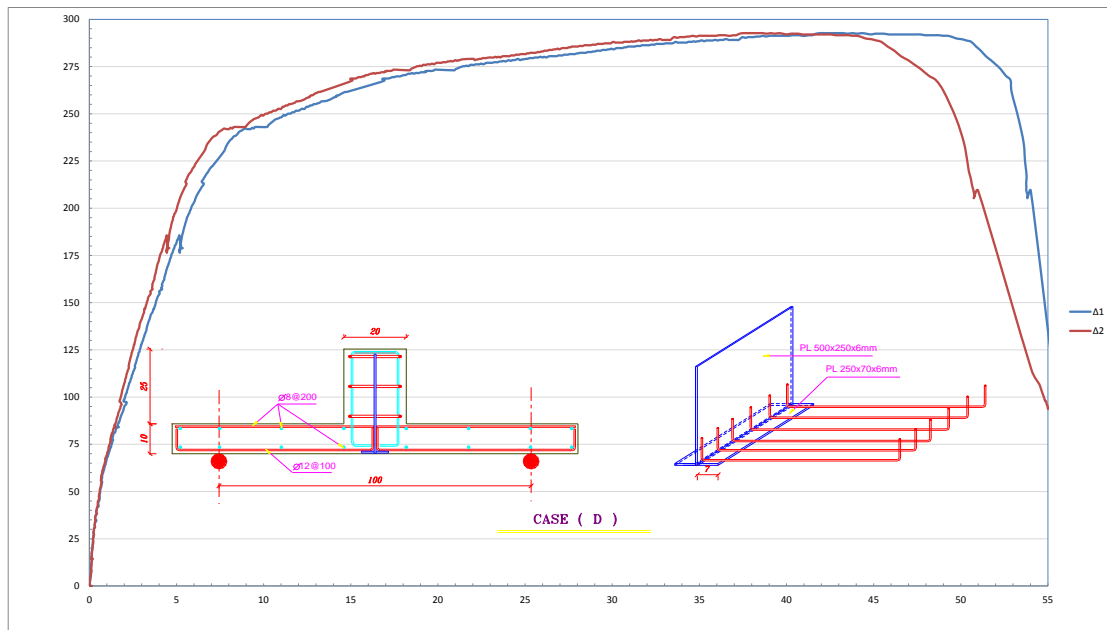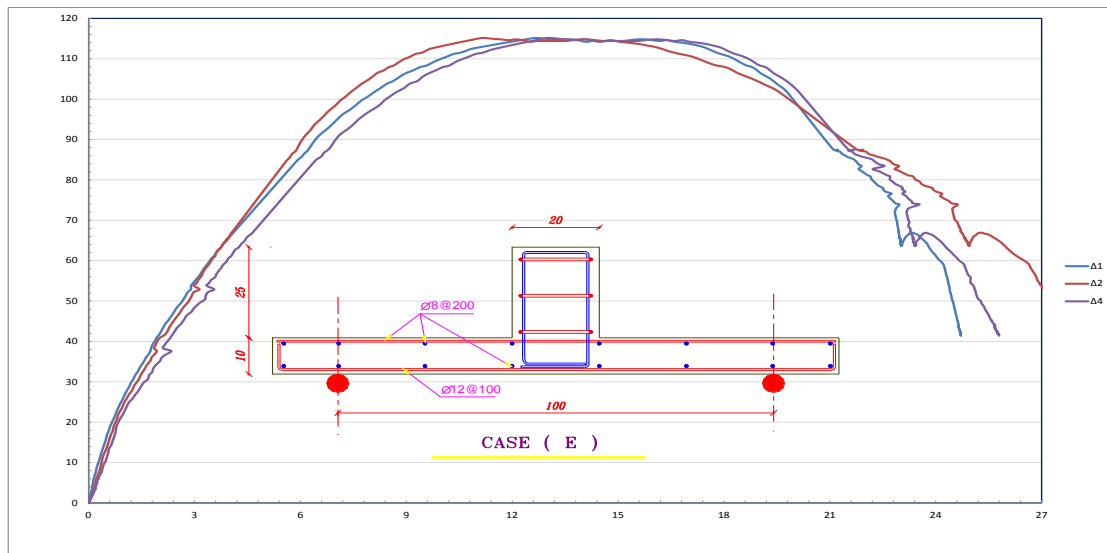

Fig. (11): Applied load versus Deflection for sample of welding cutting rebar by using fillet welding to both continuous steel plate (Pl.600×50×6mm) and column steel plate

Fig. (12): Flexure failure occurs accompanied with minor shear cracks at final stage.

Case (E), Control Specimens:

In this case, it is the control specimen, with no cutting rebar. As shown in figure (13), the applied load increases gradually till reaches its ultimate value with maximum deflection, equals to 14 mm. Then, it begins to reduce gradually till failure with maximum deformation equal to 25 mm. It is clear that, the collapse is mainly occurs due to the flexure failure, accompanied with shear failure at final stage due to excessive pending deformations, as shown in Figure (14).

Fig. (13): Applied load versus Deflection for Reinforced concrete control sample

Fig. (14): Flexure failure occurs accompanied with shear failure at final stage

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DISCUSSION OF RESULTS:

The obtained experimental results for different connection cases (Yield applied load P_{yield}, Yield deformation Δ_{yield}, Ultimate applied load P_U, Maximum deformation Δ_{max} at 0.70 P_U) are illustrated in Table (3).

Table (3) : Experimental results for different connections and control sample

Slab Id.	Case A	Case B	Case C	Case D	Case E (Control)
P_{yield} (kN)	95.90	56.50	34.00	165.70	51.30
Δ_{yield} (mm)	1.858	3.53	2.46	3.80	3.32
P_U	108.00	113.50	53.60	291.30	115.10
Δ_{PU}	9.379	26.24	7.15	43.81	13.17
Δ_{max} At 0.7 P_U	17.55	55.57	11.73	53.73	22.14

From Table (3), it is found the following:

- The yield applied load P_{yield} is greater than control sample by 86.9%,10.0%,223.0% for cases A, B, and D respectively. For Case C, it is lower than control by 33.72%.
- The yield deformation Δ_{yield} is greater than control sample by 6.30%,14.40%,for cases B, and D respectively, For Cases A and C, it is lower than control by 44.03%, 25.90%.
- The ultimate applied load P_U is greater than control sample by 153.00% for case D. For cases A and C, it is lower than control by 6.16%, 53.43% respectively. For Case B, it is almost equal to control value.
- The maximum deformation Δ_{max} is greater than control sample by 150%,142%, for cases B, and D respectively. This refers to flexural behavior of the connections. For Cases A and C, it is lower than control by 20.73%, 47.00%, and these refer to shear failure behavior of the connections.

The Ultimate Bending moment of the connection (M_U), the Ductility ($\Delta_{max}/\Delta_{yield}$), and the bending stiffness (P_{yield}/Δ_{yield}) for each case is calculated, as shown in table (4), Further, the obtained results are compared to control sample (Case E) to determine the reliability of each connection. Table (4) summarised the obtained results and the comparison between each sample to the control.

From Table (4), it can be concluded the following:

- The ultimate moment for (case D) M_U is greater than control sample by 253%. For Case B, it is equal to 98.6 % of control value. For cases A and C, it is lower than control by 6.16%, 53.43% respectively. For Case B, it is almost equal to control value. In other meaning, the connections for both cases D and B satisfies the connection moment for control sample by 2.53 and 1.0 times ratio respectively.
- The ductility μ ($\Delta_{max}/\Delta_{yield}$) is greater than control sample by 141%, 236%, and 212% for cases A, B, and D respectively. This explains the flexural behavior of the studied connections, For Case C, it is lower than control by 29%, and these refer to

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shear failure behavior of the connection. Finally, the ductility is improved for the proposed connections B, and D by more than twice the ductility of the control sample, as well as, it is almost one and half the control specimens for case A.

- The bending stiffness $(P_{yield} / \Delta_{yield})$ for connections A, B, and D is greater than connection of the control sample by 334%, 104%, and 282% respectively. This explains the flexural behavior of the studied connections, For Case C, it is lower than control by 11%, and these refer to shear failure behavior of the connection. Finally, The bending stiffness is improved for the proposed connections A, and D by two to three the times the bending stiffness of the control sample, as well as, for case A,.it is slightly greater than the value of control specimen,

Table (4): Properties comparison between different connections and control sample

Slab Id.	Case A	Case B	Case C	Case D	Case E (Control)
$M_U = P_U L/4$ (kN. mm)	27000	28375	13400	72825	28775
Connection Semi rigid ratio ($M_U/M_{U\ control}$)	0.94	0.986	**0.47**	2.53	
Ductility, μ ($\Delta_{max}/\Delta_{yield}$)	9.44	15.74	4.76	14.14	6.67
Connection Ductility ratio (Ductility $_{sample}$/ Ductility $_{control}$)	1.41	2.36	**0.71**	2.12	
Bending stiffness $_{(= P\ yield\ / \Delta\ yield)}$	51.61	16.00	13.82	43.61	15.45
Connection Bending stiffness ratio	3.34	1.04	**0.89**	2.82	

CONCLUSIONS

Currently available experimental data concerning the behaviour of proposed connections between composite column and reinforced concrete slab are studied. A series of LVDTs were used for measuring critical response quantities. They were installed at the bottom of the specimen to monitor the deflections under the connection and at the middle of the slab. The foregoing system of measurements made it possible to estimate the Ultimate Bending moment (M_U), the Ductility ($\Delta_{max}/\Delta_{yield}$), and the bending stiffness ($P_{yield} / \Delta_{yield}$) for each case of the connections. The experimental program presented herein attempted to clarify these points and shed new light on the understanding of rigid connection between composite steel column and reinforced concrete slabs. Tested Specimens (Cases B and D) failed in a predominantly flexural mode, characterized by excessive deflections at the middle. Tested Specimens (Case C) failed in a predominantly shear mode, In case A, it is recognized that, the flexure failure starts at first of applying loads then at last the shear failure is the predominant one. It was concluded that:

1. For case (A):
- The yield applied load P_{yield} is greater than control sample by 86.9%,
- The ultimate applied load P_U is lower than control by 6.16%.

- The maximum deformation Δ_{max} is is lower than control by 20.73%, and this refers to shear failure behavior of the connection.
2. For case (B):
- The yield applied load P_{yield} is greater than control sample by 10.0%,
- The yield deformation Δ_{yield} is greater than control sample by 6.30%.
- The ultimate applied load P_U is almost equal to control value.
- The maximum deformation Δ_{max} is greater than control sample by 150%. This refers to flexural behavior of the connections
3. For case (C):
- The yield applied load P_{yield} is lower than control by 33.72%.
- The yield deformation Δ_{yield} is lower than control by 25.90%.
- The ultimate applied load P_U is lower than control by 53.43%.
- The maximum deformation Δ_{max} is lower than control by 47.00%, and these refer to shear failure behavior of the connections.
4. For case (D):
- The yield applied load P_{yield} is greater than control sample by 223.0%.
- The yield deformation Δ_{yield} is greater than control sample by 14.40%.
- The ultimate applied load P_U is greater than control sample by 153.00%.
- The maximum deformation Δ_{max} is greater than control sample by 142%. This refers to flexural behavior of the connections

From above, it is obvious that, both cases B and D not only achieve but improve the connection properties when compared to the control specimen, as well as, the govern failure is flexural type. Therefore, it is recommended to use details of both connections that used in cases D, and B for composite steel column and reinforced concrete slab intersections.

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