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BOND CHARACTERISTICS AND ROTATIONAL CAPCITY FOR REINFORCED CONCERETE EXTERIOR JOINTS AS AFFECTED BY BONDED PART AND RIB GEOMETRY FOR STEEL REINFORCEMENT BARS

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The need of high quality of concrete is increased in the recent years. Using steel of high grade and maximize the benefit of using this steel became necessary. So, different ribs are used for steel bars to increase the bond strength between steel reinforcement and concrete. There are different forms of rib geometry of I he deformed bars with either Crescent or Tree profiles for all types of steel in the Egyptian market. Anchorage of reinforcing bars of the beam-column connections is an important design consideration for providing continuity and safety m reinforced concrete structures. This anchorage may be provided by development of straight bars into the exterior joints,

The Egyptian Codes of practice 208 (1995) [1] and 203 (2001) [2] give the values of development length of the deformed reinforcement bars regardless the geometry and the relative area of the ribs.

The main objectives of this research is to study the effect of the rib geometry and the relative rib area on the bond characteristics, the rotational capacity, crack pattern, the mode of failure, the deformation capacity and bond stress of the exterior joints of the structure.

Six specimens of cantilever-to-column connections, which represent exterior joints in structural system, were tested. Type of steel, rib geometry, and bonded part were variables in the tested specimens.

The study concluded that the geometry of the ribs and the bonded part in addition to the steel type have reasonable effect on the bond characteristics of the cantilever-to-column joints and hence on the ultimate load capacity of the joint

INTRODUCTION

The connections in reality are small areas at which high bending moments may take place. The assumption of fully rigid connection neglects the deformation in the areas of the joints. Thus, no relative rotation will occur between the adjoining members.

Experimental studies show that reinforced concrete and steel beam-column joints **are** deformed zones. The behavior of the beam to column connection for reinforced concrete is investigated experimentally by many researchers as Ahmed [3], Leon [4] and Springfield [5]. The partial continuity between the beam and the column is attributed to the crushing of concrete in compression zones and the cracks occur in the reinforced concrete joint which lead to bar slippage in deterioration of the bond strength between the reinforcing bars and concrete, El-Metwally [6] and Mulas [7]. The connection flexibility has a significant effect on the critical load. The results emphasize that the reduction in the critical load caused by connection flexibility should be taken into consideration.

Biddah and Ghobarah [8"] tested columns designed as compression members with the minimum eccentricity requirements of 0.1 and the anchorage length of 150mm for bottom reinforcement were provided according to the ACI code [9] specifications.

Parviz and Ki-Bong Choi [10] concluded that the values of bond stress increase with the increase of the column pressure. Although Untrauer and Henry [11] demonstrated that the increase in bond resistance at different values of slip and at ultimate load under the action of transverse compression. The influence of transverse compression increases with the increase of slip.

Parviz, Choi, Park and Asiani [10 &12] studied experimentally the effect of confinement reinforcement and compressive strength of concrete on the local bond stress-slip characteristics of deformed bars using specimens simulating the local bond condition of beam reinforcement in beam-column connections. They indicated that confining concrete by transverse reinforcement does not directly influence the local bond behavior **of** deformed bars in joint conditions where the virtual column bars are sufficient to restrain the widening of bond splitting cracks. They concluded that the ultimate bond strength increases almost proportionally with the square root of the concrete compressive strength.

EXPERIMENTAL WORK

The investigation of bond characteristics for steel reinforcements with different rib geometry and the rotational capacity of the exterior frame joints in structures are studied in this work. Six specimens of cantilever-to-column connections; which represent exterior joints in structural system; with square cross-section of 30"30 cm were tested under constant static vertical load of 30 tons applied on column and one point of loading at the free end of the cantilever.

Three specimens were reinforced with main steel bars of (BS) and three were reinforced with steel bars of (EZ). The bars were of 18mm in diameters placed in tension zone of cantilevers. Each specimen had bonded part of 5, 10, 15D and concrete compressive strength of 350kg/cm^2 for all specimens.

Specimens were tested at age of 28 days under static loading. The column was loaded by constant load of 30 tons and the cantilever was loaded gradually up to failure. Electrical strain gauges for concrete strain, dial gauges for deflection, slip, rotation, slope and the cracks propagation were recorded at the beginning and at the end of each increment of loading as shown in Figs. (l&2).

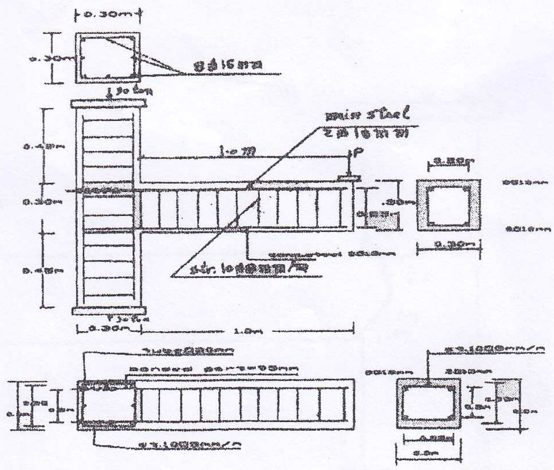


Fig. (1) : Cantilever-to-column connection specimens.

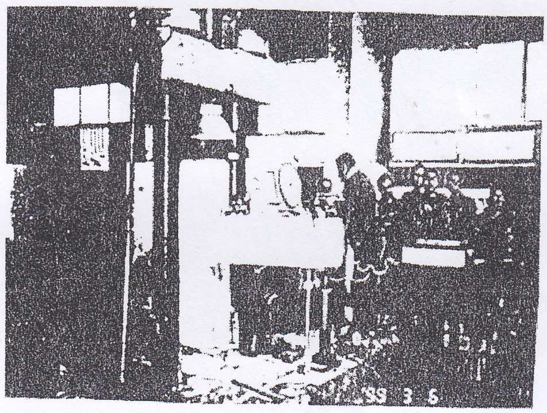


Fig. (2) : Cantilever-to-column connection specimens under test.

The properties of the deformed steel bars used in the tested specimens are given in Table (I).

Table (1); Mechanical and Geometrical Properties of Deformed bars:

Group No.	Ø (m.m)	Specimens Notation	Profile of bar	Relative rib area (α_{sR})	Yield stress Kg/cm²	Ultimate stress Kg/cm²	Elongation %	Grade of steel
H G F	18	BS	T2	0.14	4890	7230	19.4	40/60
		EZ	C1	0.06	4770	7430	20.11	40/60

TEST RESULTS AND DISCUSSIONS

The behavior of the tested specimens under loading and the modes of failures were as follows:

The initiation and propagation of the first crack was observed in the critical tension zone at maximum moment. The first crack was vertical tension crack and was at the connection between the column and the cantilever. The height and the width of crack increased with the increase of the load up to the ultimate load and then they increased without increase of the load. The major crack was formed at the same first load and the pattern of cracks is shown in Fig.(3), For the specimen reinforced by steel bars (BS) with bonded part of 15D, other cracks were diagonal tension and formed between the place of critical max. moment and of loading point as shown in Fig. (4).

The width and spacing of cracks were significantly large for a specimen with steel bars (BS). Also, the propagation of the cracks for these specimens was more than those compared for specimens with steel bars (EZ).

The final mode of failure was bond failure for all specimens; except for that with steel bars of (BS) for bonded part of 15D; was bond and flexural failure.

The geometry of the ribs and their spacing can be expressed by the relative rib area. The definition of the relative area of ribs ash **is** described by Rehm [l3] as shown in Fig. (5) as:

$$\alpha_{sb} = (K.F_R.\sin\beta_n)/(\pi.D.C_s) \qquad ....(l)$$

Where:

F_R= area of one transverse rib above the bar core

Cs = distance between transverse ribs.

k = number of transverse ribs around the bar perimeter.

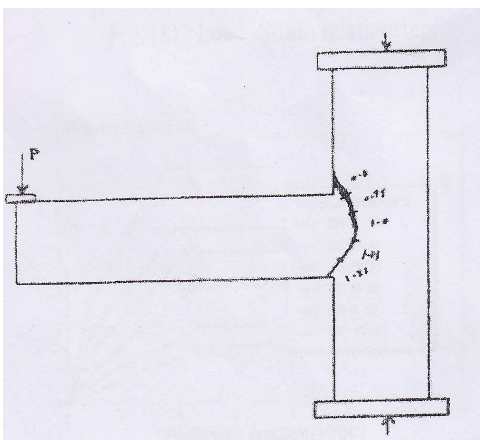
β_n = angle between rib and longitudinal axis of the reinforcement.

D = nominal diameter of bar

The measured values of deflection, end slip, strain at maximum tension zone, rotation and the slope at the free end of the cantilever-to" column connections versus the applied load are shown in Figs. (6, 7, 8, 9 and 10).

These curves can be divided into three distinct stages as follows:

a- The first stage where there was no cracks in the joint and hence it *has* a relatively high flexural rigidity



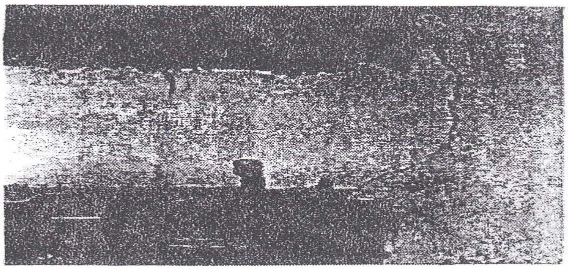
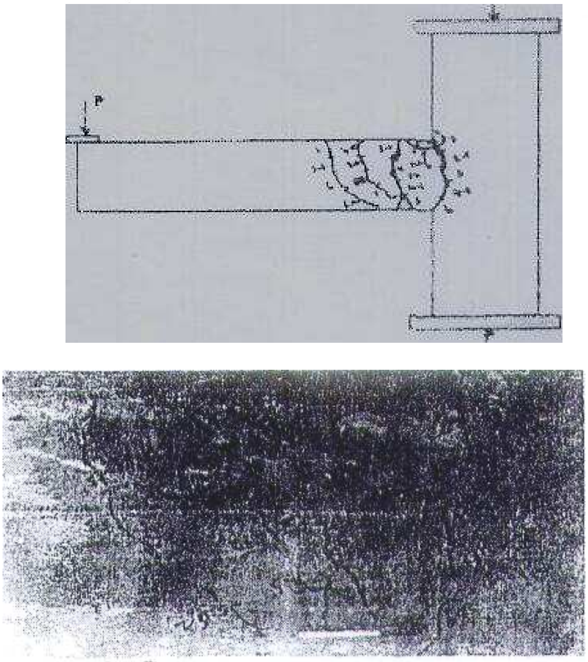


Fig. (3) : Crack shape of specimen (EZ-5D).



Fig, (4): Crack shape of specimen (BS-15D).

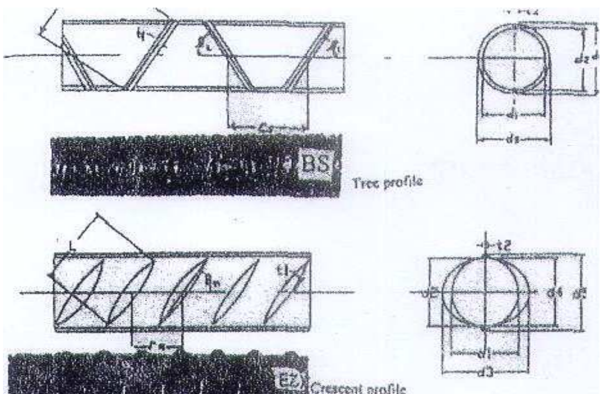


Fig.(5): Properties of the rib geometry of steel bar

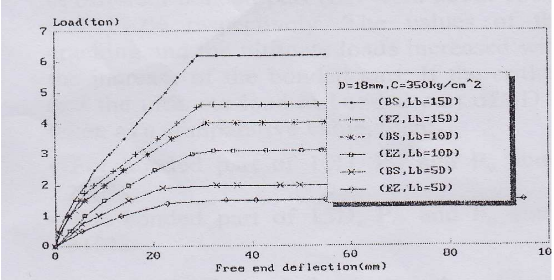


Fig .(6) : Load – deflection relationship.

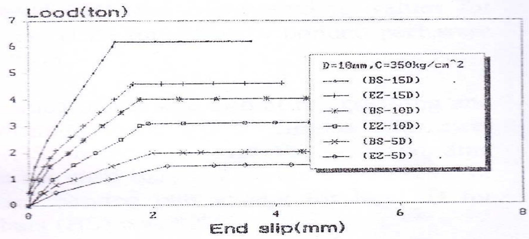


Fig .(7) : Load – end slip relationship.

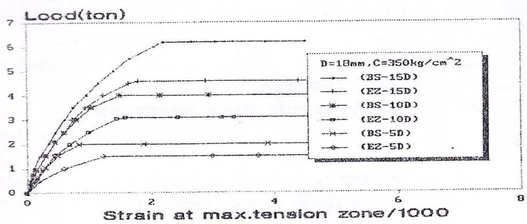


Fig. (8) : Load -Strain relationship.

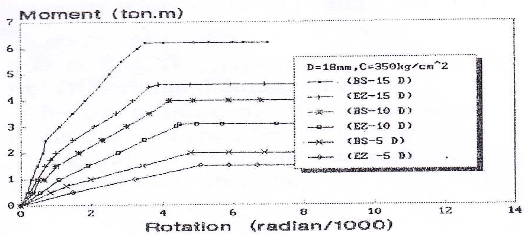


Fig. (9) : Load -rotation relationship.

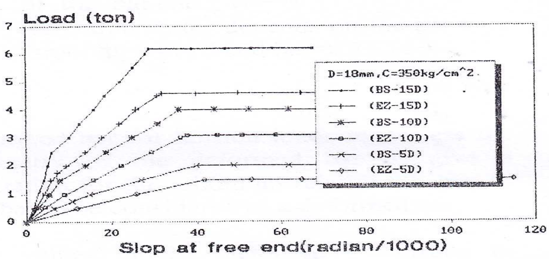


Fig. (10) : Load -Slope relationship.

b- The second stage where the flexural cracks started to form the cracks were propagated and their width and height increased too as the applied load increased.

c- The third stage where the joint of the cantilever-to-column started to fail.

The characteristics of the tested cantilever-to-column at the cracking load and at the ultimate load are given in tables (2 and 3).

Table.(2): Characteristics of cantilever-to-column connection at cracking load.

Group No.	Series	Relative rib Area (α_{rb})	P_{cr} (ton)	Deflection (m.m)	Slip (m.m)	Strain $*10^3$	Rotation (radian) $*10^3$	Slope (radian)
H	EZ-5D	0.06	0.5	5.5	0.45	0.2	15	12
	BS-5D	0.14	0.75	5.5	0.42	0.22	13	11
G	EZ-10D	0.06	1.0	6	0.39	0.25	11	9
	BS-10D	0.14	1.5	6	0.35	0.28	10	8
F	EZ-15D	0.06	1.75	6	0.32	0.33	8.5	7
	BS-15D	0.14	2.5	8	0.3	0.45	7.0	6

Table (3): Characteristics of cantilever-to-column connection at ultimate load

Group No	series	Relative rib area (α sb)	pu (ton)	Deflection (m.m)	Slip (m.m)	Striatio n strain *103*	Rotatio n (radion) *103*	Slope (radion)
H	EZ-iD	0.06	1,5	35	045	1.27	5.1	42
	BS-5D	0.14	2	33	0.42	0.06	4,g	40
G	EZ-10D	0.06	3.1	32.5	0.33	1.61	4.5	3ii
	RS-IOD	0.14	4	3i	0.35	1.51	4.5	36
F	RZ-15D	0-06	4.6	30	0.32	1.78	3.?	32
	BS-15D	.14	6.2	29	1.4	2.21	3.5	29

The values of the cracking and the ultimate loads from the above tables for steel bar (BS) having relative rib area of (α_{sb} = 0.14) compared to the corresponding values for steel bar (EZ) having relative rib area of (α_{sb} =0.06) at different bonded part (L_B) were about 150% and 130% respectively. The values of the cracking and the ultimate loads increased with the increase of the bonded part. If the critical and the ultimate load at bonded part of 5D is taken as a comparative value, then:

- For bonded part of 10D, P_{cr} and P_u about 200%.
- For bonded part of 15D, P_{cr} and P_u about 310%.

The values of the deflection at the ultimate load for specimens with steel bar (BS) Compared with the corresponding values for steel bars (EZ) for different bonded part were about 95%.

The values of the end slip at the cracking and the ultimate loads for specimens with steel bars (BS) compared with the corresponding values for stee! bars (EZ) were about 92% for different bonded part except for L_b=15D for steel bars (BS) was 82%.

The values of concrete strains for the tension zone at the cracking and the ultimate loads increase with the increase of the bonded part.

The rotations at the cracking and the ultimate loads decrease with the increase of the relative rib area and decrease with the increase of the bonded part.

The slope at the free end at the cracking and the ultimate loads decreases with the increase of the relative area or for the increase of the bonded part.
The Egyptian Codes 208(1995) [1] and 203(2001)[2] suggest

$$L_d = D.\alpha.\beta.\delta.(f_y/\gamma_s)/(4 f_{bs}) \qquad (2)$$

Where: D; nominal diameter of the bar. ^: 1,4 for top reinforcement if the thickness of the cast concrete below is more than 30 cm, and 1.0 for all other cases. f_{bu}: ultimate bond stress between steel and concrete, as given in equation

$$f_{bu} = 0..3\sqrt{fc_u/\gamma_{c.}} \quad \text{N/mm}^2 \quad \ldots\ldots\ldots(3)$$

α **:** Correction factor depends on the type of the bar end- β correction factor depends on the bar type (smooth=l or ribbed bar = 0.75).

The values of the correction factor for the type of the bar surface (β) equal lo 0.75 for deformed bar for tension loads regardless the geometry of the deformed bar or give a limitation for the minimum relative rib area to the bar to be considered as a deformed bar.

The values of the factor (β) according to eq, (2) was calculated for each specimen as shown in table (4).

Table (4): Values of (β) for cantilever-to-column connection.

BONDED PART	5D	10D	15D
Bar (BS)	0.152	0.152	0.147
Bar(EZ)	0.202	0.196	0.199

The reduction of the values of the factor (β) is due to the increase of the relative rib area (α_{sb})- This reduction causes decrease in length, •width **of** cracks and increase the stiffness of the cross-section of the cantilever.
The correction factor (β) is affected by the following factors:

a- The relative rib area (α_{sb}).
b- The bonded part (L_d),
c- The steel bar diameter (D).
d- The type of loading (direct or indirect loading).

CONCLUSIONS AND RECOMMENDATIONS

The following main conclusions and recommendations can be drawn out:

1- The final mode of failure was bond failure for all specimens; except for the specimen with steel bars type (BS) of bonded part 15D;
2- The first cracking load was early observed for cantilever reinforced with bars of small relative rib area and for small bonded part.

3- The width of cracks was significantly increased with the decrease of both the relative rib area and the bonded part.
4- The values of cracking and ultimate loads increased with the increase of both the relative rib area and the bonded part.
5- The values of the deflection and the slope at the free end increased with the decrease of both the relative rib area and the bonded part.
6- The values of concrete strain, the rotation angle and the rotational capacity decreased with the increase of both the relative rib area and the bonded part.
7- The calculation of the correction factor (β) given in equation (2) according to the Egyptian code should be consider the following factors :
   a- The relative rib area (α sb).
   b- The bonded part (L_b)
   c- The type of loading (direct or indirect loading).
8- More investigations are needed to study the effect of different parameters of the rib geometry on the characteristics of the cantilever-to-column connections.

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خواص التماسك وسعة التشكل والدوران للمفصلات الخارجية من الخرسانة المسلحة متأثرة بطول الرباط وهندسية النتوءات لاسباخ حديد التسليح

في الآونه الأخيرة زادت الحاجة لاستخدام خرسانة مسلحه عالية المقاومة ولذا أصبح من الضروري استخدام حديد تسليح صلب عالي المقاومة وللاستفادة من المقاومة القصوى لصلب التسليح فإن الأمر يتطلب قوة تماسك كافية بينة وبين الخرسانة مما يستلزم إحاطة سطح حديد التسليح بنتوءات تكفل تولد قوى التماسك الكافية.

لذلك فقد ظهرت أنواع عديدة من صلب التسليح عالي المقاومة بأشكال مختلفة ومتنوعة للنتوءات الموجودة على سطح حديد التسليح وأشهرها النتوءات ذات الشكل الشجري وأخرى ذات الشكل الهلالي وبزوايا ميل مختلفة للنتوء .

ونظرا لاختلاف شكل هندسية النتوءات لهذه الأنواع المختلفة من حديد التسليح فقد ظهرت الحاجة لمعرفة اثر ذلك الاختلاف على مقاومة التماسك بين الحديد والخرسانة. ولقد أجملت المواصفات القياسية المصرية رقم 208 (1995) رقم ( )203 لسنة (2001) – أثر نوع الحديد في طول الرباط بمعامل يعتمد على نوع الحديد سواء كان أملسا أو ذو نتوءات وأهملت شكل هندسية النتوءات واختلاف المساحة النسبية لهذه النتوءات على طول الرباط لحديد التسليح واثر ذلك على مقاومة التماسك بين الحديد والخرسانة وسعة التشكل الدورانى .

لذلك فإن الهدف الرئيسي من هذه الدراسة هو محاولة الوقوف على تأثير هندسية النتوءات للحديد المشرشر بأنواعه المختلفة على خواص التماسك وسعة التشكيل الدورانى في او صلات الخارجية للمنشآت مع الأخذ في الاعتبار بعض المتغيرات التي تؤثر على هذا السلوك مثل المساحة النسبية للنتوء وطول الرباط الفعال للحديد مع الخرسانة.

وقد تمت الدراسة باستخدام عدد (6) عينات تمثل وصلة خارجية مكونة من عمود بكابولى ولقد خلصت الدراسة بضرورة مراعاة تغير المساحة النسبية وهندسية هذه النتوءات على مقاومة التماسك وعلى طول الرباط لحديد التسليح المستخدم في الوصلات الخارجية وضرورة مراجعة المعادلة التي نصت عليها المواصفات المصرية لتضع في اعتبارها العوامل المذكورة.