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Behavior of Post Tensioned Slab-Column Connections with Punching Reinforcement Subjected to Lateral Loads

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ملخص البحث

الهدف الرئيسي من هذه الرسالة هو دراسة سلوك وصلات الأعمده الداخلية بالبلاطات المسطحة ذات سبق الإجهاد اللاحق والمعرضة لأحمال جانبية ترديه كأحمال الزلازل. لتحقيق هذا الهدف ، أجريت إختبارات معمليه لتقييم تأثير الإزاحه الجانبيه المستحثه على وصلات الأعمدة الداخلية بالبلاطات المسطحة ذات سبق الإجهاد اللاحق. تم اختبار سبعة عينات تحت تأثير حمل الجاذبية جنبا إلى جنب مع إزاحه ترديه مصاحبه بزيادة الحركه النسبية بين طرفي العمودين العلوي و السفلي حتى الإنهيار. تم مرعاة أن تكون مقاومة عزوم الانحناء لجميع العينات متساويه و إختلفت نسبة صلب سبق الإجهاد المتماسك إلى نسبه صلب التسليح فى القطاع الخرسانى

. تم تزويد أربع عينات بصلب تسليح القص الثاقب المكون من كانات مغلقه بعدد صفوف متنوعة "خمسه ، ثمانيه و عشرة صفوف" . تفاوت إجهاد الضغط فى الخرسانة بعد حدوث كل فواقد سبق الإجهاد من 0.9 ميجاباسكال إلى 2.7 ميجاباسكال في اتجاه التحميل الترددي للعينات بدون صلب تسليح القص الثاقب بينما كان للعينات المسلحه بصلب تسليح القص الثاقب مساويا 1.8 ميجاباسكال. إجهاد الضغط فى الخرسانة بعد حدوث كل فواقد سبق الإجهاد في الاتجاه العمودي للحمل الترددي لجميع العينات مساويا 1.8 ميجاباسكال. توزيع الكابلات في اتجاه التحميل الترددي إما موزع على كامل القطاع أو مركز عند منطقه العمود بينما كان توزيع الكابلات في الاتجاه العمودي مركزا بمنطقه العمود لكافة العينات.

تم الحصول على معلومات قيمة من الاختبارات المعمليه حول سلوك وصلات الأعمدة بالبلاطات سابقة الاجهاد تحت تأثير الأحمال الترديه الجانبيه. بمقارنة النتائج التي تم الحصول عليها تبين أن أداء الوصلات قد تحسن بشكل ملحوظ بعد إضافة صلب تسليح القص الثاقب فقد زادت كلا من قدرة الوصلات على نقل قيمه أكبر من العزم المنقول إلى العمود ، وقدره الوصله على الدوران ، وأقصى طاقة مبذوله ، بينما قل بشكل ملحوظ تدهور جساءة الدوران.زيادة كمية صلب تسليح القص الثاقب أثر بشكل ملحوظ على زيادة معامل الممطولية للوصلات ، معامل تعديل ردود الأفعال (تخفيض القوى) والحد الأقصى من الطاقة المبذوله. ومع ذلك لا يوجد تغيير ملحوظ في قدرة الوصله على نقل عزوم أكبر للعمود ، و تدهور جساءة الدوران

ABSTRACT: The objective of this paper is to investigate the behavior of internal bonded post tensioned flat slab-column connections with various amount of punching shear reinforcement subjected to lateral loads and present a comparison their performance with internal bonded post tensioned flat slab-column connections without punching reinforcement. To achieve this goal, experimental study was conducted to assess the effect of induced lateral displacements on bonded post-tensioned concrete flat slab interior column connections. Four specimens were experimentally tested under gravity loads combined with reversed cyclic displacement with increasing drift ratio up to failure. Specimens had been provided with punching shear reinforcements “shear links type” with varied intensities. Flexural strength for all specimens is the same; Presstressing compressive stress kept to 1.8 MPa in both cyclic loading and transverse directions for all specimens. Tendons parallel to loading direction were distributed along slab width for three specimens and banded for one specimen while kept banded in the perpendicular direction. Tests showed that adding punching shear reinforcement increase slab-column joint ductility, unbalanced moment capacity and maximum stored energy.

Keywords: Post tension; seismic; punching; ductility; slab-column connection.

I. INTRODUCTION

Architectural and constructability advantages of post-tension flat slabs systems compared to normal reinforced ones made the popular nowadays. Flat slab system in general reduce restrictions of renovation of existing building compared to beam column system. However, during earthquake slab-column connections are subjected to huge inelastic deformations which may cause loss in the connection ductility as well as unbalanced moment transfer capacity if not probably detailed. High shear stress is generated at slab-column connections from combined gravity and lateral loads if the connection is not able to sustain such stress punching failure may take place associated with immediate loss of connection stiffness and ductility. Unbalanced transferred moment are generated from inelastic story drifts, connections required to be able to sustain an inelastic story drift from 1.5% to 2% without punching failure. Punching shear resistance may be increased by adding many forms punching reinforcement such as closed stirrups, shear studs and shear bands.

This paper highlights the effect of adding punching shear reinforcement closed links type to interior post-tensioned slab-column connections and compared results with previously experimented interior post-tensioned slab-column connections without punching shear reinforcement. The work presented here in is a part of the Ph.D. works of the first author.

II. LITERATURE REVIEW

Tamer et al. [9] had examined three internal post-tensioned slab-column connections without punching shear reinforcement that had been design according to ECP 203-2017 [1] under reversed cyclic displacement with increasing drift ratio up to failure. Tested specimens had various amount of pre-compression stress ranging from 0.9 MPa to 2.7 MPa, however all specimens shared the same ultimate flexural capacity using different amount of normal reinforcement ratio. they concluded that increasing the amount of prestressing enhances the connection ability to absorb, store and dissipate energy. However, Increasing the pre-compression stress from 0.9 MPa to 2.7 MPa in the cyclic loading direction did not have a noticeable impact on the connection maximum ultimate unbalanced moment, the increasing of the pre-compression stress from 0.9 MPa to 2.7 MPa was associated with a reduction of normal reinforcement ratio from 0.7% to .2% to ensure all specimens having same flexural strength. They also concluded that ECP 203-2017 equations with a maximum cube strength of 40 MPa for estimating the maximum allowable shear stress is conservative by 20% for low to moderate pre-compression stress and by 16% for high pre-compression stress. discarding the limit cap of cube strength give a good agreement with obtained shear stress from the experimental test.

Megally and Ghali [2] had proposed a procedure of the design requirement of non-prestressed flat slab column connections subjected to seismic loading. To ensure the prevention of punching shear failure a maximum unbalanced transfer moment based on slab's flexural capacity were suggested. Ductile failure is observed for all specimens with shear head punching reinforcement. Megally and Ghali summarized the design steps for designing slab column connection under earthquake loading as shown in Figure 1.

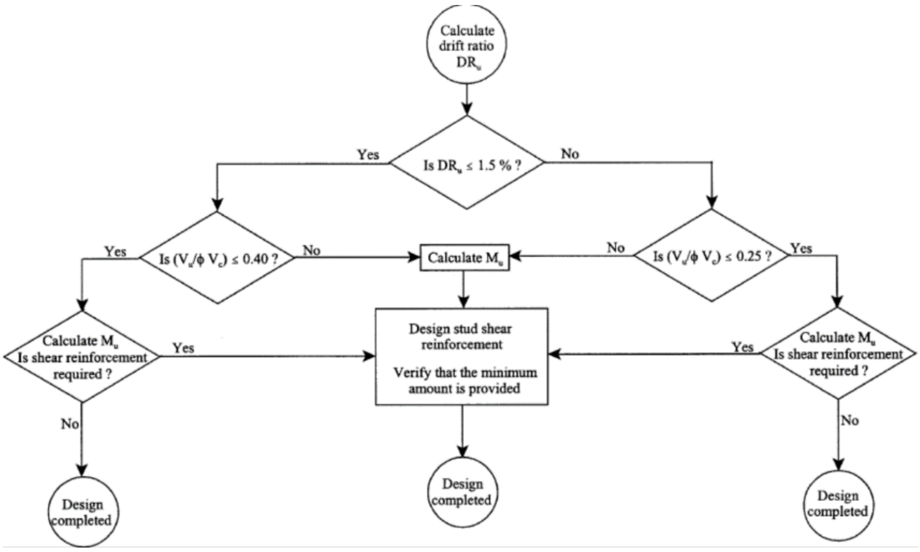


Fig. 1: Megally and Ghali proposed steps for designing non-prestressed slab column connections under seismic loads.

Austin Pan and Jack P. Moehle [4] examined previously experimented samples to identify the major parameters influencing the ductility and stiffness of slab column connections. They concluded that gravity shear ratio is an important factor that affecting the connection rotation capacity. Loading protocol had great influence on joint ductility, stiffness and unbalanced moment capacity, for a certain gravity shear ratio biaxial lateral reduces stiffness, joint ductility and unbalanced moment capacity compared to uniaxial loading one.

III. RESEARCH SIGNIFICANCE

This paper examines the performance of bonded post-tensioned interior slab-column connections with punching shear reinforcement subjected to seismic loads represented as reversed cyclic displacement with increasing drift ratio up to failure. Previous experimental programs had examined the behaviour of either unbonded post-tensioned or normal reinforced slab-column connections under lateral loads using shear studs.

IV. TEST PROGRAMS

Four full scale internal bonded post-tensioned slab-column connections with punching shear reinforcement with different intestines had been constructed and tested up to failure. Slabs in all specimens were square with a length of 2 m and 250 mm thick, two square columns with side length of 300 mm and height of 900 mm was constructed at slab center above and below slab. Figure 2 shows the specimen concrete configurations. Sample's reinforcement, test setup and loading protocol had been described in detail in Tamer et al. [9]. Table 1 shows all specimens reinforcement configuration. Where CLD means cyclic loading direction. Provided punching shear reinforcement in specimens shown in Figure 3. Figure 4 shows an actual photo of test setup, loading stages and reversed cyclic displacement protocol.

Table 1: Specimens reinforcement configuration.

Specimen number	1.8-D-0.5	1.8-B-0.5	1.8-D-1	1.8-D-1.2
Bottom Rft.	12T10 in each direction. 2 bars inside column			
Banded Rft.	8T16 in banded direction. 2 bars inside column			
Banded tendons	7 0.5” strands			
CLD Rft.	8T16	8T16	8T16	8T16
CLD tendons	7 strands	7 strands	7 strands	7 strands
Punching Rft.	5 rows	5 rows	8 rows	10 rows

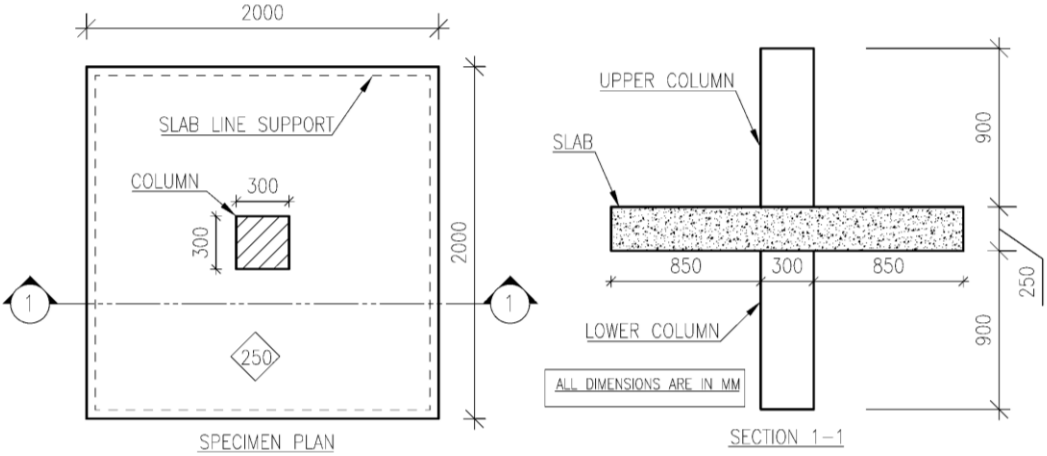


Fig. 2: Specimen concrete configuration

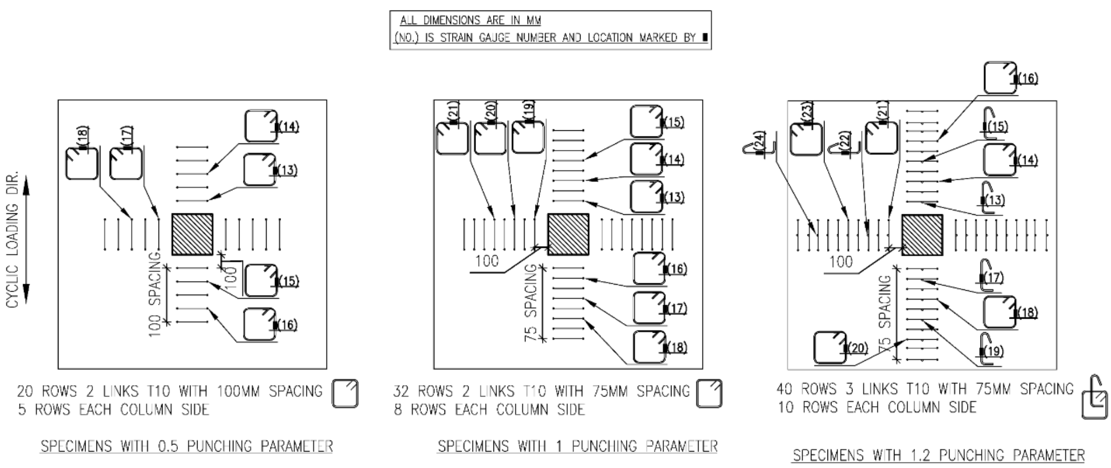
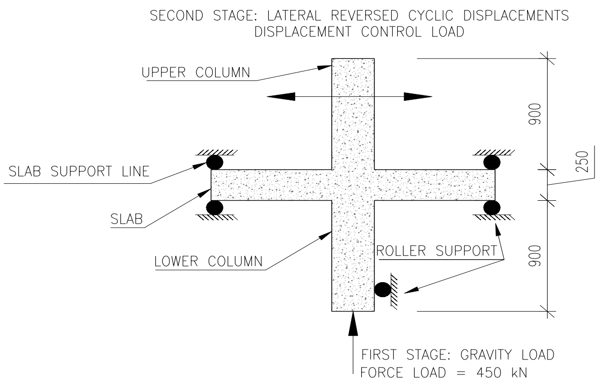


Fig. 3: Punching reinforcements configurations

V. T EXPERIMENTAL RESULTS

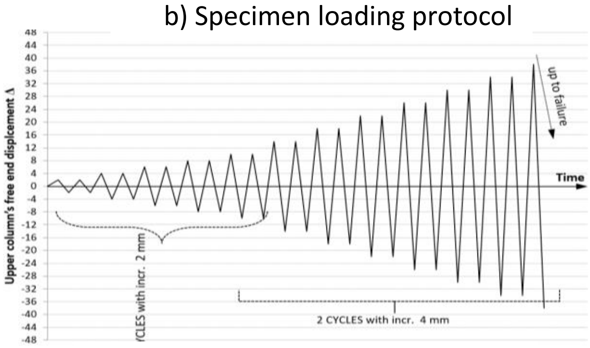
- **Crack pattern and Failure mode**

Minor top surface flexural cracks had appeared during the gravity load stage, cracking at gravity loading stage started at an approximate load level of 300 kN for all specimens. However; vertical LVDT readings versus gravity load showed that there was no drop of axial stiffness at gravity load stage as shown in Figure 5. Compared to vertical deformation obtained by Tamer et. al for specimens without punching shear reinforcement, specimens with punching shear reinforcement showed less vertical deformations. Cracks started at gravity load stage were propagated during the cyclic loading stage and new radial cracks where formed. As drift increases cracks extend more and widths became wider. Figure 6 shows crack pattern at the end of cyclic loading stage. Top flexural cracks had appeared along main top reinforcement as they were subjected to sever deformations. Punching shear failure is dominant for all specimens. Table 2 shows the maximum unbalanced transferred moment Mu for each specimen and compared to previously tested specimens without punching shear reinforcement, the unbalanced transferred moment values showed to be less than the estimated moment value that will cause flexural failure for the connection, connection yield moment had been calculated using Brown and Dilger [6] approach and found to be approximately 400 kN.m. This indicates that punching shear failure taken place prior to flexural failure. Unlike specimens without punching shear reinforcement punching failure was not associated with sudden drop of rotational stiffness and higher ductility was encountered. Ultimate unbalanced moment capacity was achieved at lateral drift ratio of approximately 2%.



b) Specimen loading protocol

a) Photo of actual test setup



c) Reversed cyclic displacement

Fig. 4 a) Photo of actual test setup, b) Specimen loading protocol and c) Reversed cyclic displacement**.**

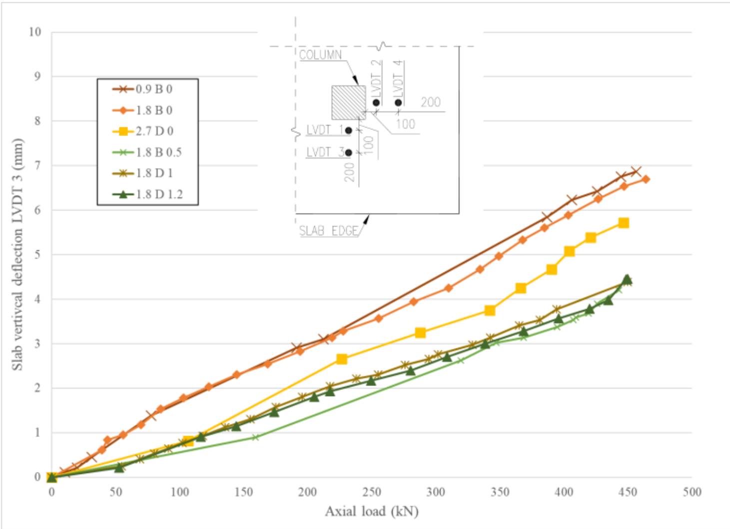


Fig. 5: Vertical deformation of slab during gravity load stage.

Fig. 6: Observed damages and crack pattern.

- ## Unbalanced moment and lateral drift ratio

Unbalanced transfer moment had been obtained from the reaction exerted from the double acting jack multiplied by the distance from the jack to the horizontal tie, this distance is kept the same for all specimen and it was 1.2 m. Figure 7 shows the unbalanced moment versus lateral drift ratio hysteresis loops for all specimens, the envelop of hysteresis loops for the all specimens compared with specimens without punching shear reinforcement are shown in Figure 8.

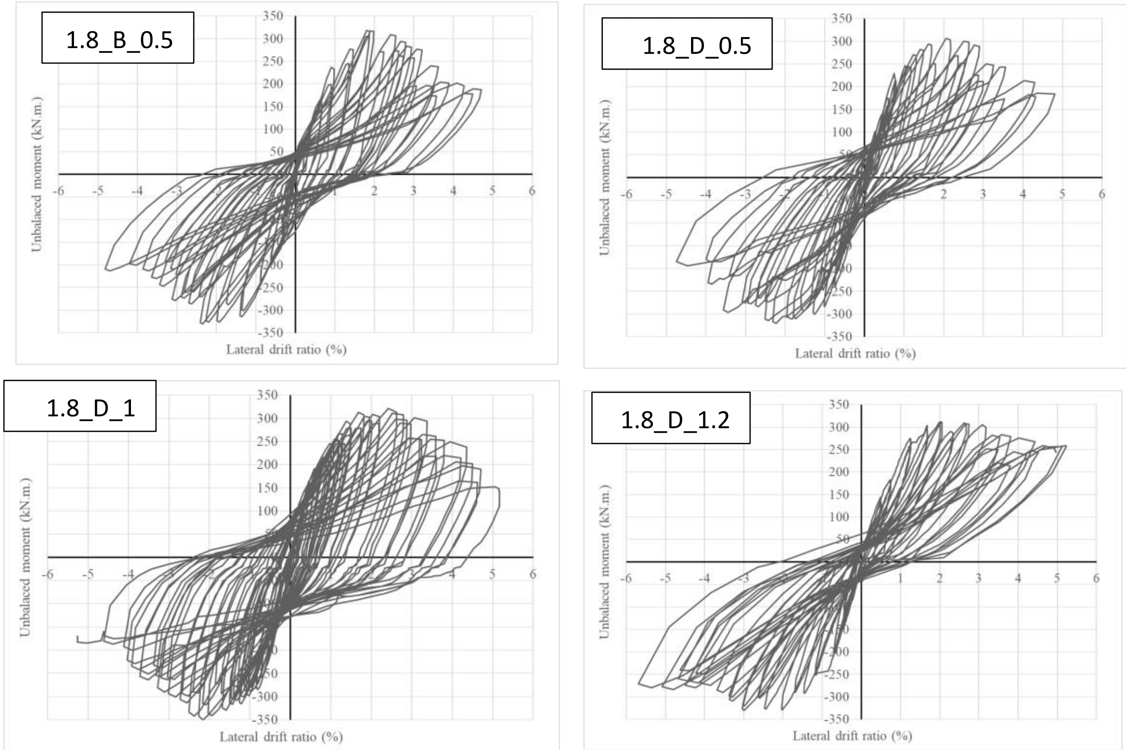


Fig. 7: Unbalanced moment versus lateral drift ratio hysteresis loops.

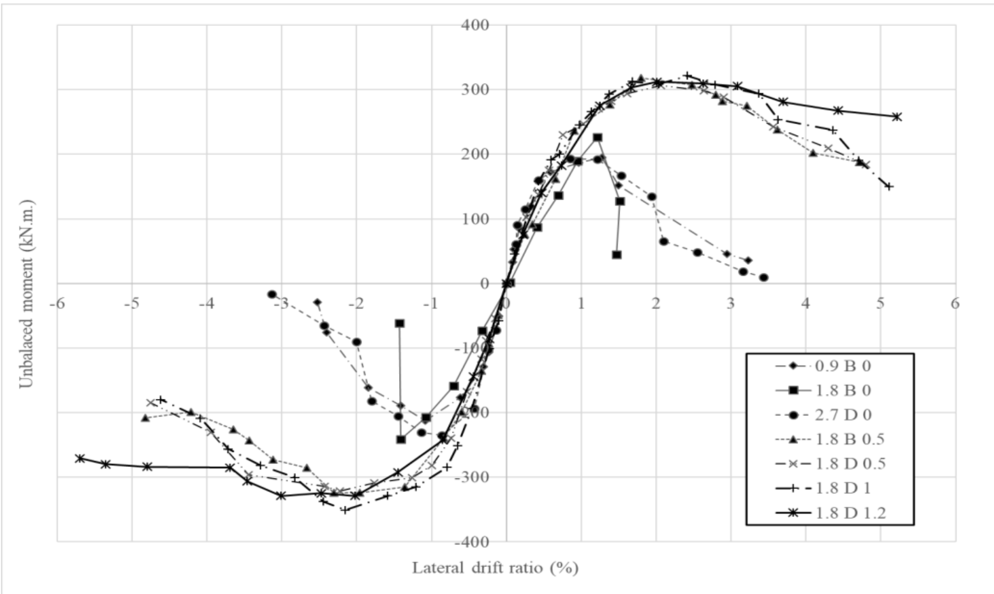


Fig. 8: Envelopes of hysteresis loops of unbalanced transferred moment versus lateral drift ratio.

Table 2 summarize the outcomes from envelop of hysteresis loops base on the recommendation of Moehle and Pan [4], where M_u is the ultimate unbalanced transferred moment, D_y is the yield drift, D_u is the ultimate drift, $D_{u80\%}$ is the drift corresponding to 80% of the maximum ultimate moment, $E_{g,max}$ is the maximum exercised energy, μ is the displacement ductility factor and R_d is response reduction factor. Displacement ductility factor μ can be calculated as; $\mu = D_{u80}\%/D_y$ and response reduction factor can be calculated as; $R= (2\ \mu\text{-}1)^{0.5}$ [5].

Table 2: Summary envelop of hysteresis loops analysis:

Specimen No.	M_u	D_y	D_u	$D_{u80\%}$	$E_{g,max}$	μ	R_d
	(kN.m.)	(%)	(%)	(%)	(kN.m.)		
0.9 B 0	213	0.59	1.08	1.72	2.8	2.92	2.2
1.8 B 0	241	1.07	1.41	1.47	2.12	1.37	1.32
2.7 D 0	235	0.53	0.87	1.7	3.15	3.21	2.33
1.8 B 0.5	324	1.05	2.3	3.26	8.41	3.1	2.28
1.8 D 0.5	321	0.99	2.22	3.59	9.38	3.63	2.5
1.8 D 1	351	0.89	2.15	3.36	9.49	3.78	2.56
1.8 D 1.2	329	1.13	2.02	5.99	16.32	5.3	3.1

- ## Punching Shear Capacity

Punching shear stresses resulted from ultimate unbalance moment associated with gravity load had been calculated using ECP 203-2017 approach and equations. Ultimate shear stresses had been compared with ECP 203-2017 limits at two sections. First section is located at d/2 from column, second section is located at distance d/2 from the outermost line of punching shear reinforcement. Allowable shear stresses were calculated using the partial safety factor for concrete strength γ_c. Figure 9 shows shear stress at line A-B at control perimeter versus lateral drift ratios.

Experimental shear stress calculated at line A-B are higher than ECP prediction by 52% for specimens 1.8_B_0.5 and 1.8_D_0.5, the conservatism was dropped to 36% for Specimen 1.8_D_1. However, for Specimen 1.8_D_1.2 and experimental shear stress agreed with ECP prediction. The conservative estimation of ECP equation raised from limiting steel stress used in punching reinforcement to 350MPa. Comparing results using punching shear reinforcement stress of 420 MPa the experimental to code shear stress ration will drop to 37% for specimens 1.8_B_0.5 and 1.8_D_0.5 and 21% for Specimen 1.8_D_1 however it would be un-conservative for Specimen 1.8_D_1.2 by 12%.

Experimental shear stress had been calculated for section at distance d/2 from the outermost line of punching shear reinforcement and compared with ECP limits. Results showed that ECP predicted exactly the shear stress limit for specimens 1.8_B_0.5 and 1.8_D_0.5 where punching failure occur outside the punching reinforcement zone and gave lower values for specimen 1.8_D_1 and 1.8_D_1.2 since failure is compound within punching reinforcement zone.

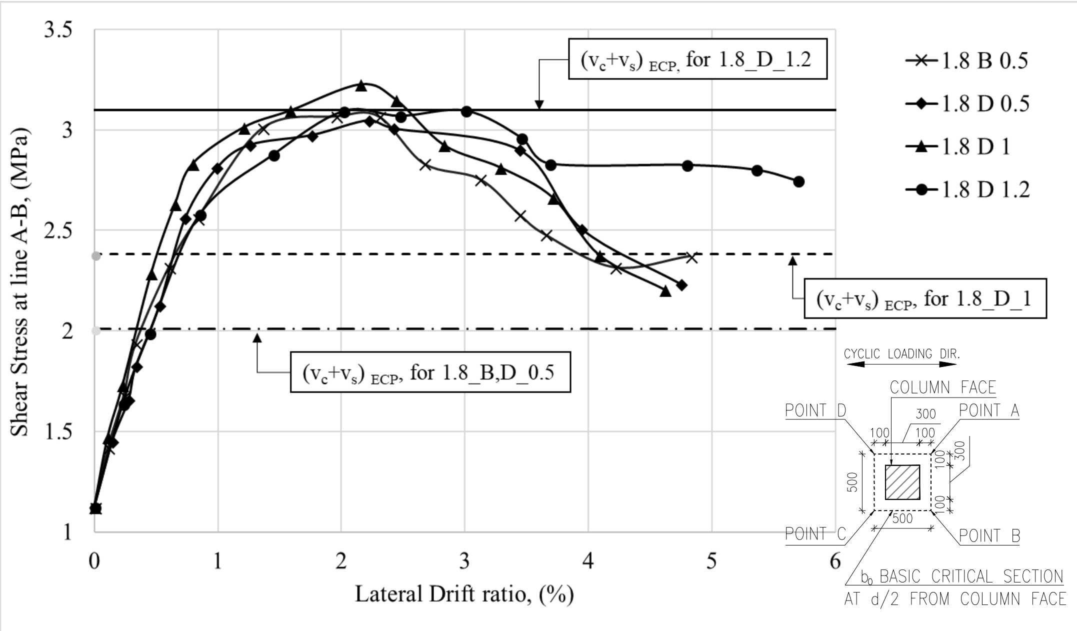


Fig. 9: Shear stresses at line A-B versus lateral drift ratio for specimens with punching shear reinforcement compared to ECP limits.

- **Lateral drifts**

ECP 201-2012 [7] limits service story drift to a value between 0.5% to 1% depending on the used type of non-structural elements and its detailing, these values are equal to inelastic drift deformation equal to 1.25% to 2.5% respectively for building in importance group one and two and equal to1% to 2% for building in importance group three and four. Inelastic drift ratio should be compared with the drift corresponding to 80% of the maximum ultimate moment, an average limit drift of 1.5% deemed to be acceptable. As shown in Table 2 specimens with punching shear reinforcement had reached a minimum 3.2% lateral drift which deemed to be acceptable in most cases. Increasing the intensity of punching reinforcement from five rows spaced at 100 mm to eight rows spaced at 75 mm have a minor enhancement impact on inelastic drift. Also changing the distribution of prestressing tendons from distributed to banded to did not enhanced the inelastic drift ratio.

- **Joint ductility and Rotational Stiffness**

Displacement ductility factor is $\mu = D_y/D_{u80\%}$. Newmark and hall [8] recommended that displacement ductility factor for any component to have a minimum value of 3. Unlike specimens without punching shear reinforcement, all specimens with punching shear reinforcement exceeded the specified minimum displacement ductility factor. Increasing the punching shear reinforcement extent and intensity have a remarkable change on the displacement ductility factor.

Cyclic loading will cause slab and column to be cracked and the more drift ratio is obtained the more cracking degree is taken place. However, the deterioration of rotational stiffness of slab column connection gives a clear idea on the behavior of connection under cyclic lateral loads and connection degradation with increased inelastic lateral drift ratio. Joint rotational stiffness K_{Rot} is equal to the unbalanced moment Mu divided by the resultant rotation or lateral drift ratio. For each specimen rotational stiffness K_{Rot} for each point if the envelop of hysteresis loops is calculated and plotted in Figure 10. The trend of stiffness degradation for lateral drifts less than 1% is almost the same percentage except for specimen 1.8-B-0 due to rapid loading program obtained stiffness were well below others. Increasing lateral drift ratio more than 1% showed a sudden deterioration of rotational stiffness for specimens without punching shear reinforcement while all specimens with punching shear reinforcement showed a gradual loss of rotational stiffness.

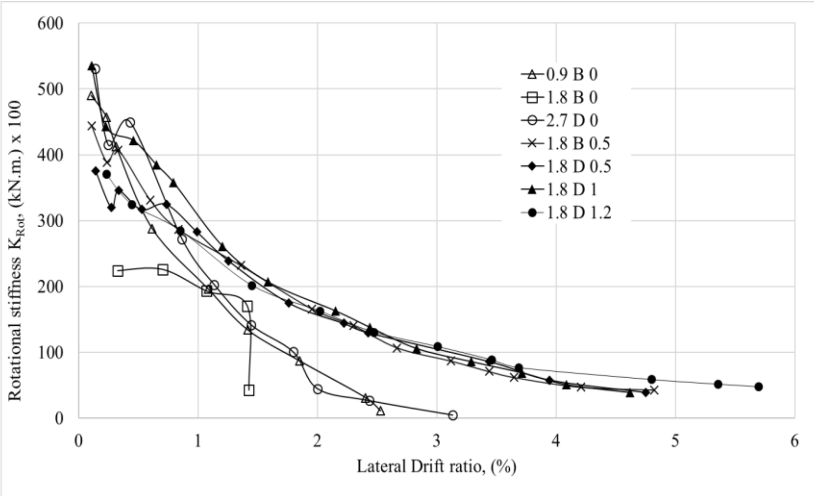


Fig. 10: Rotational stiffness versus lateral drift ratio

VI. CONCLUSIONS

The paper aimed to study the behavior of internal bonded post-tensioned slab-column connections with punching reinforcement under cyclic loading, based on four full scale specimens with different punching shear reinforcement intensities, the following were concluded:

- Performance of bonded post-tensioned slab column connections is significantly improved by adding punching shear reinforcement “closed link type” in terms of maximum unbalanced transfer moment capacity, rotational capacity, rotational stiffness degradation, maximum input energy and deformation capacity.
- For bonded post tension slab-column connections with punching shear reinforcement, increasing the amount and extent of punching shear reinforcement noticeably increasing joint ‘s displacement ductility factor, response reduction factor and maximum input energy. However; it there are no remarkable change in joint’s maximum unbalanced transfer moment capacity, rotational stiffness degradation and ultimate drift ratio.
- Recommendation of Megally and Ghali [2] for the design of non-prestressed flat slab column connections subjected to seismic loading may apply to bonded prestressed flat slab column connections.
- specimens with punching shear reinforcement showed a gradual loss of rotational stiffness while all specimens without punching shear reinforcement showed a sudden deterioration of rotational stiffness.
- ECP 203-2017 equations for slabs with punching shear reinforcement below the maximum allowed shear reinforcement showed a conservative results prediction if the failure within the punching shear reinforcement zone, however, ECP prediction

for failure outside punching shear reinforcement zone showed an acceptable agreement with the experimental results.

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