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BEHAVIOUR OF WIDE REINFORCED CONCRETE BEAMS WITH DIFFERENT SHEAR STEEL PLATES SPACING

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Abstract: This paper presents an experimental investigation on the behavior of wide reinforced concrete beams with different shear steel plate spacing. Four reinforced concrete wide beams with the dimensions of 215x560x1800mm are investigated. The variables studied in this work is using the 10mm stirrups with 125mm spacing and 3mm thickness steel plate with spacing 125, 166 and 250mm instead of reinforcing stirrups. Shear steel plates is a good alternative for stirrups and gives identical results in yield and ultimate loads and increase the ductility by 55% and reduced the strain in exterior and interior legs by 17% and 46% respectively compared with stirrups also reduced the total weight of wide beams by 2.7%. By increasing the spacing of shear steel plate by 33% and 100%, the results showed that the yield load reduced to 5% and 12% respectively, but the deflection was increased with 2% and 16% respectively(at yield). The strain in interior legs is more than the strain in exterior leg (having spacing 125, 166 and 250mm) by 31%, 70% and 1% respectively. ACI 318-14[(1)] and EC 2[(2)] codes are given a predicted deflection less than the experimental deflection by 19% and 21% on average respectively.

Keywords: *Reinforced Concrete; Wide Beam; Stirrups, Shear Steel Plate, Spacing.*

تصرف العتبات الخرسانية المسلحة العريضة بأختلاف المسافات بين صفائح حديد القص

الخلاصة: يتضمن هذا البحث دراسة عملية لتصرف العتبات الخرسانية العريضة بأختلاف المسافات بين صفائح حديد القص. اشتملت الدراسة على صب اربع عتبات خرسانية مسلحة عريضة بابعاد (560x215x1800)ملم. لقد تم دراسة متغيرات عديدة، منها استخدام اطواق بقطر ١٠ملم بمسافات ١٢٥ملم واستخدام صفائح من الحديد بسمك ٣ملم وبمسافات ١٢٥، ١٦٦ و ٢٥٠ملم بدلا من حديد القص. ان استخدام الصفائح كبديل عن الاطواق اعطى تطابق في نتائج حمل الخضوع والفشل وزاد من المطيلية بنسبة ٥٥% وقلّت فيه الانفعالات في الساق الخارجية والداخلية بنسبة ١٧% و٤٦% على التوالي مقارنة مع الاطواق اضافة الى تقليله الوزن الكلي للعتبات العريضة بنسبة ٢,٧%. ان زيادة المسافات بين صفائح حديد القص بنسبة ٣٣% و ١٠٠%، قلل من حمل الخضوع بنسبة ٥% و١٢% على التوالي، ولكن زاد من الهطول بنسبة ٢% و١٦% على التوالي (في مرحلة الخضوع). ان الانفعال في الساق الداخلية اكبر من الانفعال في الساق الخارجية (للمسافات ١٢٥ ، ١٦٦ و٢٥٠ملم) بنسبة ٣١%، ٧٠% و ١% على التوالي. الكود الامريكي ACI 318-14[(1)] والاوربي EC 2[(2)] اعطى نتائج للهطول اقل من النتائج العملية بنسبة ١٩% و ٢١% على التوالي.

1. Introduction

The use of wide concrete beams in structural framing systems has improved in latest years. This is alteration responds to the necessity for inexpensive keys which reduce

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structural high and building complexities. For example, engineers of new high-rise buildings are frequently tasked with conveying column loads from the tower portion above required column-free spaces in the pedestal or parking areas below. Wide beams may provide suitable cross-sectional areas to do the required ability in a shallower depth than a system of slenderer beams at a parallel spacing in the plan, as well as reinforced concrete wide beam-column connections, it has high efficiency to resist earthquake loads.

Adam S. Lubell, et. al (2006)[٣], carried out an experimental study to investigate the shear behaviour of the wide beams and thick slabs as well as the influence of member width. In their study they tested five specimens of normal strength concrete with a nominal thickness of 470 mm and varied in width from 250 to 3005 mm. Their study demonstrated that the failure shear stresses of narrow beams, wide beams, and slabs are all very similar.

Adam S. Lubell, et. al (2009)[4], investigated the influence of the shear reinforcement spacing on the one-way shear capacity of wide reinforced concrete members. A series of 13 normal strength concrete specimens were designed and tested. Shear reinforcement spacing was a primary test variable. The specimens contained shear reinforcement ratios close to (ACI 318-11) minimum requirements. The study concluded that the effectiveness of the shear reinforcement decreases as the spacing of web reinforcement legs across the width of a member increases, the use of few shear reinforcement legs, even when widely spaced up to a distance of approximately 2d, has been shown to decrease the brittleness of the failure mode compared with a geometrically similar member without web reinforcement. To ensure that the shear capacity of all members with shear reinforcement are adequate when designed according to ACI 318-11, the study recommended that the transverse spacing of web reinforcement should be limited to the lesser of both the effective member depth and 600 mm.

Mohamed M. Hanafy, (2012) [6], investigated the contribution of web shear reinforcement to shear strength of shallow wide beams and the test results clearly demonstrate the significance of the web reinforcement in improving the shear capacity and the ductility of the shallow wide beams which is consistent with the recognized international codes and standards provisions.

In this paper, an experimental and theoretical analysis to predict the deflection, strain, cracks patterns based on the four full scales wide reinforced concrete beam tested.

2. Research significance

The study focuses on behavior of wide reinforced concrete beams using shear steel plate with different spacing. This new technic treats the crowded of stirrups in wide concrete beam because the shear component provided by concrete is very small compared with high depth concrete beams. Also this study is an attempt to study the effect of use the steel plate with different spacing on the deflection, strain and crack pattern of wide beams.

3. Details of experimental test

3.1 Outline of program

The experimental program consisted of testing four beams with nominal compressive strength of f_c' =33MPa (Self Compacting Concrete SCC) in a four-point loading arrangement. All beams were constructed in the laboratory of the Engineering College of Diyala University. All beams were 560 mm wide, 215 mm deep, 1800mm long and were tested at a shear span of 600 mm (the width of supports is 600mm). This gives a shear span-depth ratio (a/d) equal to 3.56. The longitudinal steel reinforcement ratio was ρ=2.1%, with 16mm diameter and using 10mm diameter in compression reinforcement with 415MPa and 397MPa yield strength respectively. All the specimens were reinforced with identical longitudinal steel bars. Four wide beams are tested, one as reference with shear steel reinforcing (stirrups) (WBS), and three with shear steel plate have equivalent cross sectional area for stirrups at mid legs height and having 135mm circular opening and 3mm thickness. The spacing between stirrups is 125 mm and it was (125, 166 and 250mm) for shear steel plate (WBP3-1, WBP3-2 and WBP3-3) and the yield strength of shear steel plate is 210MPa.

Typical concrete dimensions and reinforcement details of the tested specimens are illustrated in Fig. 1 and cross sections in Fig 2 and 3. The placement of longitudinal reinforcement, shear steel plate and mold specimen shown in Fig. 4.

3.2 Tested method and measurement

All beams were tested under simply supported condition over a span 1.8m with their tension faces uppermost as shown in Fig.1. For all beams, the first crack load, deflection under loading point, steel plate strains and yield, ultimate load were measured.

4. Test results

The strength characteristics of all specimens(f'c, yield, ultimate load and deflection at yield and ultimate loads also the values of ductility) are tabulated in Table (1).

Accenting was taken in marking the load at which the first crack formed. The experimental values of the cracking loads were obtained from load-deflection diagrams.

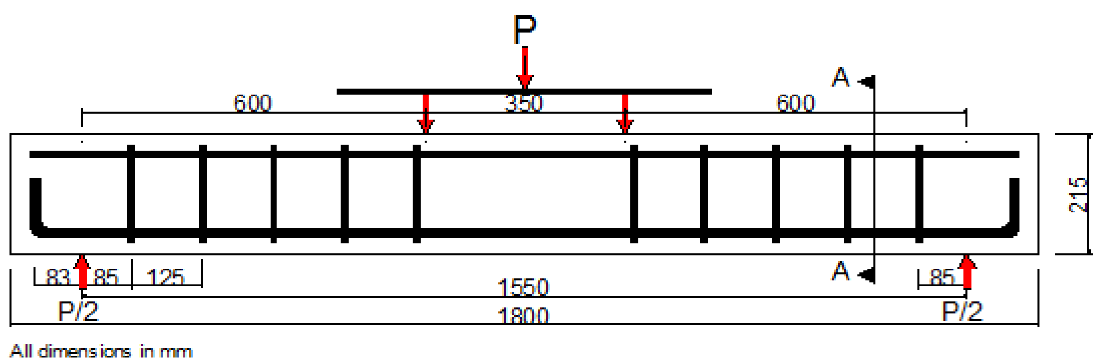


Fig. 1 Loading details

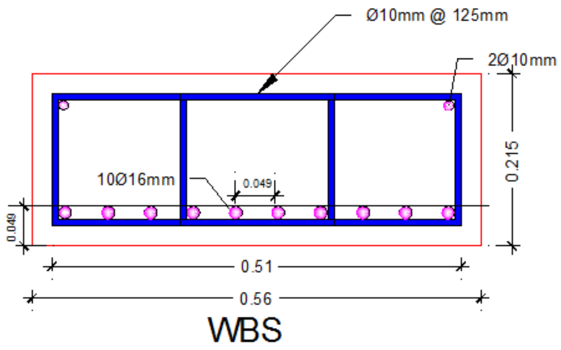


Fig. 2 Section A-A for stirrups(WBS beam)

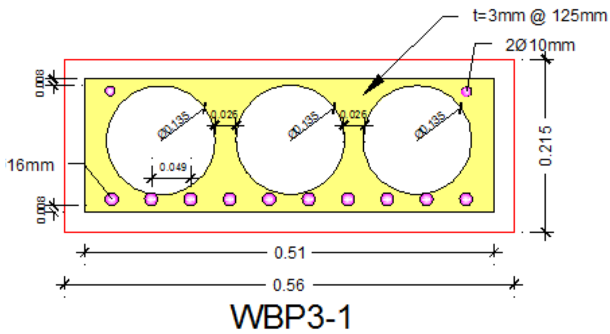


Fig. 3 Section A-A for plates(WBP3-1 beam)

Fig. 4 Peparation the mold specimen and placing the reiforcement

4. 1 Load deflection relationships

Table (1) shows the values of deflection at yield and ultimate load that were obtained from load-deflection diagrams. It can be seen from Table (1) that the deflection at yield was increased in specimens that used the shear steel plate with 24%, 21% and 5% for specimens WBP3-1, WBP3-2 and WBP3-3 respectively. This increasing of deflection was clear in ultimate load for the specimens WBP3-1, WBP3-2 and WBP3-3 when it compared with the WBS specimen by 92%, 22% and 20% respectively.

Also the ductility index was increased for the specimen WBP3-1 and WBP3-3 by 55% and 15% compared with WBS.

By increasing the spacing of shears steel plate by 33% and 100%, it can be seen from Table (1) that the deflection at yield was increased with 2.2% and 16% respectively this is obvious, due to increasing of shear steel plates spacing. This increasing of deflection was clear in ultimate load for the specimens WBP3-2 and WBP3-3 when comparing with the WBP3-1 specimen by 37% as a result of increasing of spacing of shear steel plate.

Table (1) Comparison between the strength characteristics of specimens

Beam Specimens	f'_c (MPa)	P_y	P_u	Δ_y	% diff. of Δ_y	Δ_u	% diff. of Δ_u	Ductility $\frac{\Delta_u}{\Delta_y}$	Weight (ton)	% diff. of Weight
WBS	36.6	400	440	10.83	---	18.93	---	1.75	0.522	---
WBP3-1	33.5	420	431	13.45	+24%	36.45	+92%	2.71	0.510	-2.3%
WBP3-2	34.0	400	441	13.15	+21%	23.10	+22%	1.76	0.508	-2.7%
WBP3-3	32.8	370	376	11.33	+5%	22.82	+20%	2.01	0.506	-3.1%

Fig (5) shows the load- mid span deflection curves for the specimens. It can be seen that the deflection at yield were close between all specimens, but the behaviour is different at ultimate load corresponding to decrease of ultimate load. Also it can be seen that the ductility of WBP3-1 is more than 55% compared with WBS specimen.

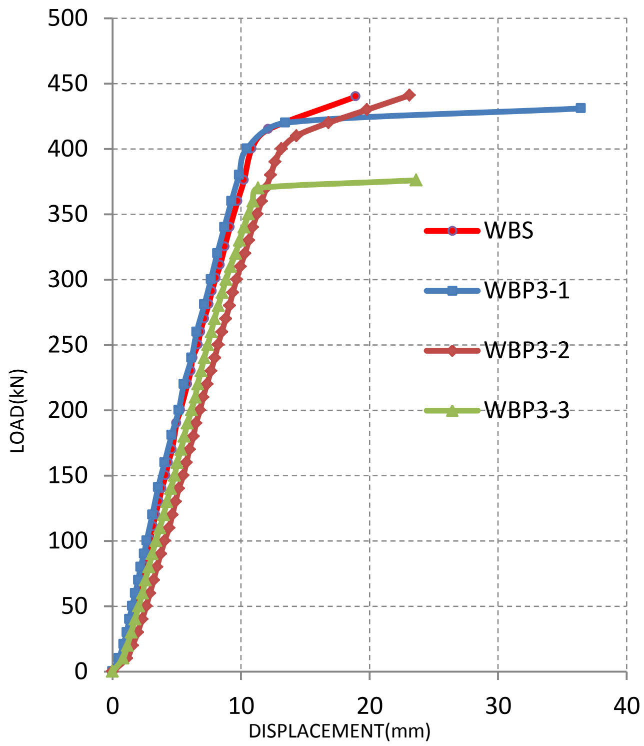


Fig. (5) Load-mid span deflection curves of tested specimens

4.2 Comparison the deflection predicted by ACI 318-14[1] and EC 2[2] codes

Table (2) shows the values of deflection at service load (assume 60% from the ultimate load) obtained from load-deflection diagrams, also the analytical results of all specimens computing by ACI 318-14[1] and EC 2[2] codes at service load were presented in Table (2), it can be seen that, the predicted deflection of wide beams calculated by

ACI 318-14[1] and EC 2[2] codes were less than the experimental deflection by 19% and 21% on average respectively.

It can be explain this increasing in experimental deflection because the dial gauge was recorded the deflection in center of wide beams in longitude and transferred directions and not consider the deflections at edges for centre of beam.

This case attributed to Saint-Venant’s principle. Saint- Venant's states that in a body under the action of a system of forces which are applied in a limited region of its boundary, the stresses and strains induced by those forces in another region of the body, located at a large distance from the region where the forces are applied, do not depend on the particular way the forces are applied, but only on their resultant. This "large distance" may be considered, in most cases, as the largest dimension of the region where the forces are applied [6].

Table (2) Experimental deflection comparing with deflection computing by of ACI 318-14[1] and EC 2[2] cods at service load

Beam Specimens	Deflection at Service Load, Δ_s (mm)				
	Measured (mm)	Predicted			
		ACI 318M-14		EC 2	
			%Difference		%Difference
WBS	3.5	3.01	-13.93	3.00	-14.28
WBP3-1	3.1	2.96	-4.284	2.83	-8.71
WBP3-2	4.4	3.00	-31.70	2.92	-33.63
WBP3-3	3.6	2.61	-27.29	2.60	-27.78

4.3 Strain characteristics in longitudinal reinforcement and compression face of concrete of specimens

Table (3) shows the values of strain in middle of longitudinal reinforcement bar and on parallel place of concrete face (in compression) at yield and ultimate load that were obtained from strain gauge connected to data logger.

4.3.1 Strain in longitudinal reinforcement

It can be seen from Table (3) that the strain in longitudinal reinforcement at yield load is regular gradation increasing in specimens WBP3-1 and WBP3-2 that used the shear steel plate with 13% and 9% respectively; this is obvious, due to the regular gradation increasing of spacing of steel plate. At ultimate load the strain in longitudinal reinforcement for the specimens WBP3-1, is more than that for the specimens WBS by 27%, this may be as a result of constraint by the large numbers of closed hole in shear steel plate.

By increasing the spacing for shear steel plate by 33% and 100%, the strain in longitudinal at yield load is regular gradation decreasing in specimens WBP3-2 and WBP3-3 compared with WBP3-1 by 10% and 26% respectively as a result of reduction of yield load. Also at ultimate load the strain in longitudinal reinforcing decreasing in specimens WBP3-2 and WBP3-3 compared with WBP3-1 by 74% and 28% respectively. This may be as a result of reduction in ultimate load.

Indeed the experimental yield strength of longitudinal steel reinforcement was 415MPa and the modulus of elasticity of steel was 200000MPa, so the experimental yield strain is 2.075×10^{-3} and it can be seen from Table (3) that the experimental yield strain of longitudinal reinforcement of all specimens were above the theoretical yield strain.

Table (3) Strain characteristics in longitudinal reinforcement and concrete specimens

Beam Specimens	*Longitudinal Reinforcement*				*Concrete (Compression)*			
	ε_y x10^{-3}	% diff. of ε_y	ε_u x10^{-3}	% diff. of ε_u	ε_y x10^{-3}	% diff. of ε_y	ε_u x10^{-3}	% diff. of ε_u
WBS	2.30	---	2.32	---	-1.27	---	-2.30	---
WBP3-1	2.62	13.8	2.95	27.4	-1.74	37.27	-2.63	14.54
WBP3-2	2.50	8.70	0.778	-66.5	-1.69	33.10	-4.41	91.74
WBP3-3	2.07	-10.0	2.120	-8.62	-1.71	34.64	-1.79	-22.17

4.3.2 Strain in compression face of concrete

Based on Table (3), the strain in compression face of concrete (at middle top face of specimens) at yield and ultimate loads is increased for the specimen WBP3-1, WBP3-2 and WBP3-3 by 37%, 33% and 35% (at yield) and 15%, 92% and 22%(at ultimate) respectively.

By increasing the spacing for shear steel plate by 33% and 100%, the strain in compression face of concrete was decreased for the specimen WBP3-2 and WBP3-3 by 3% and 1.7% (at yield load). For ultimate load stage, the strain in compression face of concrete was increased for the specimen WBP3-2 compared with WBP3-1by 67%, but it decreased for the specimen WBP3-3 by 32% as a result of reduction of ultimate load. The strain profile of four specimens is shown in Figure 6, 7, 8 and 9.

4.4 Strain characteristics in exterior and interior legs of shear steel plate

Table (4) shows the values of strain in exterior and interior legs of shear steel plate at yield and ultimate load that were obtained from strain gauge connected to data logger.

It can be seen from Table (4) that the strain exterior leg of shear steel plate is increased by 55% and 49% for WBP3-2 and WBP3-3 respectively in comparison with WBS at yield load, but it was decrease by 17% for WBP3-1. At ultimate load it is decreased by 62%, 28% and 46% for the specimensWBP3-1, WBP3-2 and WBP3-3 respectively, compared with WBS.

By increasing the spacing for shear steel plate by 33% and 100%, it can be seen from Table (4) that the strain in the exterior leg at yield load was increased in specimens WBP3-2 and WBP3-3 compared with WBP3-1 by 87% and 80% respectively. And at ultimate load the strain exterior leg was increased in specimens WBP3-2 and WBP3-3 compared with WBP3-1 by 92% and 46% respectively. This increasing in strain at yield and ultimate load stages may be due to the reducing action of shear strength providing by shear steel plates as a result of increasing of distance between it.

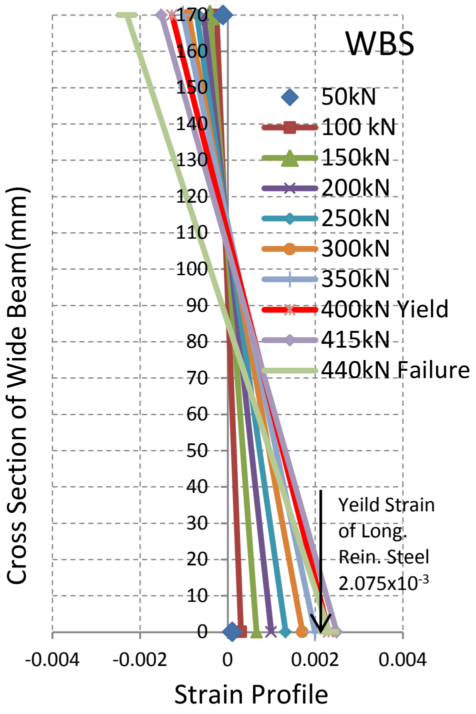


Fig. (6) Mid span strain profile of WBS specimen

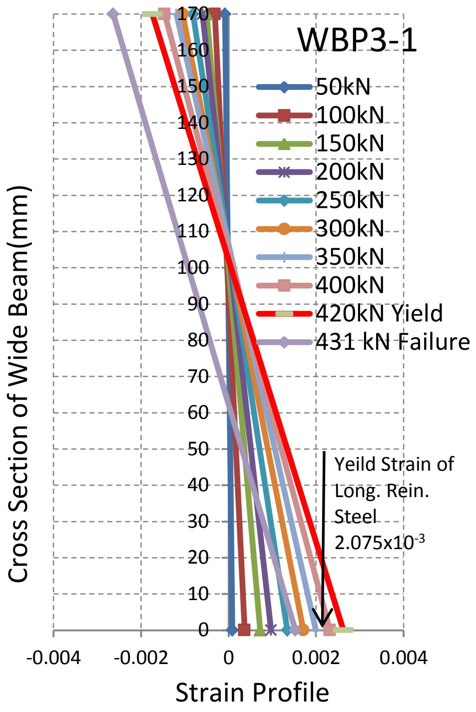


Fig. (7) Mid span strain profile of WBP3-1 specimen

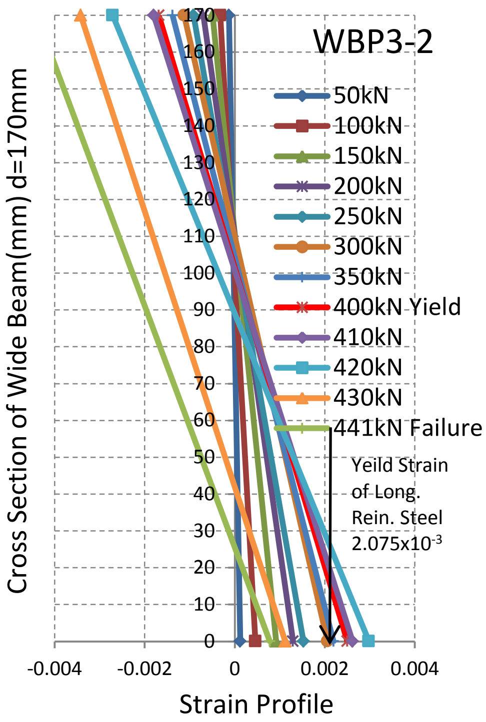


Fig. (8) Mid span strain profile of WBP3-2 specimen

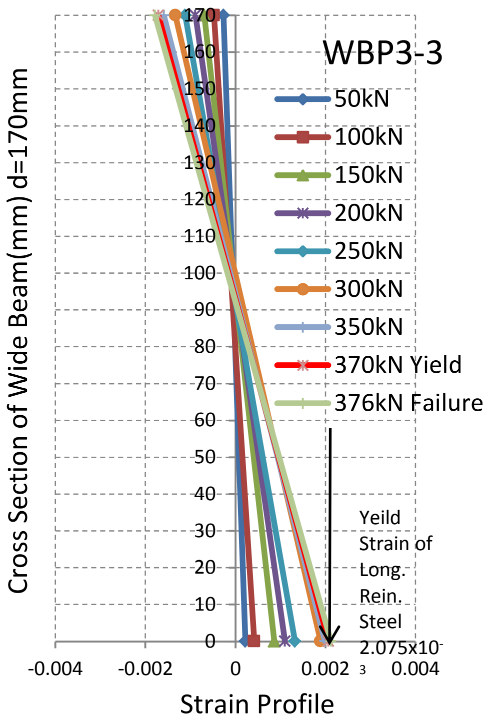


Fig. (9) Mid span strain profile of WBP3-3 specimen

4.4.2 Strain in interior leg of shear steel plates

It can be seen from Table (4) that the strain in the interior leg of shear steel plate is increased by 88% and 2% for WBP3-2 and WBP3-3 respectively(at yield load), but it was decreased by 23% for WBP3-1. At ultimate load it was increased by 104% and 236% for the specimen WBP3-2 and WBP3-3 respectively but it was decreased by 46% for WBP3-1.

By increasing the spacing for shear steel plates by 33% and 100%, it can be seen from Table (4) that the strain in the interior leg at yield and ultimate load stages the strain in interior leg was increased in specimens WBP3-2 compared with WBP3-1 by 60% and 280% respectively. This increasing in strain at yield and ultimate load stages may be due to the reducing action of shear strength providing by shear steel plate as a result of increasing of distance between it. But for WBP3-3, the strain interior leg was decreased by 13% and 37% at yield and ultimate load stage respectively. This decreasing in strain may be due to the earlier sudden shear failure of the specimen WBP3-3.

There are two main issues are observed:

1. It is clear that the strain in interior leg is more than strain in exterior leg at yield load by 39%, 31%, 70%, for WBS, WBP3-1, WBP3-2 respectively, but it was decreased by 4% for WBP3-3, and at ultimate load the strain in interior leg is more than strain in exterior leg about 23%, 76%, 248%, and 663% for WBS, WBP3-1, WBP3-2 and WBP3-3 respectively.
2. By increasing the spacing by shear steel plate the strain in exterior and interior legs was increased except the strain in interior legs of WBP3-3 because the specimen was fail due to reduction of shear force strength.

Table (4) Strain characteristics in shear steel plates (Exterior and Interior legs) of specimens

Beam Specimens	*Shear Reinforcement or Steel Plate(exterior leg)*				*Shear Reinforcement or Steel Plate(interior leg)*			
	ε_y $x10^{-3}$	% diff. of ε_y	ε_u $x10^{-3}$	% diff. of ε_u	ε_y $x10^{-3}$	% diff. of ε_y	ε_u $x10^{-3}$	% diff. of ε_u
WBS	0.614	---	1.59	---	0.855	---	1.96	---
WBP3-1	0.507	-17%	0.598	-62%	0.662	-23%	1.05	-46%
WBP3-2	0.949	55%	1.150	-28%	1.610	88%	4.001	104%
WBP3-3	0.912	49%	0.864	-46%	0.870	2%	6.591	236%

4.5 Comparison between the yield and ultimate strain (nominal and experimental) of steel plates

Table (5) explains a comparison of the yield and ultimate nominal strain and experimental strain in shear reinforcement (stirrups) and shear steel plate for interior leg. It can be seen that:

1. In all specimens the experimental strain at yield doesn't reached to the theoretical yield strain of stirrups or plate except WBP3-2 specimen.
2. The experimental strain at yield load is less than theoretical strain by 43% for the specimen WBS while the experimental strain at yield load is less than theoretical strain by 37% and 17% for the specimen WBP3-1 and WBP3-3 respectively, but it was upper than the theoretical strain by 53% for WBP3-2.

Table (5) Strain characteristics in shear steel interior plate legs

Spec.	f_y (MPa)	f_u (MPa)	ε_y x10^{-3}	ε_y x10^{-3} Exp.	$\frac{\varepsilon_y \text{ exp.}}{\varepsilon_y}$
WBS	397	685	1.985	0.855	0.43
WBP3-1	210	300	1.050	0.660	0.63
WBP3-2	210	300	1.050	1.611	1.53
WBP3-3	210	300	1.050	0.870	0.83

4.6 Crack Pattern

The tested beams at different stages of loading are shown in details in Plate (1). The bearing numbers inside the circles represent the sequence of formation of the cracks, while the numbers shown under the beams between those representing the sequence are the cracks spacing. The sign (*) represents to the first crack width appeared.

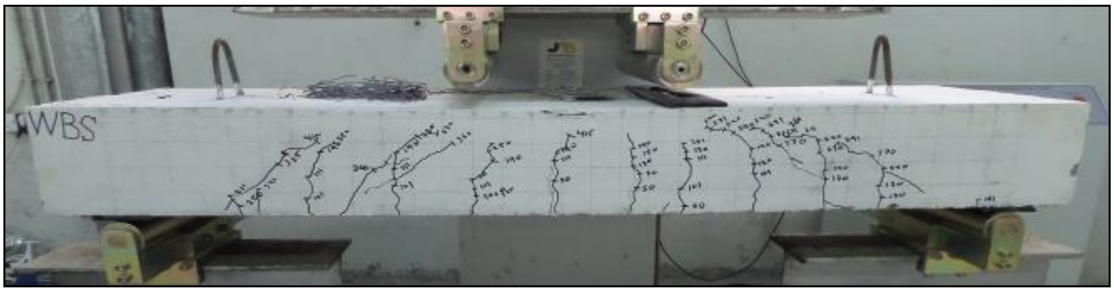
From these figures the following conclusions can be drawn:

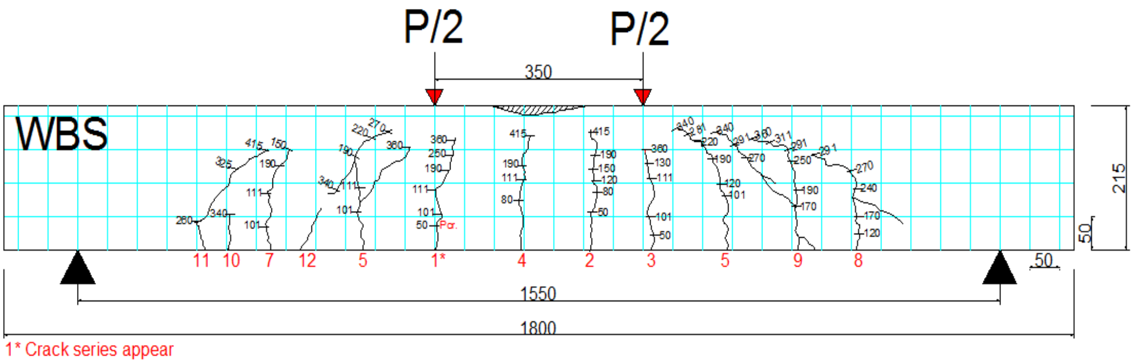
1. Due to the constant moment applied within the middle third of the beam the sequence of formation of cracking was random, and cracks grew upward with the increase of the applied load.
2. Cracks forming within the middle third of the beams were generally vertical due to the pure moment applied on this part of the beam. Outside this zone the cracks became inclined due to the presence of shearing forces in addition to the moment.

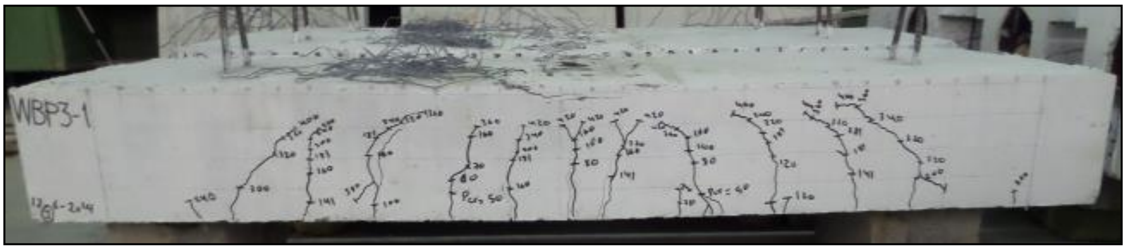
The specimen WBP3-3 was failed under high shear action. When the diagonal crack occurs, there must be a redistribution of internal forces at the cracked section. And when the beam has no web reinforcement, the external shear resisted by the concrete web must be redistributed partly to the tensile reinforcement through dowel action but mainly to the compression zone of concrete. The redistribution must take place by the web reinforcement. For the WBP3-3 the failure was immediately happen, it is possible interpreted that the member does not accept any redistribution when the diagonal crack forms. In this case, the web reinforcement will yield immediately and the compression zone will be destroyed immediately [7].

4.7 First crack width

It can be seen from Table (6), the crack width of the first crack at cracking and yield load. The first crack appeared randomly within the middle third of the span (zone of maximum moment), was not necessarily the widest one. The first crack width at cracking load is equally in all specimens as a result of the same properties of concrete and longitudinal reinforcing. But at yield load the crack width of the first crack was decreased by 50% for the specimen WBP3-1 and increased by 62.5% for the specimen WBP3-3 compared for WBS specimen. But it can be seen that the number of shear cracks is increased by 12.5% and 25% for the specimens WBP3-2 and WBP3-3 respectively compare with WBS, but it decrease by 25% for WBP3-1.







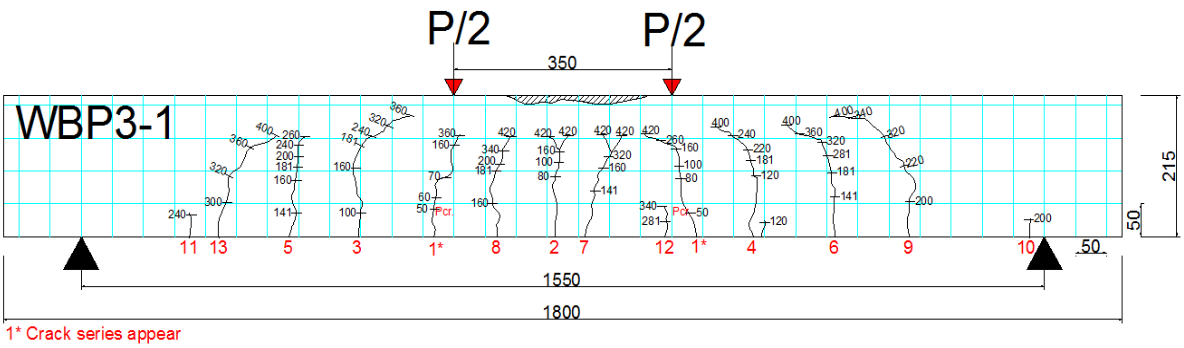
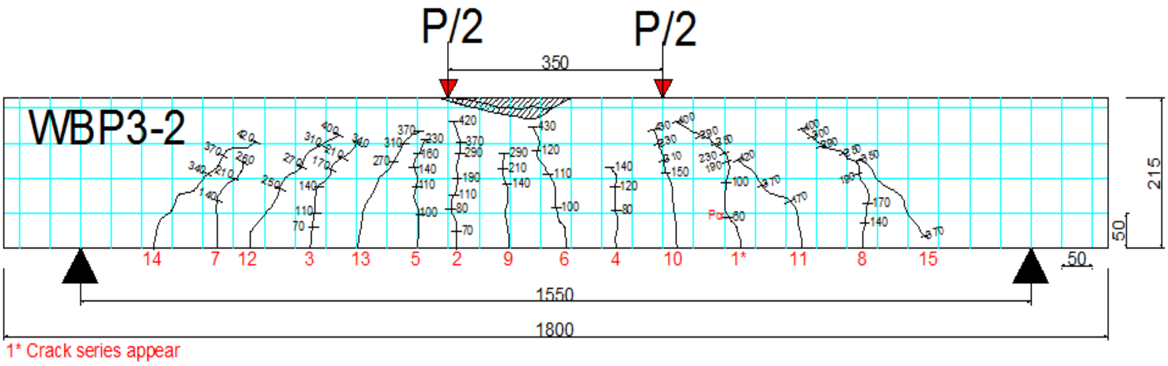
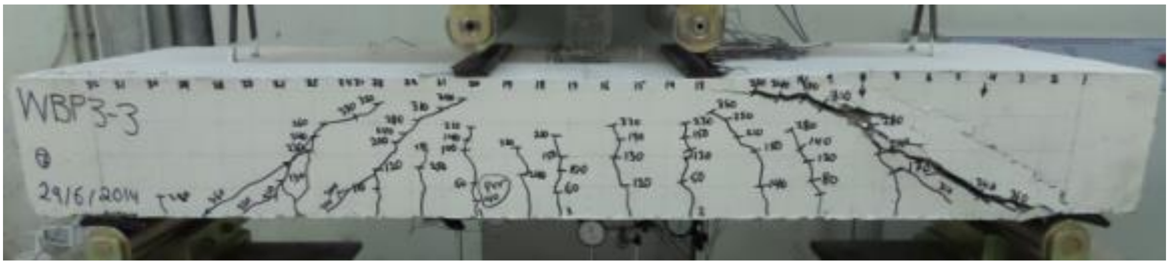
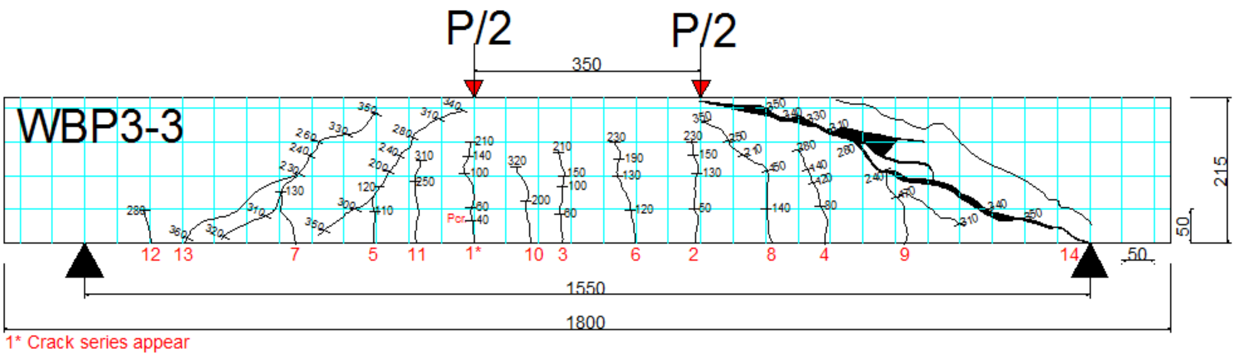


Plate (1) Cracking patterns of specimens







Continued- Plate (1)

4.8 Shear cracks spacing

It can be seen from Table (6) that with increasing the spacing of steel plates, the minimum spacing of shear cracks was equal with WBS but it was increased by 20% for the specimen WBP3-2 compared with others. And no significant effect for average crack spacing.

By increasing the spacing for shear steel plate, the maximum spacing of WBP3-2 was decreased by 15% compared with WBP3-1 as a result of increasing of shear cracks, but for the WBP3-3 the maximum crack spacing was increased by 115% as a sudden shear failure. The average crack spacing was decreased for WBP3-2 and WBP3-3 by 12.5% and 17% respectively compared with WBP3-1 specimen as a result of increasing of shear cracks.

Table (6) First crack width, number and spacing of shear cracks for specimens

Beam Specimens	*1st Crack at Cracking*		*1st Crack at Yield*		*No. of Cracks*		*Spacing of Shear Cracks(mm)*		
	Load (kN)	Width (mm)	Load (kN)	Width (mm)	Flex.	Shear	Min. Spacing	Max. Spacing	Average Spacing
WBS	50	0.005	400	0.16	4	8	50	120	110
WBP3-1	50	0.005	420	0.24	6	6	50	150	116
WBP3-2	60	0.005	400	0.16	6	9	60	110	105
WBP3-3	40	0.005	370	0.26	5	10	50	280	100

4-9 Comparison the first crack width computing by ACI 318M-14(1) and EC 2(2) codes

It can be seen from Table (7), the experimental crack width of the first crack at service and yield load comparing with predicted first crack width according the ACI 318M-14(1) and EC 2(2) codes. The experimental crack width of all specimens at service and yield load was 0.13mm and 0.22mm on average. The predicted crack width at service and yield loads of wide beams computing by ACI 318M-14(1) and EC 2(2) codes (0.21 and 0.33) mm and (0.12 and 0.19) mm on average respectively. So it can be seen that:

1. The experimental crack width at service and yield load was less than predicted crack width computed by ACI 318M-14(1) by 38% and 33% respectively.
2. The experimental crack width at service was close to predicted crack width computed by EC 2(2) but it was upper by 16% at yield load.
3. From (1) and (2) above EC 2(2) code was more conservative than ACI 318M-14(1).

Table (7) Comparing of experimental first crack width with crack width computing by of ACI 318M-14(1) and EC 2(2) cods at service and yield load

Beam Specimens	*Crack Width (mm)*					
	Experimental		According ACI 318M-14		According EC 2	
	At Service load	At Yield load	At Service load	At Yield load	At Service load	At Yield load
WBS	0.14	0.16	0.217	0.327	0.126	0.190
WBP3-1	0.14	0.24	0.213	0.327	0.124	0.190
WBP3-2	0.10	0.16	0.217	0.327	0.126	0.190
WBP3-3	0.14	0.26	0.187	0.327	0.109	0.190

4-10 Comparison the crack spacing computing by ACI 318M-14[1] and EC 2[2] codes

Table (8) shows the measured and predicted values of crack spacing according to BS8110-85[9] and EC 2[2] only because no such formulas were proposed in other codes of design [1]. In BS8110-85[٨], the average crack spacing approximately equal 1.67(h-x) for primary cracks, in this method the height of neutral axis determines the spacing of cracks. It can be seen from Table (8) that:

(1) The predicted mean crack spacing according to BS8110-85[٨] ranged between 215mm and 224mm for all four wide beams tested. And the experimental average crack spacing ranged 91mm to 111 mm. By comparing the values obtained experimentally and those predicted using BS8110-85[٨] formula, it can be seen that these values are different from those obtained experimentally by 120% on average.

(2) The predicted minimum and maximum crack spacing according to EC 2[2], 92mm and 160mm respectively all four wide beams tested. From Table (8) it can be seen that the EC 2[2] formula did not consider the concrete compressive strength, thus for all the investigated specimens, the crack spacing were the same for beams with the identical reinforcement. This was not the situation obtained experimentally; the minimum and maximum crack spacing was bounded by (40mm to 60mm), and by (51mm and 152 mm) respectively. By comparing the values obtained experimentally and those predicted using EC 2[2] formula, it can be seen that these values are different from those obtained experimentally by 45% and 22% on average.

A modification in the formulas proposed by BS8110-85[8] and EC 2[2] are needed to consider the spacing of shear reinforcement.

4. 11 Comparison between the weights of specimens

It can be seen from Table (1) that using the shear steel plates was reduced the weight by 2.3%, 2.7% and 3.1% for specimens WBP3-1, WBP3-2 and WBP3-3. It is clear that using shear steel plates is reduced the weight of specimens by 2.7%.

Table (8) Comparing of number of cracks and experimental crack spacing with crack spacing computing by of BS 8110[٨] and EC 2[2] cods

Beam Specimens	*No. of Cracks*		*Spacing of Cracks (mm)*					
			Experimental			According BS 8110	According EC 2	
	flexural	Shear	Min. Spacing	Max. Spacing	Average Spacing	Mean Spacing	Min. Spacing	Max. Spacing
WBS	4	8	50	150	106	223	91.8	160
WBP3-1	6	6	50	135	100	221	91.8	160
WBP3-2	6	9	60	110	91	224	91.8	160
WBP3-3	5	10	50	280	111	216	91.8	160

5. Conclusions and Recommendations

5.1 Shear steel plates is a good alternative for replacing stirrups (as web reinforcement) and gives identical result in yield and ultimate load (with increasing 5% of yield load).

5.2 Although the specimen with shear steel plates gives an increase in deflection by 24% compared with that of stirrups, but it gives increasing in ductility with 55%.

5.3 Using shear steel plate, reduce the strain in exterior and interior legs by 17% and 46% compared with stirrups, although the yield strength of shear steel plates is less than stirrups yield strength by 47%.

5.4 Using the shear steel plates instead of stirrups reduced the total weight of wide beams by 2.7%.

5.5 By using steel shear plates with 3mm thickness, with spacing between steel plates of 125, 166 and 250 mm (increasing the spacing by 33% and 100%), it can be notified:

5.5.1 The deflection at yield was increased by 2% and 16% respectively.

5.5.2 The yield load reduced to 5% and 12% respectively.

5.5.3 The strain in longitudinal reinforcing was decreased by 10% and 26% respectively at yield load.

5.5.4 The strain in exterior leg was increased by 87% and 80% respectively at yield load and by 143% and 31% respectively for the interior legs.

5.5.5 At yield: the strain in interior legs is more than the strain in exterior leg by 31%, 70% and 1%.

5.5.6 The number of shear cracks is increased by 50% and 67% respectively, so that the average crack spacing was decreased by 12.5% and 17% respectively.

5.6 The predicted deflection of wide beams computing by ACI 318-14(1) and EC 2(2) codes were less than the experimental deflection by 19% and 21% on average respectively.

5.7 EC 2(2) code was more conservative than ACI 318M-14(1) to predicted the crack width.

5.8 A modification in the formulas proposed by BS8110-85(8) and EC 2(2) are needed to consider the spacing of shear cracks.

6. References

1. ACI Committee 318M-14, 2014,“Building Code Requirements for Structural Concrete and Commentary,” American Concrete Institute, Farmington Hills, MI, 465 pp.
2. Eurocode 2, 2004, “Design of Concrete Structures—Part 1-1: General Rules and Rules for Buildings (EN1992-1-1),” European Committee for Standardization, Brussels, Belgium, Dec225 pp.
3. Adam S. Lubell, Edward G. Sherwood, Evan C. Bentz, and Michael P. Collins, 2006, “One way shear strength of thick slabs and wide beams” ACI Structural Journal, vol. 103, no. 6, Nov. - Dec. 794-802 pp.

4. Adam S. Lubell, Evan C. Bentz, and Michael P. Collins, 2009 “Shear reinforcement spacing in wide members, ACI Structural Journal, vol. 106, no. 2, March - April, 205-214 pp.
5. Mohamed M. Hanafy, Hatem M. Mohamed and Nabil A.B. Yehia, 2012, “On the Contribution of Shear Reinforcement in Shear Strength of Shallow Wide Beams” Life Science Journal vol. 9, no. 3, 484-498 pp.
6. Ferdinand P. Beer E. Russell Johnston, Jr. John T. De Wolf**,** Mechanics of Materials", Lecture Notes: J. Walt Oler Texas Tech University, 2009, 415-428pp.
7. ACI-ASCE Committee 326, shear and diagonal Tension, (1962), 1-124pp.
8. 1985, “Structural Use of Concrete –Part 2 Code of Practice for Design and Construction (B.S.8110-2)”, British Standard Institution, London, P.20 .