

Shear Behavior of Steel or GFRP Reinforced Concrete Beams Without Stirrups

Renata Kotynia¹(✉), Monika Kaszubska¹, and Joaquim A.O. Barros²

¹ Faculty of Civil Engineering, Architecture and Environmental Engineering,
Lodz University of Technology, Al. Politechniki 6, 90-924 Lodz, Poland
{renata.kotynia,monika.dymecka}@p.lodz.pl

² Department of Engineering, School of Engineering, University of Minho,
Campus de Azurém, 4800-058 Guimarães, Portugal
barros@civil.uminho.pt

Abstract. The paper presents results of experimental test carried out on 12 T-shaped concrete beams reinforced with glass fiber reinforced polymer (GFRP) or steel bars without stirrups. The aim of this research was to analyze the influence of a type of longitudinal reinforcement (GFRP or steel) on shear capacity and deformability of concrete beams without stirrups and to investigate a dowel effect of the reinforcement on the shear strength. The beams varied mainly with a type of flexural reinforcement, its reinforcement ratio (ρ_l) corresponding to about: 1%, 1.4% and 1.80%, a number of bars, their diameter and a number of reinforcement layers (1 or 2 layers). All beams failed in shear, but diagonal shear cracking was affected by a type longitudinal reinforcement. The GFRP reinforced beams failed due to gradual development of diagonal cracks, while in beams with the steel reinforcement diagonal cracking developed rapidly leading a brittle failure. Steel reinforced beams indicated higher shear capacity than the GFRP reinforced beams. Beneficial influence of two reinforcement layers was confirmed especially for the high longitudinal reinforcement ratio equal of 1.80%, while for the low reinforcement ratio about 1.0%, no difference in the shear capacity due to number of layers was observed. Due to almost four times lower elasticity modulus of the GFRP bars than steel, the GFRP reinforced beams indicated higher ductility than the steel reinforced beams.

Keywords: Shear strength · T-shaped beams · GFRP and steel reinforcement · Failure

1 Introduction

Shear issue in the support regions of reinforced concrete (RC) beams is a complex problem intensively investigated all over the world. It is due to combination of shear force and bending moment appearing in the support region of RC beams. The shear strength of RC members without transverse reinforcement is affected by several shear mechanisms including: un-cracked concrete compressive zone, friction forces (aggregate interlock action) developing along a diagonal shear crack length (only in limited slip range of the shear crack edges and limited crack opening), the residual tensile

strength existing between inclined cracks and the shear dowel action of longitudinal reinforcement (the last two mechanisms have a small contribution in the shear strength of the beams without transversal reinforcement). The contribution of each shear component changes depending on the external load level and a crack pattern. Shear issue is much more complicated when the longitudinal reinforcement is made of fiber reinforced polymer (FRP), which in opposite to steel is an anisotropic material (El-Sayed et al. 2006; Jin et al. 2009; Tureyen and Forsch 2002; Alam and Hussein 2013). In a case of FRP reinforced beams without stirrups the crack width is bigger than it is observed in steel reinforced members. Recently this non-metallic reinforcement gained common acceptance, due to its good durability resistance, high strength-to-weight ratio, good fatigue and non-magnetic properties. However, the FRP bars indicate mechanical properties different from steel, including a linear elastic stress-strain characteristic up to failure that implies a lack of the plastic behavior and significantly lower elasticity modulus than steel. The aim of the undertaken research was to investigate the influence of a dowel effect on the shear capacity of FRP reinforced beams.

2 Test Program

A comprehensive experimental program was carried out to investigate an effect of axial stiffness of longitudinal reinforcement made of steel and GFRP bars on the shear strength of RC beams without stirrups. Other investigated parameters contained: number of layers (one or two), reinforcement ratio, diameter and number of bars.

2.1 Test Specimens, Setup and Methodology

Test program included twelve T-section single span, simply supported concrete beams with a clear span of 1800 mm, tested in three-point monotonic loading. Shear span to depth ratio, (a/d_{eq}) was approximately 3.0 in all beams. Two main series of beams were reinforced with longitudinal GFRP bars (7 beams) and steel bars (5 beams) of diameters: 12 mm, 16 mm and 18 mm. All straight steel bottom bars were welded on the vertical steel plate, while GFRP bars were anchored in the steel box filled with an epoxy resin. The top longitudinal reinforcement consisted of two straight bars of 10 mm diameter (GFRP or steel, respectively to the series). There were no stirrups along the tested shear span, while the opposite non-tested shear span was reinforced with steel stirrups of 8 mm diameter at 130 or 150 mm spacing and steel bent bars of 14 mm diameter to prevent the shear failure in this support (Fig. 1).

For identifying each beam a designation $Z-n\phi X(m\phi Y)$ was used, where Z was a type of longitudinal reinforcement (namely: G for GFRP, S for steel), n was a number of bars of X diameter (in mm) in the first layer, and when a second layer exists, m was a number of bars of Y diameter of the second layer. For instance, S-3 ϕ 16 represented the beam reinforced with one layer of 3 steel bars of 16 mm diameter, while G-3 ϕ 18/1 ϕ 18 was the beam of two layers, the first one composed of 3 GFRP bars of 18 mm diameter and the second one composed of 1 GFRP bar of 18 mm diameter.

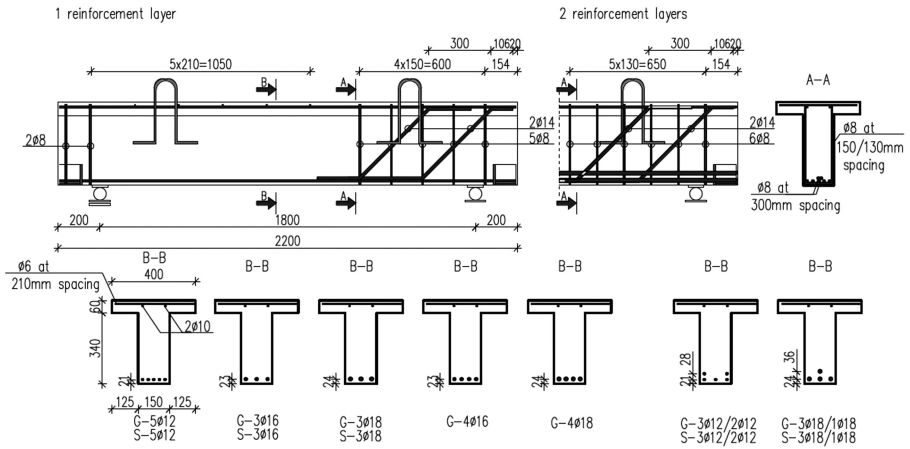


Fig. 1. Reinforcement of tested beams

The beams were casted using ready mix concrete class C25/30 delivered from the local batch plant. The average cube and cylinder concrete strength (obtained on 16 specimens) was 33.4 MPa (COV = 5%) and 31.7 MPa (COV = 8%), respectively. The average tensile strength equaled of 2.9 MPa (COV = 8%) and average modulus of elasticity was 26.7 GPa (COV = 6%). The maximum aggregate size of concrete mixture was 8 mm diameter.

The experimental characteristics of steel and GFRP reinforcement was determined on 18 and 12 specimens, respectively. The average modulus of elasticity and the maximum tensile strength registered in the test of GFRP bars equal of 51.5 GPa (COV = 9%) and 1091 MPa (COV = 11%), respectively. In case of steel bars, the average modulus of elasticity was 213 GPa (COV = 6%), the yield strength was 532 MPa (COV = 6%) and the tensile strength was 642 MPa (COV = 3%).

The beams were simply supported on two steel supports with the movable one closed to the shear tested part of the beam (Fig. 2). The system for displacement control

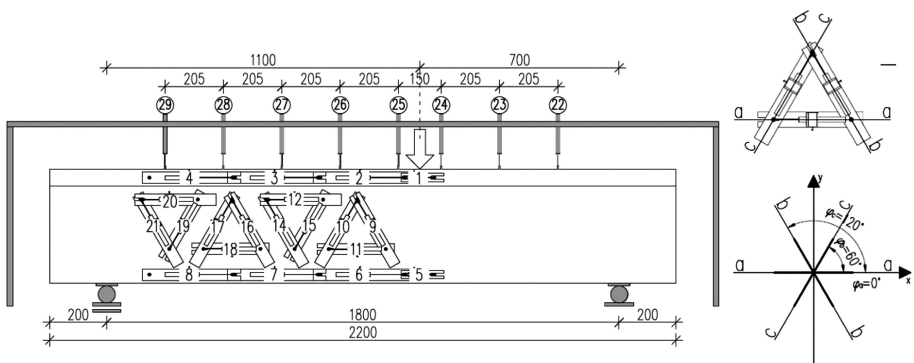


Fig. 2. Test set up

loading was applied monotonically with velocity of 10 $\mu\text{m/s}$ by the steel cross-beam on the whole width of the flange. Concrete strain were registered with LVDT gauges in compressive and tensile zone (no 1–4 and no 5–8, respectively) as well as on the triangular rosettes (no 9–21). Vertical displacements were registered by LVDTs of 100 mm range (no 22–29) mounted to an independent steel frame (Fig. 2).

3 Analysis of Test Results

All beams failed due to shear. Diagonal cracks formed within the shear span developed along the longitudinal tensile reinforcement. The first flexural cracks occurred in the beam mid-span. During the load increasing flexural cracks developed and a diagonal crack appeared in the shear region. Then the shear crack developed its width increased until shear failure. The test results are presented in Table 1.

Table 1. The main results

Elements	f_{cm} [MPa]	f_{ct} [MPa]	ρ_l [%]	$\rho_l E$ [GPa]	A [mm ²]	d_{eq} [mm]	V_u [kN]	τ_u [MPa]
G-5 Φ 12	30.2	2.75	0.99	0.51	565	379	34.3	0.60
G-3 Φ 16	28.8	2.95	1.07	0.55	603	377	31.7	0.56
G-3 Φ 18	28.8	2.95	1.35	0.70	763	376	38.6	0.68
G-4 Φ 16	30.5	2.70	1.42	0.73	804	377	34.8	0.61
G-4 Φ 18	28.8	2.95	1.80	0.93	1018	376	38.2	0.68
G-3 Φ 12/2 Φ 12	31.7	3.05	1.02	0.53	565	368	34.8	0.63
G-3 Φ 18/1 Φ 18	31.7	3.05	1.85	0.95	1018	367	47.7	0.87
S-5 Φ 12	28.8	2.95	0.99	2.11	565	379	55.6	0.98
S-3 Φ 16	31.7	3.05	1.07	2.28	603	377	52.6	0.93
S-3 Φ 18	33.1	2.85	1.35	2.88	763	376	56.1	0.99
S-3 Φ 12/2 Φ 12	33.1	2.85	1.02	2.17	565	368	50.9	0.92
S-3 Φ 18/1 Φ 18	33.1	2.85	1.85	3.94	1018	367	61.8	1.12

$\rho = A/b_w d_{eq}$: A – cross section of reinforcement, d_{eq} – equivalent depth of reinforcement
 $d_{eq} = (A_1 * d_1 + A_2 * d_2)/A$: d_1, d_2 – depth of first and second reinforcement layer, A_1, A_2 – the cross section of first and second reinforcement layer

To avoid the influence of different flexural reinforcement depth (d_{eq}) on the ultimate shear force (V_u) further analysis was performed according to the shear stress calculated as $\tau_u = V_u/(b_w d_{eq})$.

Due to four times higher elasticity modulus of steel than GFRP bars, the beams steel reinforced indicated significantly higher shear strength than the beams reinforced with GFRP bars. The ultimate shear stress in the steel reinforced beams was over 60% higher (in the beams with reinforcement ratio of $\rho_l = 1.0\%$) and 45% higher (in the beams with $\rho_l = 1.35\%$) than ultimate shear stress in GFRP reinforced beams with corresponding reinforcement ratio.

Due to high difference in modulus of elasticity of steel and GFRP bars it is reasonable to consider axial stiffness ($\rho_l E$) influence on the shear strength of beams instead of separate effects of the reinforcement ratio (ρ_l) and the elasticity modulus (E).

Only 5% increase in the shear strength was observed by application of two reinforcement layers instead one layer in case of the lowest reinforcement ratio (0.99% and 1.02%) for both types of bars (GFRP and steel). However, application of two reinforcement layers instead of one layer in beams G-3 ϕ 18/1 ϕ 18 and G-4 ϕ 18 (with reinforcement ratio of 1.8% and 1.85%, respectively) indicated an increase in the shear strength of about 28% (Table 1). Comparing beams GFRP and steel reinforced beams with two reinforcement layers the differences between the shear strength of steel and GFRP reinforced beams were 46% and 29% for the reinforcement ratio equal of 1.02% and 1.85%, respectively. Configuration of two reinforcement layers was more effective than one layer, because a larger volume of concrete surrounding the bars was resisting to the tensile force, which not only led to the higher resistance to the opening of the shear cracks (improving the aggregate interlock effect), but also led to the sliding of these cracks (due to dowel effect). No significant change of the shear strength in beams with the reinforcement ratio $\sim 1\%$ and the significant increase in shear strength in beams with the reinforcement ratio $\sim 1.8\%$ confirmed a higher influence of the dowel effect than the aggregate interlock effect on the shear strength.

In GFRP reinforced beams almost twice increase in the reinforcement ratio (from 0.99% to 1.8%) caused an increase in the normalized shear strength only 13% for one reinforcement layer, while this increase reached 38% for two reinforcement layers. The increase in the reinforcement ratio from 1.02% to 1.85% in the steel reinforced beams with two layers caused the increase in the shear strength of 22%, that was much smaller than in GFRP reinforced beams (Table 1). The GFRP reinforced beams indicated gradual and slow diagonal cracking development, while failure of the steel reinforced beams was sudden and brittle.

The shear cracking stress ($\tau_{cr} = V_{cr}/bd$, where V_{cr} is the cracking shear force) reached values $0.11f_{ct} - 0.14f_{ct}$ in almost all GFRP reinforced beams, which correspond to $0.49\tau_u - 0.64\tau_u$ (τ_u - the ultimate shear strength, calculated as $\tau_u = V_u/(b_w d_{eq})$, where V_u is the ultimate shear force) (Fig. 3).

One exception was the beam G-4 ϕ 16, with shear cracking stress $\tau_{cr} = 0.18f_{ct}$, which corresponds to $\tau_{cr} = 0.78\tau_u$. The shear cracking stress in the beams reinforced with steel bars was much higher than the cracking stress in the beams GFRP reinforced, that corresponded to $\tau_{cr} = 0.25 - 0.35 f_{ct}$ and $\tau_{cr} = 0.76 - 0.98 \tau_u$ (Fig. 3).

Four times higher elasticity modulus of the steel bars than GFRP bars led to significantly stiffer flexural behavior of the steel reinforced beams, which delayed shear concrete cracking. The lower modulus of elasticity of GFRP than steel bars caused, that almost all vertical flexural cracks in GFRP reinforced beams extended into the entire height of the web and more cracks appeared in the GFRP reinforced beams than in steel reinforced beams with the same longitudinal reinforcement ratio (Fig. 4).

Several new small cracks appeared closed to the support, which confirmed bond loss of GFRP bars to concrete. The splitting cracks did not pass beyond the support.

The critical shear crack inclination ranged from 35° to 51° in the GFRP reinforced beams and from 30° to 53° in the steel reinforced beams. The GFRP reinforced beams

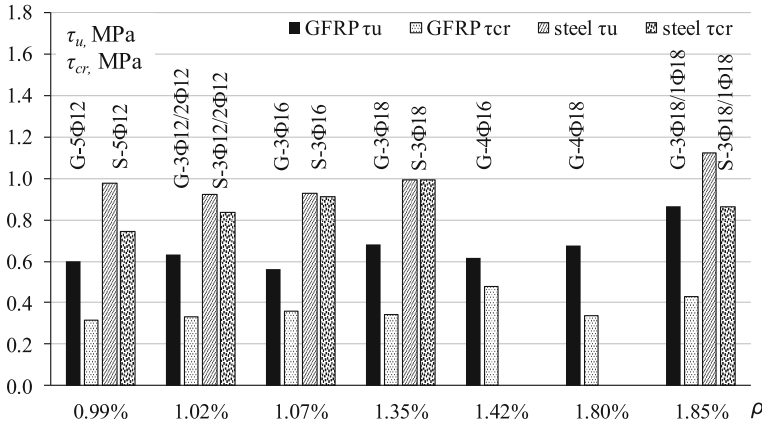


Fig. 3. Comparison of ultimate and cracking shear stress in tested beams

with the same reinforcement ratio indicated more cracks in the beams with a larger number of bars than in the beams with smaller number of bars (Fig. 4).

The average strains ϵ_a , ϵ_b , ϵ_c were measured by LVDT gauges on the bases of 200 mm length (Fig. 2) and they were used for calculation of the principal strain ϵ_1 and ϵ_2 according to the following formula:

$$\epsilon_{1,2} = \frac{1}{3}(\epsilon_a + \epsilon_b + \epsilon_c) \pm \frac{\sqrt{2}}{2} \sqrt{(\epsilon_a - \epsilon_b)^2 + (\epsilon_b - \epsilon_c)^2 + (\epsilon_a - \epsilon_c)^2} \quad (1)$$

For analysis of the principal strain ϵ_1 and ϵ_2 the only rosettes covering the critical shear crack were taken into consideration. The principal strain plots in a function of the shear stress τ are shown for steel reinforced beams and GFRP reinforced beams in Fig. 5a and b, respectively. Beams G-4 ϕ 18 and S-3 ϕ 18 were excluded from the analysis of the principal strain, because the shear crack missed triangular rosettes.

With increase in the longitudinal reinforcement ratio, the increase in the shear cracking stress τ_{cr} was noted. The shear cracking was assumed based on any concrete strain values ϵ_a , ϵ_b , $\epsilon_c > 0.0001$, registered by LVDTs crossed by the shear crack (Fig. 5a and b). The beams G-4 ϕ 16 and G-3 ϕ 18/1 ϕ 18 with the higher reinforcement ratio (1.42% and 1.85%, respectively) indicated smaller principal tensile strain and higher shear cracking stress, while the beams G-5 ϕ 12 and G-3 ϕ 12/2 ϕ 12 with the reinforcement ratio about ($\sim 1\%$) had similar principal tensile strain, but beams with two reinforcement layer had much higher shear cracking stress (both for steel and GFRP reinforced beams). Due to much higher ductility of GFRP RC beams, the principal tensile strain at cracking and failure in these beams were much higher than corresponding principal strains in steel RC beams (compare Fig. 5a and b), while the diagonal shear cracking stress in GFRP reinforced beams were much lower than those in steel reinforced beams.

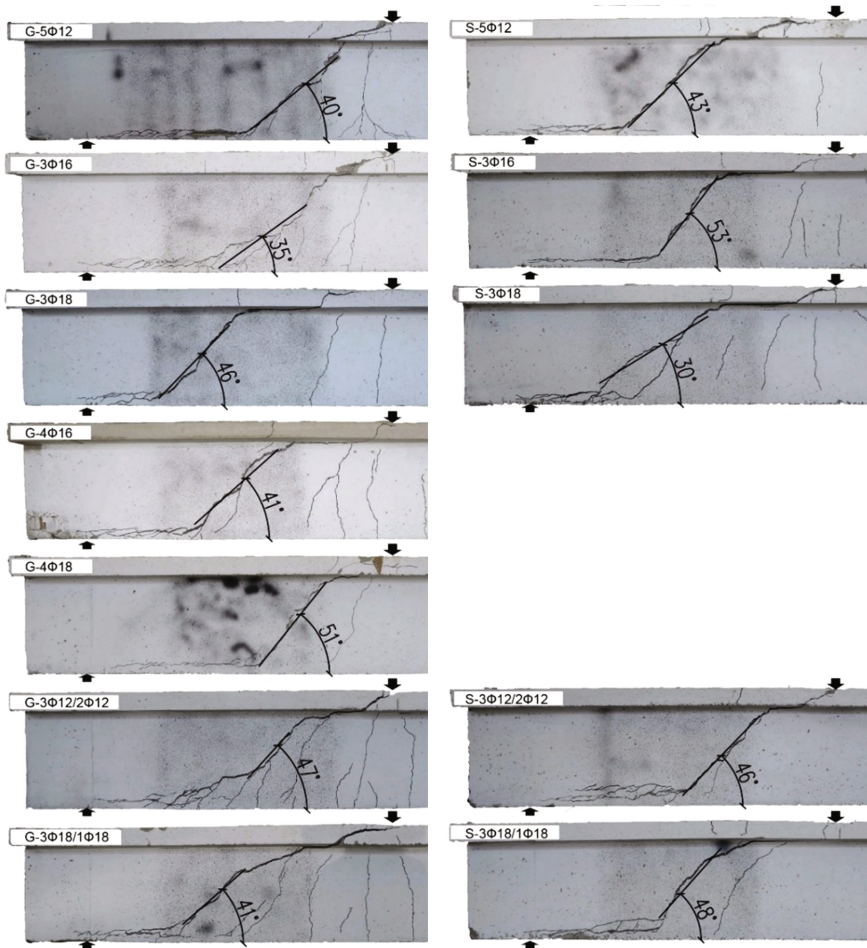


Fig. 4. Crack pattern of tested beams

The average vertical displacement registered by LVDTs no. 24 and 25 (see Fig. 2b) are shown in Fig. 6. All the beams showed linear behavior before and after cracking. The stiffness of the beams, before flexural cracking, was similar for beams with corresponding reinforcement ratio irrespectively of the reinforcement (GFRP or steel). The shear stress vs. deflection curves indicated differences after beams cracking, depending on a type and amount of reinforcement. Significantly lower deflections were observed in the steel reinforced beams (up to 2.5 mm) than in the GFRP reinforced beams (up to 5 mm). As it was expected, the increase in the reinforcement ratio caused the decrease in deflection.

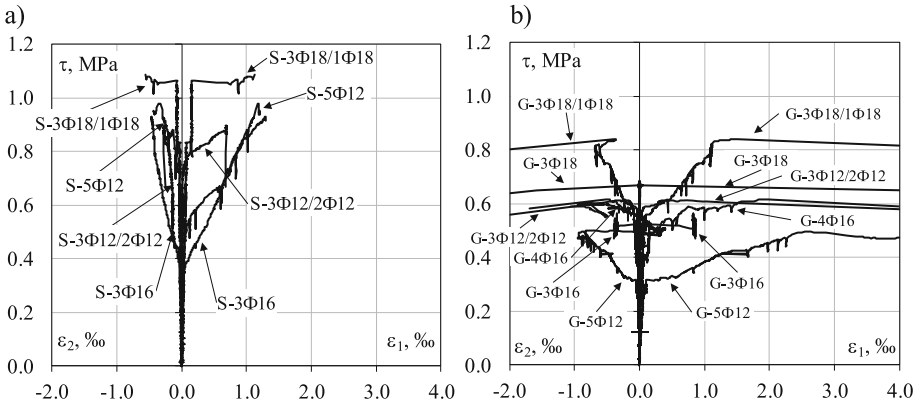


Fig. 5. Principal concrete strain ϵ_1, ϵ_2 for (a) steel, (b) GFRP reinforced beams

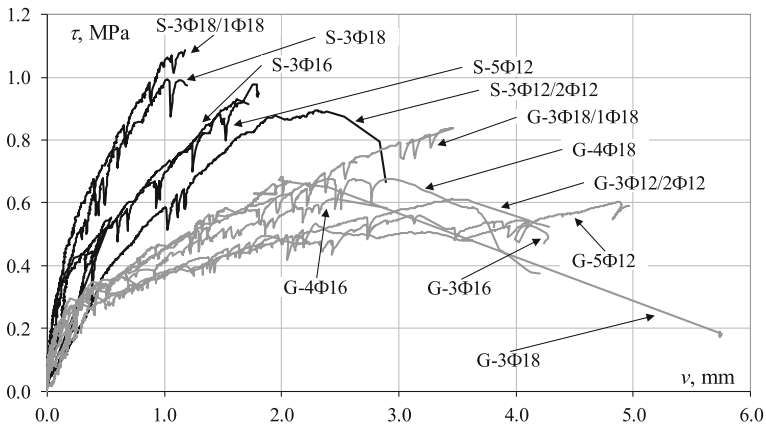


Fig. 6. Shear stress vs. average mid-span deflection in tested beams

4 Conclusions

Based on the analysis of the test results the following conclusions can be drawn:

- the steel reinforced beams reached much higher ultimate shear strength than the GFRP reinforced beams;
- shear cracking stress was mainly affected by the concrete strength and a type of reinforcement;
- both GFRP and steel RC beams were not susceptible to changes of reinforcement ratio if the reinforcement ratio was below 1.40%;
- application of the GFRP longitudinal tensile reinforcement in two layers delayed the shear failure and increased the shear strength by near 28%;
- beams reinforced with GFRP reinforcement indicated higher ductility than the beams reinforced with steel.

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