Strengthening of reinforced concrete roof girder with unbonded tendons cracking due to the exploitation

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ABSTRACT: In the paper the practical possibility of strengthening of reinforced concrete roof girder with unbonded tendons is presented. The mention above girder, double T in cross section, has been cracked due to the external dead loading. During control of existing technical state it was located about 140 normal cracks in the bottom flange coming to the web as well as the diagonal cracks. The maximal width of normal cracks was equal to the 0.4 mm, and the diagonal cracks was equal to 0.6 mm. The system of strengthening is discussed. The influence of prestressing to structure deformation has been investigated. The readings of concrete strains at upper, bottom flanges and web girder have been taken with mechanical gauges DEMEC type. The results obtained from investigations were compared to the ones from FEM numerical analysis. The final findings dealing with the applied technology of strengthening has been also discussed.

1 INTRODUCTION

Strengthening reinforced concrete girders by means of external prestressed unbonded tendons is widely used on the territory of Poland. For many years, technical literature sources have been discussing the problems related to the proper way of strengthening. The lack of load capacity occurring in flexural reinforced concrete beams can be dealt with by using additional reinforcement together with epoxy concrete, using prestressed bars, normal or prestressed CFRP plates, and external unbonded steel tendons. The choice of an appropriate strengthening method usually depends on technological potential, a kind of strengthened element, its cross section, access to a given element in case of a beam for the bottom flanges of a given girder and its support region, it depends on the lack of load capacity level and an designer's preferences.

External unbonded tendons are used both in new structures and in strengthening already existing reinforced concrete and prestressed structures.

In new structures, unbonded tendons are used in box girder bridges where tendons are placed inside the cross section of a given girder but beyond its concrete cross section, and in prefabricated water tanks. When it comes to structure strengthening, prestressed unbonded tendons can also be used in case of reinforced concrete or prestressed girders. The main advantage of using prestressed unbonded tendons is the possibility of element strengthening by means of adding external prestressed tendons without the need of exchanging faulty elements of the structure. The use of unbonded tendons makes it possible to perform a periodical screening of all tendons and their exchange, if needed. External prestressed tendons attached to already existing girders can have linear or curved tendon layouts. In case of a linear tendon layout one has to use tendon layout stabilizers in order to avoid second-order effects causing the change in tendon eccentricity during beam deflection. Curved tendon layouts are implemented by attaching one or two deviators, depending on beam slenderness.

The research presented below, based on real roof girders, prestressed at their dead load, initiated a new series of research on strengthened structures, which was carried out in the Institute of Building Materials and Structures of Kraków University of Technology. Experiments carried out on the real structure and laboratory research will make it possible to answer the question whether a given strengthening method is effective, and existing lack of structure load capacity was eliminated, and whether it is necessary to implement difficult and very costly injection of occurring cracks in strengthened reinforced concrete beams.

2 METHOD OF STRENGTHENING

The research presented below was based on two slope single span reinforced concrete girders, described as “L” (Fig. 1), which are 25 m long and constitute part of the structure of an industrial plant covering. The industrial plant has got a column and beam structure. The girders were placed every 7.5 m. Every second girder is supported with an intermediate 15 m beam or directly with a column. Double T cross section of the investigated element is changeable along its length,
from 1.6 m in its support region to 1.8 m in its middle span. Along the web, there are six openings 0.8 m in diameter, placed every 2.2 m, in order to decrease its dead load (Fig. 1–2). The dead load and live load of roof structure, together with the load of airconditioning machinery placed on the roof, are transmitted through of four steel beams placed in the distance of 5 m, along the length of the beam. Figure 2 shows the girder’s view. The main reinforcement contains 6 ø28 mm bars and 2 ø14 mm bars, made of normal steel, f_{pk} = 410 MPa. The stirrup were made of ø6 mm reinforced steel bars, placed every 0.1 m. There was C45/55 class for concrete.

During the inventory control of structure elements, numerous cracks in girders were observed. Figure 3 shows the example of crack pattern for the girder of this type. The average crack distance was estimated for 145 mm. Flexural crack widths reaches 0.4 mm, shear cracks can be up to 0.6 mm wide. There was the lack of load capacity: 20% taking into account dead load, 31% at live load; in this case snow loading indicate live load.

Each beam was strengthened with by means of four unbonded tendons of 7 ø5 mm type, with the cross sectional area of 150 mm². The tendons were made of galvanised steel of f_{pk} = 1770 MPa and the modulus of elasticity equaling 190 GPa. Galvanised tendons were strengthened with a double HDPE duct. The pre-stressed force of each tendon relating to the “L” girder was 200 kN. The tendons were fixed at the bottom of girder (Fig. 1). The fixed anchorage of external tendons were fixed in steel elements in support region.
Stressing anchorage was fixed in the middle span of the girder (Fig. 4–5).

Four prestressed tendons leading from fixed anchorage were installed in stressing anchorage. The result was 4 tendons divided into 8 segments. The tendons were placed in wedge anchorages.

Prestressed tendon eccentricity was achieved by means of two steel deviators (Fig. 6). The deviators made it possible to deflect tendon layouts by 270 mm from the straight line joining fixed anchorage.

All steel elements fixed on the girder were galvanized. Due to shear, FRP mesh fabrics were attached to the web in the support region and in the area between the first and the second opening, counting from the support region of the beam. All cracks wider than 0.1 mm were injected with epoxy resin.

3 THE RESULTS OF THE RESEARCH AND NUMERICAL CALCULATION

During the renovation process an extensive research programme was carried out. The 270 measuring points were fixed on the “L” girder. As a result, it was possible to measure girder strains during prestressing. The results were provided by DEMEC mechanical gauges, base 200 mm and 150 mm (vertical upper flange strain measurement). Vertical direction strains were not presented. The pattern of the measuring points on the “L” girder is presented in Figure 7. Figure 8–10 presented the “L” girder horizontal direction stress deriving from the measured strains (adopting the concrete modulus of elasticity like for C45/55 concrete, $E_{cm} = 36$ GPa), and presented stress horizontal direction deriving from numerical analysis.

The whole “L” girder was subject to numerical analysis. This paper presents the comparative results at measuring points. The numerical analysis was carried out by means the FEM, in “Robot” programme (linear analysis). In Figure 8–10 it can be seen the increase of horizontal direction compression stresses in the whole bottom flange of the girder, the occurrence of tensile stresses in the upper flange and in the upper part of the web. In both cases it can be notice the lowering of the neutral axis at the web in the area between the openings. The neutral axis is placed quite high.

In the research neutral axis can be found in the area where the web touches the upper flange. In the support region, in the upper flange, there is tensile stresses, and the lower its value, the closer one is to the edge of the beam. It can be notice the decrease of stresses on the left and the right side of the openings.

While comparing the research and numerical results, it can be seen that, in case of the research carried out along the height of the girder, stress distribution is irregular. It is observable in the bottom
flange, and also, to a lesser extent, at the web and in the upper flange.

The research and measurement results concerning the upper flange are quite compatible, which cannot be said about the bottom flange, where the results differ considerably. Analysing the picture of girder pattern cracks (Fig. 3) it can be seen that both the bottom flange and the web are very seriously cracked. The cracks are close to each other and their widths is considerable. The upper flange is not cracked. As it was presented above, the cracks were injected before the process of prestressing. The measuring points were regularly placed, every 200 mm, measuring bases led through the injected cracks. All visible cracks, wider than 0.1 mm were injected. Smaller non-injected cracks could appear. What is more, nothing is known about the penetration of epoxy resin inside a given crack.

The geodesic girder “L” deflection measurement was carried out during the prestressing process. Girder cambering in the middle span was 13.6 mm.

4 CONCLUSION

The difference between measured horizontal strain values and the results obtained from the engineering computer programme was stated. The strain values (converted to stress) obtained during the concrete prestressing process taking place on cracked areas define the extent of base points displacement being the result of not subjecting the other cracks, narrower than 0.1 mm, to injection. That is why it is very difficult to draw conclusions about the value of occurring stress in the bottom flange on the basis of the measurement. The values obtained should be reduced by shortening deriving from crack tightening. Additionally, one has to bear in mind the fact that the material structure does not allow complete tightening of non-injected cracks.

To estimate the quality of the strengthening process, prestressing force in the girder has to be measured, not taking into account only the lengthening of the tendon and tension jack measurements. Geodesic methods should be implemented to control the deflection of prestressed elements. Only complete research makes it possible to define the actual increase of concrete strains, and what follows, define the forces introduced into the structure, which makes it possible to talk about the effectiveness of the strengthening.