Experimental results of steel truss nodes strengthen by high strength concrete encasement

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Abstract. In the old steel truss bridges one particular point of interest are the nodes. While for strengthening/refurbishment of the elements of the truss engineers can rely on more conventional methods, usually, for the nodes unique solutions have to be adopted. One integral solution, which could be implemented in a variety of steel truss nodes, is strengthening/refurbishment by totally encasing the node in high performance fibres reinforced concrete. In order to validate this idea laboratory testing of steel truss nodes strengthened by encasement with different concrete classes was initiated. The connection between the concrete and the structural steel is ensured by application of epoxy resin, no mechanical elements are added. The results of the strengthened specimens are compared with one reference bare steel node test. The strengthened specimens show reduced values of the stresses in the gusset plate as well as increase in the load bearing capacity of the connection of the elements entering the node. These results are observed even for encasement in normal class concrete of C40/50.

1. Introduction

The idea of strengthening steel bridges (structures) with the help of concrete, in order to take advantage of the joint work of the two materials in one united composite section is not new [1]. Perhaps it is not so wildly spread because of the additional self-weight of the concrete, its low tensile strength and the possible durability and maintenance problems. In the recent decades with the development of the new materials like ultra-high performance concrete the above mentioned restrictions have been overcome. Examples for strengthening steel bridges with ultra-high performance concrete are recorded under [2], [3] and [4].

Most of the cited examples are related to orthotropic steel bridge decks. Until now no evidence was found by the authors for the implementation of this technic to old steel trusses and more specifically to their nodes, which are of particular interest to the structural engineers. While for strengthening the elements of the truss, design engineers can rely on more conventional methods, usually, for the nodes unique solutions have to be adopted [5].

The main problems found in the truss nodes are related with:

- Gusset plates - too thin or corroded gusset plates, where local buckling or shear tearing might govern the design;
- Connecting rivets – strengthening the connection of the elements entering the node might be restricted by the possibilities to add additional bolts or rivets, because of the small space available between the existing connectors and the optimised geometry of the gusset plates.

One integral solution to the above problems, which could be implemented in a variety of steel truss nodes, is strengthening by totally encasing the node in high performance concrete (HPC). The authors believe that this solution will not only lead to higher load bearing capacity of the node, but will also improve its fatigue behavior by reducing the stresses in the gusset plate and connecting elements from live loads. Other benefits as higher durability and reduced maintenance costs are also expected.

In order to prove the efficiency of the proposed strengthening method an experimental program has been developed under the research project PROLIFE. Five full scale steel truss nodes were tested: one bare steel reference node, one strengthened by normal class concrete and three strengthened by High-Strength Fiber Reinforced Concrete (HSFRC). In order to ease the encasement process no additional conventional reinforcement was envisaged and the connection between structural steel and the concrete is realized by epoxy resin only without any shear connectors.

2. Description of the specimens
The truss nodes chosen for the experiments are full scale having the geometry shown in Figure 1. In order to make it possible to test the nodes in the laboratory i.e. apply load and add supports, it was decided to incorporate them into truss girder.

![Figure 1. Geometry of the test specimen (left) and the steel node (right).](image)

Unlike the original riveted truss nodes, the connector used in the specimens are fitted bolts M16 (placed in holes $\Phi 16.5\text{mm}$). This was accepted in order to ease the production as rivets are not widely produced nowadays. All other nodes of the truss are designed as welded having higher load bearing capacity than the one under consideration.

Except the reference bare steel test four more tests are performed, where the node under consideration is encased in concrete (Figure 2).

The chosen parameters to be varied for the strengthened specimens are: concrete class, concrete cover and encasement length of the diagonal truss elements. Two of the specimens have concrete cover of 30mm and the other two of 50mm. The encasement length varies between 140mm and 210mm and the different concrete strengths are listed in Table 1.

According to the initial design of the specimens, the tuned failure mode is shear of the two fitted bolts connecting the diagonal members. This failure mode is later proven by the tests.
Figure 2. Geometry of the encased nodes with concrete cover c=50mm.

Table 1. Variable parameters in the specimens.

| Specimen | Concrete cover c [mm] | Encasement Length L [mm] | Concrete f\textsubscript{cm,cube} [MPa] | f\textsubscript{ck} [MPa] | f\textsubscript{ctm} [MPa] | Class |
|----------|------------------------|--------------------------|----------------------------------------|----------------|-----------------|-------|
| No1 - RN | -                      | -                        | -                                      | -              | -               | -     |
| No2 - C50L140 | 50              | 140                      | 57.6                                   | 40             | 3.4             | C40/50 |
| No3 - C50L210 | 50              | 210                      | 71.4                                   | 55             | 3.8             | C55/67 |
| No4 - C30L140 | 30              | 140                      | 102.1                                  | 80             | 11.0            | C80/95 |
| No5 - C30L210 | 30              | 140                      | 102.1                                  | 80             | 11.0            | C80/95 |

All specimens are loaded with static monotonic loading with a load-displacement protocol of 0.01mm/s except the last one, in which cyclic loading to verify the bonding between the two materials has been applied.

3. Measuring Equipment

During each test, the displacements are recorded with minimum of 7 displacement transducers (LVDT’s), additional transducers were installed in two specimens to measure the slip between the two materials (Figure 3).

Figure 3. Position of the displacement transducers (LVDT’s).
Linear strain gauges (TML FLA-6-11) are placed in the mid-cross section (outside the encased node) of each truss element in order to catch the overall stress state of the truss during loading.

![Diagram showing position of rosettes and strain gauges at the node.](image)

**Figure 4.** Position of the rosettes and strain gauges at the node.

In order to measure the stresses into the steel gusset plate two rosettes are placed at the end point of the diagonals entering the node. Their position was determined after linear elastic analysis performed over initial FEM of the bare steel specimen. Strain gauges were also installed over the concrete after the encasement (Figure 4).

In Figure 5 the reference specimen as well as one encased (strengthened) specimen are shown prior to start the testing. The out of plane restrains could also be seen on the pictures (with green color).

![Specimen prior to testing, reference (left) and strengthened (right) one.](image)

**Figure 5.** Specimen prior to testing, reference (left) and strengthened (right) one.

### 4. Materials properties

The nominal class of the structural steel for the profiles of the specimens is S275. Three coupon tests were taken from unstressed pieces in order to find the real mechanical properties of the steel. The mean yielding strength obtained by the tests is 329 MPa, which is around 19% higher than the nominal value. The tests were performed in accordance with EN ISO 6892-1 using a Multipurpose Servo hydraulic Universal Testing Machine rated to 200kN.

![Steel coupons test specimens.](image)

**Figure 6.** Steel coupons test specimens.
The concrete properties were tested with cube specimens sized 150x150x150mm after each casting. The compressive tests were performed according to EN 12390 with 8 concrete cubes in total, two are tested at age of 3 days, three are tested at age of 14 days and the remaining three at age of 28 days.

The tensile strength was obtained in an indirect way through the application of a compressive load on a small area along the length of cube until it splits into two halves along a vertical plane due to indirect tensile stress generated by Poisson's effect (Figure 7). Thereby, 3 concrete cubes with 150 mm size were tested at the age of 28 days.

The mean values of the concrete tensile and compressive strengths, for different concrete castings, are presented in Table 1.

![Indirect test to obtain tensile concrete strength.](image)

5. Results from the experiments
Graphical representations of some of the obtained results are presented under this section. The results from the last test specimen are not presented as they were not fully processed to the date of the submission of the article. Face 1 and 2 on the presented graphs are used to denote the two strain gauges situated symmetrically over z axis of one and the same cross section.

In Figure 8 the vertical displacement, measured at the loaded node, in relation to the applied load are plotted.

In Figure 9 the normal stresses in the tension diagonal of the truss are plotted against the applied load.

In Figure 10 the stresses in the gusset plate at 45° angle, found from rosettes R1 (compression side diagonal) and R3 (tension side diagonal) are plotted against the applied load.

![Applied load – Vertical Displacement graphs for LVDT 4.](image)
Figure 9. Applied load – Stress in the tension diagonal.

Figure 10. Applied load – Stress in the gusset plate (Rosettes R1 and R3).

Figure 11. Applied load – Stress in the concrete (Strain gauge C6).
In Figure 11 the stresses in the concrete from strain gauge C6 (compression side) are plotted against the applied load.

6. Summary and conclusions
Failure mode: All specimens failed by shear of the two bolts connecting the tension diagonal to the node, as for the encased nodes it is accompanied with prior cracking of the concrete. This fully matches with the initially expected failure mode from the preliminary calculations;

Debonding: The connection between the concrete and steel is ensured only by the application of epoxy resin. The maximum measured slip between the concrete and the tension diagonal, for both specimens, in which the slip has been recorded, is around 0.50mm. It could be stated that debonding has not occur due to the preceding cracking of the concrete in the tension side of the node;

Breaking load: If we accept failure criteria of 50mm vertical displacement at loaded point, the respective breaking load for the specimens is 356kN, 418kN, 386kN and 423kN.

Stresses in the gusset plate: the stresses obtained from rosettes R1 and R3 located right after the compression and tension diagonals are listed in Figure 10. The average reduction of the stresses at 45° angle, for loading magnitude equal to the theoretical breaking load of the bolts in the reference specimen (~250kN), is 77% for R1 (compression side) and 52% for R3 (tension side). This reduction is confirmed also by the results presented on Figure 11, where the stresses into the concrete are plotted.

In summary it could be stated that the obtained experimental results are quite promising even for concrete classes C40/50 and C55/67. The encasing appears more effective for the gusset plate itself by reducing the stresses and preventing local buckling, rather than the loadbearing capacity of the connecting elements. Nevertheless increase of the breaking load and the overall stiffness is also
observed. Positive effects, which are expected to be even more, pronounce for higher concrete classes and especially for encasing in UHPFRC.

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