# Testing of composite girders with coiled spring pin shear connectors

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## Abstract

Today, steel girder bridges with concrete deck slabs are generally constructed as steel-concrete composite structures, to utilize the material and the structural parts in an efficient way. However, many existing bridges constructed before the early 1980´s were designed without shear connectors at the steel-concrete interface. With increasing traffics loads and higher amount of load cycles, there is sometimes a need to strengthen these bridges. One way to increase the bending moment capacity is to create composite action by post-installation of shear connectors. The authors have studied the concept of strengthening by post-installed shear connectors, with a focus on a connector called coiled spring pin. This paper presents the results from the first beam tests performed with this kind of shear connector. In line with the previous push-out tests, the test results indicate a very ductile shear connection, with a potential to be a material- and cost-efficient strengthening alternative.

**Keywords:** bridge; strengthening; rehabilitation; shear connector; test; composite girder; coiled spring pin

## 1 Introduction

Modern steel-concrete girder bridges are generally designed as composite structures, to utilize the different materials and structural parts in an efficient way. Existing steel-concrete girder bridges in the Nordic countries from the early 1980´s and older were, however, generally designed as non-composite structures without shear connectors transferring a longitudinal shear flow at the steel concrete interface. These bridges are often in good condition and could remain in service for several decades. However, since the traffic loads have been increasing over time and also the number of load cycles from heavy vehicles, the traffic load capacity might be a problem nowadays. If an assessment indicates that the traffic load capacity is too low for future demands, in terms of global bending moment capacity, post-installed shear connectors in one possible strengthening method.

By installing shear connectors in non-composite steel-concrete girders, the traffic loads will act on a composite section instead. Since there are more than 2000 existing bridges of this type only in Sweden, Norway and Finland, there is a potential need for a cost- and material-efficient strengthening method. Especially since the allowed traffic loads have been increased to higher and higher levels in Sweden and Finland in recent years.

Strengthening of non-composite steel-concrete girders has been studied by several researchers, covering different types of shear connectors and installation methods. A research group at the University of Austin at Texas has performed a comprehensive study of different types of post-installed shear connectors, through modified push-out tests, girder tests and pilot test on an existing bridge [1-8]. Example of laboratory tests on post-installed shear connectors performed by other researchers are [9-12], while [13-15] reports from field monitoring and strengthening by post-installed shear connectors.

Different types of shear connectors are more or less suitable for post-installation. The authors of this paper have focused on an interference fit connector called Coiled Spring Pin (CSP), with the major advantage that it can be installed from below the concrete deck without any need of grouting or chemical anchor. The CSP is a standardized product [16] for fastening application, although it is not that common in steel-concrete connections. The connectors are manufactured from a steel plate that is spirally wound 2,25 times around its longitudinal axis, see Figure 1.

During the last decades some research and tests have been performed on CPSs used as shear connectors at steel-concrete connections. However, the majority of the tests have been conducted on push-out test specimen, often designed to be representative for the conditions in a specific project [17-20]. In recent years, the authors of this paper have conducted larger test series on CSPs, aiming to produce more general design guidelines for CPSs used as post-installed shear connectors at steel concrete interfaces. The previously performed tests includes static push-out tests, fatigue tests on push-out specimens and field monitoring on two strengthened bridges [14,15,21,22]. These tests have resulted in recommendations for the static design capacity, the fatigue capacity and also some guidelines for the installation procedure [23]. However, until now there have been no girder tests on CSPs, verifying the results from the push-out tests. This has been identified as a gap of knowledge, since questions have been raised regarding the risk of separation/uplift at the steel-concrete interface and since the results of the push-out tests should be verified also in a beam structure. The tests presented in this paper is believed to be the first beams tests with CPSs as shear connectors and a step further in the development of a general design guidance for post-installed CPS shear connectors.

## 2 Laboratory tests

The test campaign presented in the following sections has been performed on two girder specimens. Both were first loaded as non-composite girders, up to a load level a bit below the yield strength of the steel girders. The first loading stage was used to study the elastic behaviour of a corresponding non-composite girder and to generate a slip at the steel-concrete interface, reducing any kind of bond that could be non-conservative in comparison to an existing structure strengthened this way. After these initial tests, both girders were strengthened by post-installed CSPs and then again tested. First at corresponding load levels as the non-strengthened specimen and then finally loaded towards failure.

### 2.1 Test method, set-up and specimen

A three-point bending test set-up has been used for the tests, see Figure 2. However, different kind of test methods and specimens have been considered prior to the tests. Three-point bending was chosen, ahead of four-point bending, since it is preferable to use only one point of loading in the part of the beam with theoretically no slip (i.e. in the midspan of a simply supported beam). This is explained by the fact that the friction at the steel concrete interface increases if the normal force component increases. A four-point bending test might

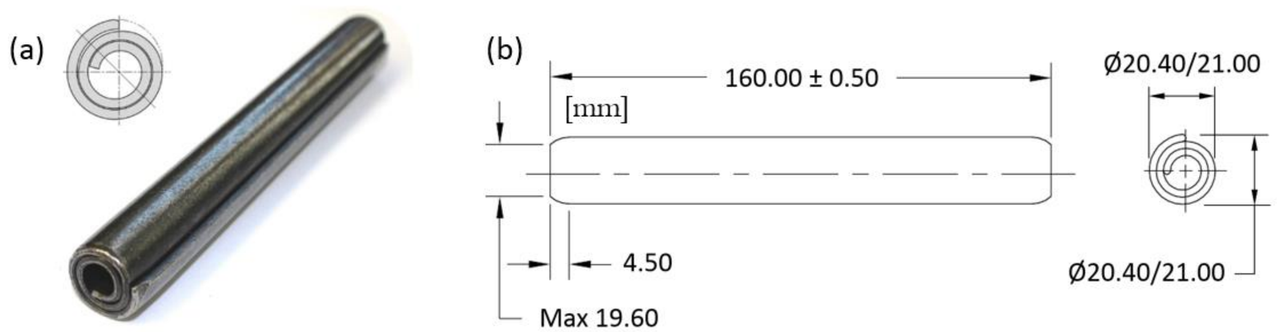



*Figure 1. (a) Coiled Spring Pin, (b) CSP 20x160 dimensions*

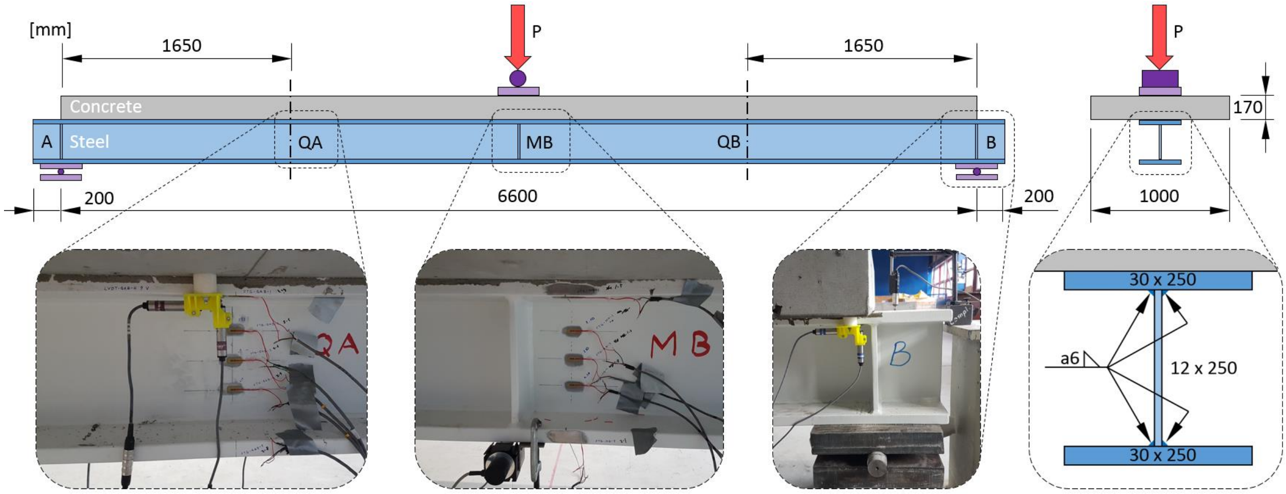



*Figure 2. Schematic illustration of the test set-up and photos of the measurement devices*

introduce a non-conservative frictional shear connection/interlock. The need of a uniform bending moment in the midsection is believed to be less important in these tests, which are focused on the behaviour of the shear connection, not the plastic moment capacity of the cross-section.

### 2.1.1 Test set-up

The test set-up consists of a simply supported girder that is tested in three-point bending, as schematically presented in Figure 2. The load is applied through a steel roller, as a single line load in midspan, and distributed to the concrete through a thick steel plate centred above the steel girder.

Measuring devices were installed at the support sections (A, B), the quarter points (QA, QB) and in midspan (MB), see Figure 2. Figure 3 shows a schematic illustration of the different measurement devices along the girder. In addition to these measurements, the force and the stroke of the hydraulic actuator were registered continuously during all tests.

To study the bending stiffness of the specimen two wire gauges were installed in midspan (MB), measuring the vertical displacement. These gages were vertically connected between the floor and each side of the steel bottom flange. In order to be able to adjust the deflection for the support deformations, vertical displacements at the supports (A, B) were also measured by linear variable differential transformers (LVDTs).

In order to register the relative displacement at the steel-concrete interface, LVDTs were used to measure the horizontal slip and the vertical uplift. This was done in four section (A, QA, QB and B). The LVDTs can be seen in the pictures in Figure 2.

To be able to study the degree of composite action in the girders, several strain gauges were mounted on the steel girders. The general positions of the strain gages are shown in Figure 3. However, the number of strain gauges was reduced in some sections.

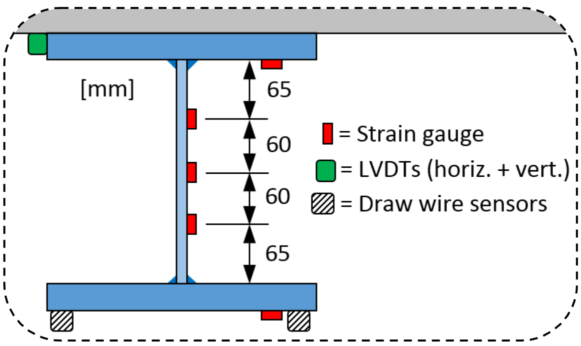



*Figure 3. Position of the measurement devices*

### 2.1.2 Test specimens

The test specimens consist of welded steel girders (S355) with reinforced concrete slabs on top (C35/45). The dimensions of the steel and concrete parts are presented in Figure 2 and the detailed material parameters in section 2.2. After an initial test of the stiffness of the non-composite girder, heavy duty CPSs with a diameter of 20 mm were installed.

In the design of the test specimen, the maximum load (1,0 MN) of the available hydraulic actuator were limiting the dimensions of the test girders.

The steel girders were designed with a length and a bending stiffness low enough to ensure that slips of a few millimetres would occur when the non-composite girders were loaded. The girders were produced in a steel workshop and ordered in steel grade S355.

The thickness of the concrete slab was primarily designed thick enough to ensure that the studied type of shear connectors, with a length of 160 mm, could be installed in a similar way as in a real structure. The maximum width of the concrete slab, made in C35/45, was limited to ensure that the composite girder could be loaded to failure at a load level < 1,0 MN. The minimum width was limited by the will of using a cross-section with a similar reinforcement layout as in a real bridge structure. The concrete slab was manufactured in the test laboratory, including the formwork, reinforcement work and the casting of the concrete. In order to avoid shear forces at the steel-concrete interface, due to the weight of the concrete. The concrete slabs were cast while the steel girders were simply supported in the ends, at the position A and B in Figure 2. The load from the formwork and the wet concrete was carried by the similar static system as later tested. Figure 4 shows one of the specimens before the concrete was cast.

It would have been preferable to use higher steel girders in the tests, to get a bending stiffness ratio between the steel and the concrete that was more similar to existing bridge girders. However, as mentioned above the hydraulic actuator and the desired structural behaviour limited the height of the girder.

*Figure 4. Photo of one of the specimens, before the concrete was cast*

Because of the limited space between the steel flanges an alternative installation method had to be used, in comparison to the method recommended by the authors in [23]. In the general case the drilling is done from below the steel girder, see Figure 5a. For the test specimens the drilling was performed from above, see Figure 5b, since there was no drilling equipment small enough to fit between the flanges and with a stroke large enough to drill 160 mm. A diamond core drill with a diameter of 19,95 mm was used to drill through the concrete until reaching the upper side of the steel top flange. Then the drill was changed to a cutting steel drill with similar dimensions. The given tolerances of the holes were in line with the recommendations in [23], i.e. 19,85 - 20,25 mm and was checked for each hole after drilling. The majority of the measured hole diameters were within the given tolerances. After the holes were drilled, the CSPs were installed by using a jack and

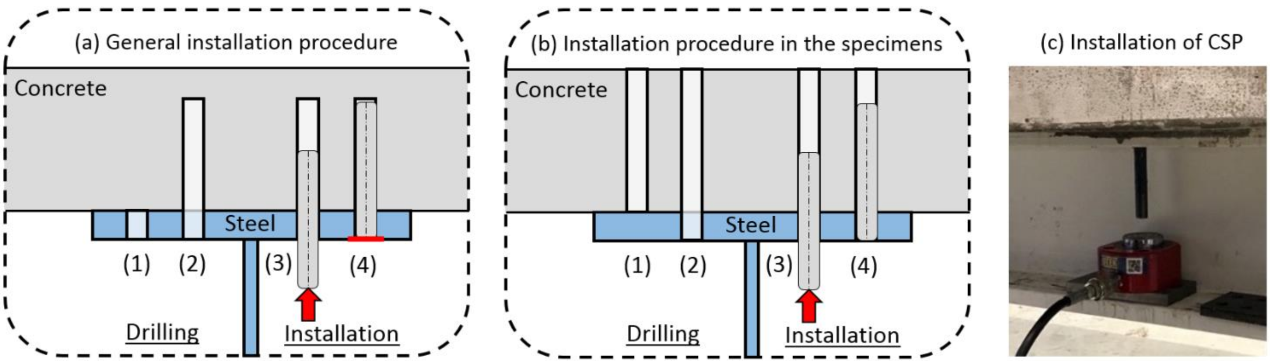



*Figure 5. Installation of CSPs (a) in general, (b) in the specimen, (c) photo of a CSP and the jack*

pushing the oversized pins into the undersized holes, see Figure 5c.

In the planning of the tests it was decided to use 24 CSPs in each specimen. This is far below the number that corresponds to full composite action and was chosen to assure that the shear connection would not have a far higher load capacity than other parts. The positions of the CSPs is schematic illustrated in Figure 6, together with a schematic layout of the transversal reinforcement bars, which governs the positions of the CPSs. As shown in Figure 6, the positions of the CSPs was altered between girder 1 and 2, to get an indication of the importance of the distribution of the shear connectors.

## 2.2 Material properties

The steel girder was ordered in steel grade S355J2 but delivered in grade S355K2C+N, in line with the steel standard SS-EN 10025-2:2004. The tested material properties are presented in Table 1.

*Table 1. Steel properties*

| Part | Grade | $R_{eH}$ [MPa] | $R_m$ [MPa] | Elong. [%] |
|---|---|---|---|---|
| Web | S355K2C+N | 447 | 579 | 29 |
| Flanges | S355J2 | 407 | 541 | 27 |
| CSP | AISI 6150 | 1516 | 1606 | 5 |

The CSPs were of type ISO 8748 – 20x160 – HWK, which means that the pins are of type Heavy Duty (H) and made out of AISI 6150 Alloy Steel (W) with no surface treatment (K). The digits represent the nominal outer diameter of 20 mm and the length of 160 mm (see Figure 1). The thickness of the steel plates used to manufacture this type of pins are 2,2 mm. The manufacturer of the pin reported a yield stress of 550 MPa and a tensile strength 690 MPa in the base material. To establish the material properties for the final pin, four test coupons were taken from the pins in the ordered batch, and tensile tests were undertaken according to EN ISO 6892-1. The tests showed that the mean value of the yield stress and the tensile strength were 1516 MPa and 1606 MPa, respectively, and that failure occurred at an elongation of about 5%.

The reinforcement bars were of grade B500B. No material tests were conducted on the bars used in the slab.

The concrete was ordered in concrete class C35/45 and cubes were cast at the same time as the test specimens to investigate the compressive and the tensile strength of the concrete. The cubes were cured in the same environment as the specimens and tested at the same age as the specimens. The cube tests were conducted in accordance with SS-EN 12390-3 and SS-EN 12390-6.

Table 2 shows some of the important concrete parameters.

*Table 2. Concrete properties*

| C35/45 | Girder 1 | Girder 2 |
|---|---|---|
| $f_{ccm}$ [MPa] | 54,8 | 51,5 |
| $f_{ctm,sp}$ [MPa] | 4,4 | 4,3 |
| $E_{cm}$ [GPa] | 34,3 | 33,6 |

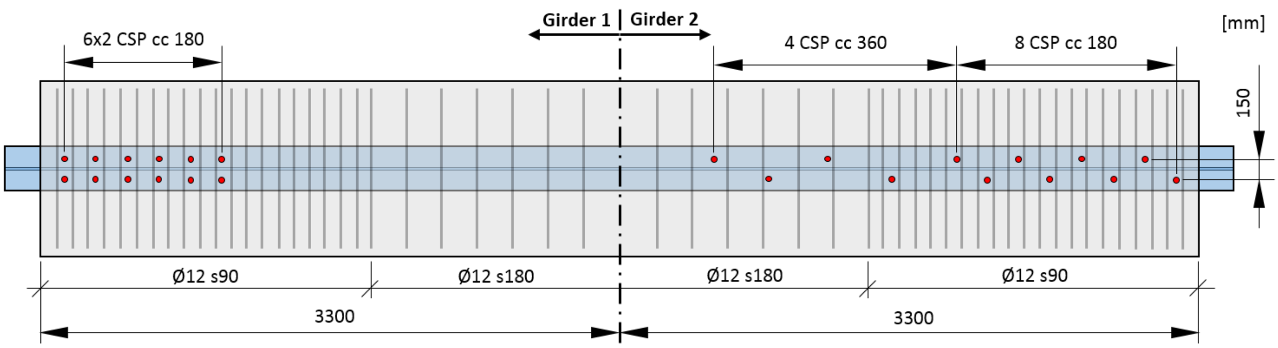



*Figure 6. CSP postions and transversal reinforcement layout*

The mean values of the secant modulus of elasticity have been estimated in line with equation (1) from [24], where $f_{cm}$ is the compressive cylinder strength given in MPa. The concrete cube strength has been transformed into the cylinder strength by multiplying with factor 0,8.

$$E_{cm} = 22 \cdot (f_{cm}/10)^{0.3} \quad (1)$$

## 2.3 Test procedure

The test procedure involved five load stages. The first three load stages were conducted on the non-composite girders, while the last two stages were done on the strengthened composite girder.

In order to break any existing bond at the steel-concrete interface, the non-composite girders were loaded and unloaded in three sequences with P = 0-220 kN, 0-250 kN and 0-350 kN. An additional test with P = 0-420 kN was also done on Girder 2. After these stages, the girders were strengthened with CSPs and the last two load stages were performed. The fourth load stage was performed on the same load level as stage three (P = 0-350 kN) in order to get comparable results for the composite respectively the non-composite cross-section. In the fifth and last load stage it was aimed to load the girders to a final failure or until the tests had to be interrupted due to limitations in the test set-up. Possible limitations might be the stroke length of the hydraulic actuator, load capacity of the hydraulic actuator, very large rotations at the supports and very deformations in midspan limited downwards by the floor in the test-lab.

In all test stages, the loading was displacement controlled with a rate of 0,1 mm/s.

# 3 Test results and analysis

In this paper, the first test results from the tests are presented together with a brief analysis focusing on the bending stiffness of the tested girders and the stress levels.

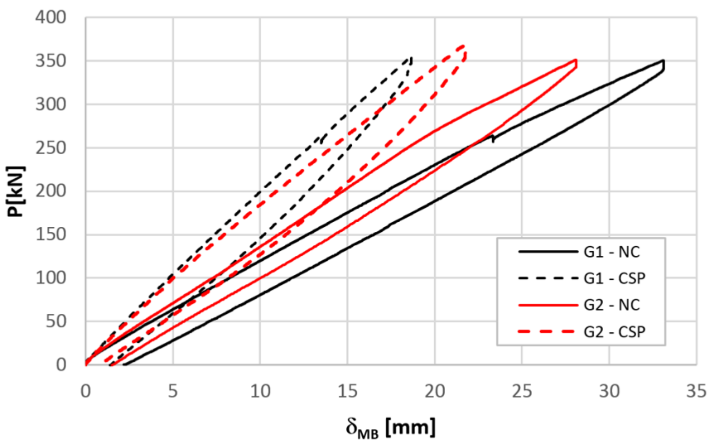
## 3.1 Girders loaded to 350 kN

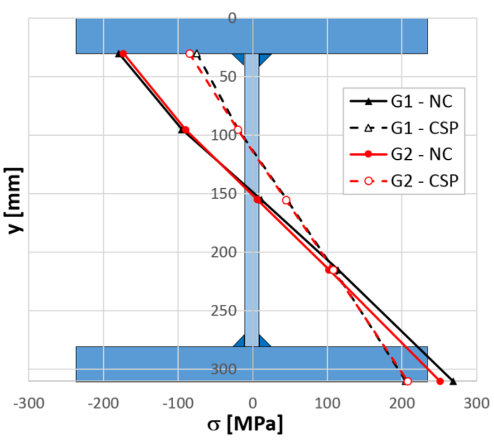
In this section the two test specimens are denoted G1 and G2 (Girder 1 and 2). The following notations, NC and CSP, indicates if the specimens are tested with or without any shear connectors (NC = No composite action, CSP = composite action by CSPs).

The load-deflection curves (P-$\delta$) for the four corresponding tests, up to 350 kN, are shown in Figure 7.

As shown in Figure 7, some of the tests were run slightly higher than 350 kN. However, when the steel stresses are compared the values are taken at 350 kN and from the loading sequence in each test series (not from the unloading part of the curves). Figure 8 illustrates the stress levels in the steel girders at different heights. The measured steel strains have been converted to stresses by Hooke's law, assuming a fully elastic steel material and an elastic modulus of 210 GPa.

The tests results indicate that the non-strengthened specimens, after the first bond breaking load stage, behave as non-composite sections with neutral bending axes in the middle of the double symmetric steel girder. The bending stiffness during the loading sequence indicate a stiffness that are close to, or slightly higher, than the stiffness of the steel girder itself. This is



expected since the bending stiffness of the concrete deck slab itself gives an additional contribution to the total bending stiffness.

The results also show that the contribution from the concrete deck is increased, after the installation of CSPs in the specimens, and that the neutral bending axis is shifted upwards. The measured stress level in the bottom flange is decreased by 17,5-23,6 %. The bending stiffness of the partial composite section corresponds to a stiffness in the middle between a full composite section and a non-composite section. This is also in line with the expectations since the number of installed CSPs corresponds to a moderate level of partial composite action.

### 3.2 Girders loaded towards failure

In the final test stage, the strengthened girders were loaded towards failure. Figure 9 shows the load-deflection diagrams for the two girders.

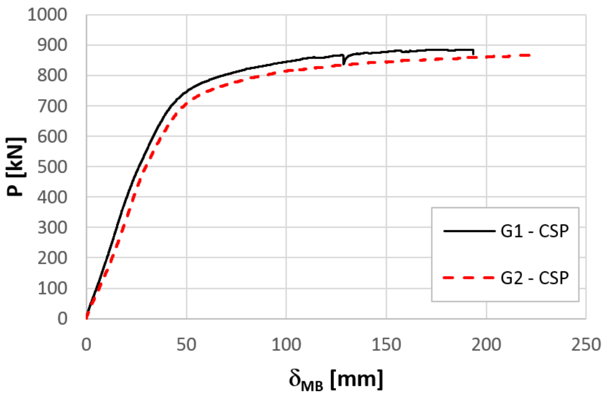



*Figure 9. Load-deflection curves for P = 0 - Max*

It can be noted that both tests were terminated before a final failure in the structural components. There were several reasons behind the decision to stop the test at these levels of large deformations, e.g. the risk that the specimen fall down from the supports, the limited distance down to the floor, the risk of damaging measurement devices located at the bottom of the test beam etc.

It can also be noted that the draw wire gauges, that were used to measure the midspan deflection, run out of stroke at 856 kN and 844 kN for G1 and G2, respectively. The load-deflection curves above these load levels are based on the stroke measured directly at the hydraulic actuator. This should not affect the results significantly, since the maximum load for G1 and G2 were 885 kN and 867 kN, respectively. Implying that the additional load after the run-out of the draw wire gauges is quite low, which implies that additional deformations in the test set-up should also be quite low.

The test results show that the installed CPSs can ensure a ductile shear connection that enables large plastic deformations of the partial-composite section without failure in the shear connection.

## 4 Discussion, conclusions and further research

In this paper, the first results and analysis from two girder tests are presented. These results indicate that the post-installed CPSs can be used to strengthen a non-composite steel-concrete girder, in terms of bending stiffness and lowered steel stresses. The test results also indicate a very ductile behaviour of the girder, with large plastic deformations prior to the failure, without any observed failure in the shear connectors.

The test results will be further analysed and compared to calculations models and FE-models. The results of the upcoming work will be reported in a more comprehensive report or paper.

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## 5 References

[1] Schaap B. A. *Methods to develop composite action in non-composite bridge floor systems: Part I*, M.Sc. Thesis, The University of Texas at Austin, USA; 2004

[2] Hungerford B. E. *Methods to develop composite action in non-composite bridge floor systems: Part II*, M.Sc. Thesis, The University of Texas at Austin, USA; 2004.

[3] Kayir H. *Methods to develop composite action in non-composite bridge floor systems: Fatigue behavior of post-installed*

*shear connectors*, M.Sc. Thesis, The University of Texas at Austin, USA; 2006

[4] Kwon G., Engelhardt M. D. & Klingner R. E. Behavior of post-installed shear connectors under static and fatigue loading. *Journal of Constructional Steel Research*. 2010; **66**(4): 532-541.

[5] Kwon G., Engelhardt M. D. & Klingner R. E. Experimental behavior of bridge beams retrofitted with postinstalled shear connectors. *Journal of Bridge Engineering*. 2010; **16**(4): 536-545.

[6] Kwon G., Engelhardt M. D. & Klingner R. E. Parametric studies and preliminary design recommendations on the use of postinstalled shear connectors for strengthening noncomposite steel bridges. *Journal of Bridge Engineering*. 2012; **17**(2): 310-317.

[7] Kwon G., Engelhardt M. D. & Klingner R. E. Strengthening bridges by developing composite action in existing non-composite bridge girders, *Structural Engineering International*. 2009; **19**(4): 432-437.

[8] Kreitman K., Ghiami Azad A. R., Engelhardt M. D., Helwig T. & Williamson E. Shakedown Behavior of a Continuous Steel Bridge Girder Strengthened With Post-Installed Shear Connectors, *Structures.* 2016; **8**(2): 245-251.

[9] Pathirana S. W., Uy B., Mirza O. & Zhu, X. Strengthening of existing composite steel-concrete beams utilising bolted shear connectors and welded studs. *Journal of constructional steel research*. 2015; **114**, 417-430.

[10] Pathirana S. W., Uy B., Mirza O. & Zhu X. Flexural behaviour of composite steel–concrete beams utilising blind bolt shear connectors”. *Engineering Structures*. 2016; **114**, 181-194.

[11] Pathirana S. W., Uy B., Mirza O. & Zhu X. Bolted and welded connectors for the rehabilitation of composite beams”. *Journal of Constructional Steel Research*. 2016; **125**, 61-73.

[12] Henderson I. E. J., Zhu X. Q., Uy B. & Mirza O. Dynamic behaviour of steel-concrete composite beams retrofitted with various bolted shear connectors. *Engineering Structures*. 2017; **131**: 115-135.

[13] Peiris A. & Harik I. Steel bridge girder strengthening using postinstalled shear connectors and UHM CFRP laminates, *Journal of Performance of Constructed Facilities.* 2014; **29**(5).

[14] Hällmark R. & Collin P. Post-installed shear connectors: monitoring a bridge strengthened with coiled spring pins. *Structural Engineering International*. 2019; **29**(2): 225-233.

[15] Arason M., Ragnarsson G., Collin P. & Hällmark R. Strengthening of steel girder bridges using coiled pins. *In proceedings from: 2019 IABSE Congress, 4-6 September* 2019, New York City, New York, USA, **1**, 38-43.

[16] ISO 8748, *Spring-type straight pins – Coiled. heavy duty*. ISO – the International Organization for Standardization; Geneva. Switzerland. 2007.

[17] Pritchard B. *Bridge Design for Economy and Durability: Concepts for New, Strengthened and Replacement Bridges*. Thomas Telford, London, UK; 1992.

[18] Buckby R., Ogle M., Johnson R. P. & Harvey D. The performance of coiled spring pin connectors under static and fatigue loading”. *Proceedings of the International IABSE Conference: Composite construction - conventional and innovative*, Innsbruck, Austria, September; 1997.

[19] Fahleson C. *Laboratory report: Testing of shear connectors, Laboratory Report No: 05035*, Luleå University of Technology, Luleå, Sweden; 2005.

[20] Hällmark R., Jackson P., Collin P. & White H. *Strengthening Bridges with Postinstalled Coiled Spring Pin Shear Connectors: State-of-the-Art Review*, Practice Periodical on

Structural Design and Construction. 2019; **24**(1).

[21] Hällmark R., Collin P. & Hicks J. S. Post-Installed Shear Connectors: Fatigue push-out tests of coiled spring pins, *Journal of Constructional Steel Research*. 2019; **153**: 298-309.

[22] Hällmark R., Collin P. & Hicks J. S. Post-Installed Shear Connectors: Push-out tests of coiled spring pins vs. headed studs, *Journal of Constructional Steel Research*. 2019; **161**, 1-16.

[23] Hällmark R. *Composite Bridges: Innovative ways of achieving composite action*, Doctoral thesis, Luleå University of Technology, Luleå, Sweden, 282 p, 2018.

[24] EN 1992-1-1 Eurocode 2- Design of concrete structures – Part 1-1: General rules and rules for buildings, European Committee for Standardisation; Brussels. Belgium, 2005