Instrumentation and Full-Scale Test of a Post-Tensioned Concrete Bridge




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ABSTRACT

To meet new demands, existing bridges might be in need for repair, upgrading or replacement. To assist such efforts a 55-year-old post-tensioned concrete bridge has been comprehensively tested to calibrate methods for assessing bridges more robustly. The programme included strengthening, with two systems based on carbon fibre reinforced polymers (CFRPs), failure loading of the bridge's girders and slab, and determination of post-tension cables' condition and the material behaviour. The complete test programme and related instrumentation are summarised, and some general results are presented. The measurements address several current uncertainties, thereby providing foundations for both assessing existing bridges' condition more accurately and future research.

Key words: Assessment, bridges, carbon fibre reinforced polymer, concrete, destructive test, ductility, flexure, full-scale test, monitoring, near-surface mounted reinforcement, non-destructive test, prestressed laminates, post-tension, punching, robustness, shear, strengthening, structural behaviour.

1 INTRODUCTION

In order to meet current and future demands for sustainability and structural resistance, existing bridges might be in need for repair, upgrading or replacement. For instance, responses to a questionnaire by infrastructure managers in 12 European countries, acquired and analysed in the *MAINLINE* project, indicated a need for strengthening 1500 bridges, replacing 4500 bridges and replacing 3000 bridge decks in Europe during the coming decade [1]. The Swedish Government Proposal 2012/13:25 recommended an investment of SEK 522 billion (EUR 60.4 billion) from 2014 to 2025, to meet transport infrastructure requirements in Sweden [2]. With adjustment for inflation this represents a 20 % increase relative to the previous investment level, as detailed in Prop. 2008/09:35 [3], indicating a need for substantial actions to maintain robust and sustainable infrastructure. Due to budgetary constraints and the major social, economic and environmental benefits of avoiding demolition and reconstructing existing bridges [4], they should be repaired and strengthened rather than replaced in cases where this is cost-effectively feasible [5]. Thus, advanced methods should be used for accurately assessing bridges' condition [6], and identifying the optimal operations to maintain, strengthen or replace them, from a perspective based on life-cycle cost minimisation [7].

To obtain reliable assessments of existing bridges, which are crucial for rigorous life-cycle cost analysis, it is essential to address current uncertainties regarding key variables, such as structural and loading parameters and possible deterioration mechanisms [8]. In the past decade monitoring concepts have been developed to update models for bridge assessment, reducing the uncertainties, based on empirical data [9]. Moreover, proof loading has been suggested [10], and subsequently implemented for reinforced concrete structures in ACI Standard 437.2-13 [11], as an approach to verify the reliability of relevant models and reduce uncertainties regarding the true condition of existing bridges. Thus, testing and monitoring of bridges at service-load levels is an accepted and well-known approach for assessment.

Detailed, large-scale laboratory tests of bridges and their materials have been reported, e.g. [12] and [13]. Destructive investigations of prestressed concrete [14], post-tensioned concrete [15-16] and non-prestressed reinforced concrete bridges [17-22] have also been described. However, such studies have generally focused on specific components or elements, for instance, the bridge slab [15]. Few complete full-scale bridges have been tested to failure in order to improve understanding of their true structural behaviour, and rigorously calibrate methods and models. Hence, more comprehensive empirical information on the behaviour of concrete bridges, especially of prestressed and post-tensioned concrete, as they approach failure, and cost-effective methods to avoid risks of failure, is required.

Thus, in the study presented here a 55-year-old post-tensioned concrete bridge was thoroughly instrumented (with up to 141 sensors) and tested to failure. The aims were to calibrate and refine methods and models for assessing existing reinforced concrete bridges, and to assess the utility of methods using carbon fibre reinforced polymers (CFRPs) for upgrading reinforced concrete structures [23]. Since there have been few full-scale tests on post-tensioned bridges, a particular focus was on assessment of the post-tensioned system. The complete test and measuring programme is described here, and selected general results to provide insights about the tests. More detailed results will be presented later.

2 THE KIRUNA BRIDGE

2.1 General description

The Kiruna Bridge, located in Kiruna, Sweden, was a viaduct across the European route E10 and the railway yard close to the town's central station (Figure 1). It was constructed in 1959 as part of the road connecting the city centre and the mining area owned by LKAB. The sub-level caving method for extracting the ore causes subsidence. Thus to ensure the continuing utility of the Kiruna Bridge, in 2006 LKAB initiated geodetic position measurements of the bridge supports. In 2008 Luleå University of Technology (LTU) started to monitor the bridge continuously [24]. Due to ongoing subsidence, LKAB decided to permanently close the bridge in October 2013 for demolition in September 2014, providing an opportunity for LTU to test it to failure in May-August 2014.

Figure 1 – Photograph of the Kiruna Bridge from the north-east, showing the slag heap from the LKAB iron ore mine in the background (2014-06-25).

2.2 Geometry

The bridge was a 121.5 m continuous post-tensioned concrete girder bridge with five spans: 18.00, 20.50, 29.35, 27.15 and 26.50 m long (Figure 2). According to construction drawings both the longitudinal girders and bridge slab in the western part (84.2 m) were supposed to be curved with a radius of 500 m. However, inspection of the actual geometry showed that the slab's girders consisted of straight segments with discontinuities at the supports. Moreover, there were 5.0 % and 2.5 % inclinations in the longitudinal and transverse directions, respectively.

Longitudinal movements of the bridge were allowed at the eastern abutment by three rolling bearings (support 6 in Figure 2), but not the western abutment (support 1). Devices were installed at the bases of the intermediate supports 2-5, each consisting of three columns, in 2010 to enable vertical adjustment of the supports to counter uneven settlement of the basements.

The superstructure consisted of three parallel, 1923 mm in height, longitudinal girders connected with a slab on top (Figure 3). Including the edge beams the cross-section was 15.60 m

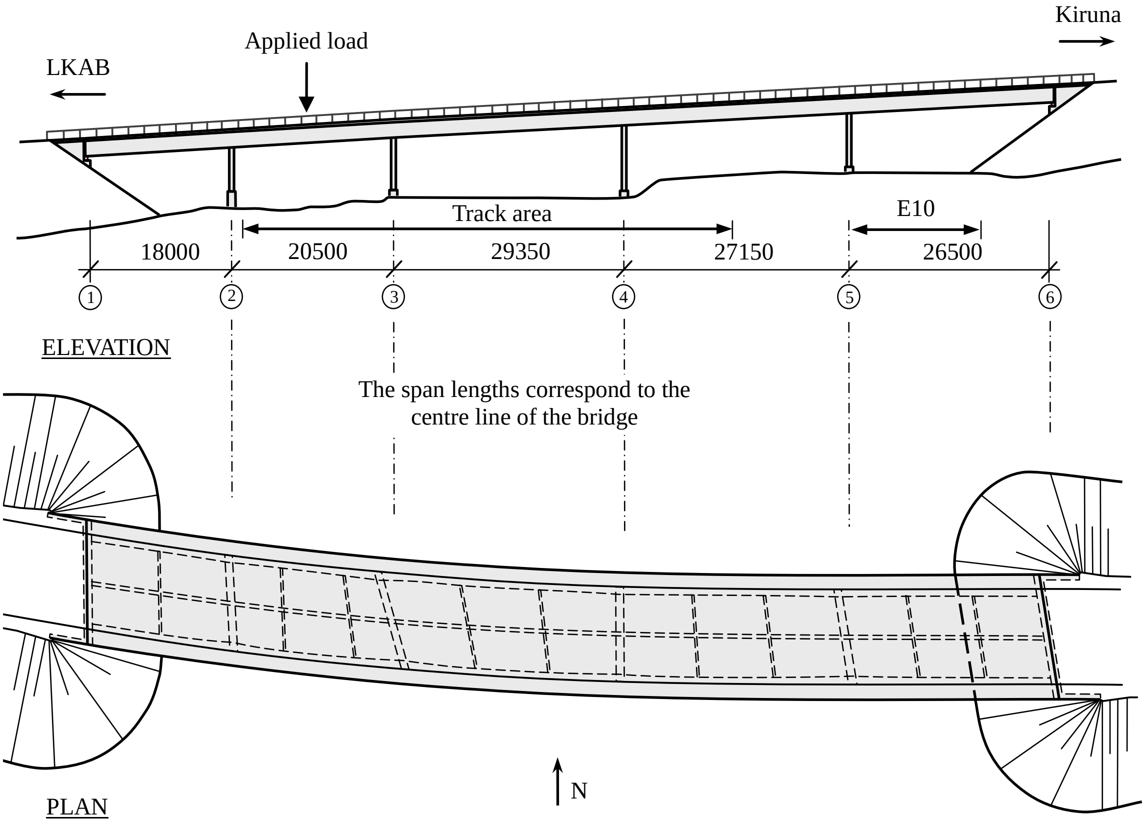

Figure 2 – Geometry of the Kiruna Bridge and location of the load application in the test programme.

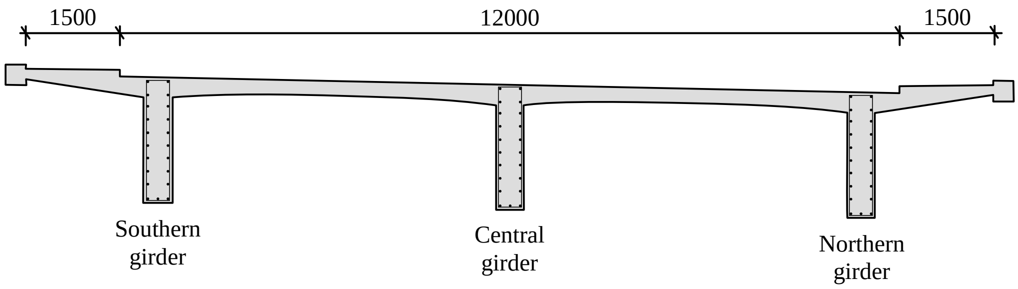

Figure 3 – Cross-section of the Kiruna Bridge.

wide, and the free distance between the girders was 5.00 m. In the spans the girders were 410 mm wide, gradually increasing to 650 mm 4.00 m from the intermediate supports and widened to 550 mm at anchorage locations of the post-tensioned cables, two fifths of the span lengths west of support 3 and three tenths of the span length east of support 4. The bridge slab was 300 mm thick at the girder-slab intersection and 220 mm 1.00 m beside to the girders.

The Kiruna Bridge was post-tensioned in two stages with the BBRV system. In the first stage, six cables per girder were post-tensioned in each end of the central segment. In the second stage, four and six cables per girder were post-tensioned from the free end of the western and eastern segments, respectively. Each cable was composed of 32 wires with a 6 mm diameter.

The girders were each reinforced with three 16 mm diameter bars at the bottom, and 10 mm diameter bars at the sides with either 150 mm spacing for the central girder or 200 mm for the others. The vertical reinforcement also consisted of 10 mm diameter steel bars with 150 mm spacing. The concrete cover was 30 mm thick, except for the 16 mm diameter reinforcement bars, for which the horizontal concrete cover was 32 mm thick.

Before the tests the pavement on the slab was removed from the road crossing the bridge. The bridge was originally designed according to *Provisional Regulations of the Royal Civil Engineering Board* issued in 1955 [25].

2.3 Material

According to construction drawings the concrete quality in the substructure and the superstructure was K 300 and K 400, respectively, while the reinforcing steel quality was generally Ks 40, except in the bridge slab (Ks 60). The steel quality for the post-tensioned reinforcing BBRV reinforcing system was denoted St 145/170. The bridge was constructed in accordance with the *National Steel Regulation* [26] and *National Concrete Regulation* [27], issued in 1938 and 1949, respectively.

3 TEST PROCEDURE

3.1 General description

An experimental programme was designed to assess the behaviour and load-carrying capacity of the bridge using both non-destructive and destructive test procedures. For safety reasons, related to continuing use of the European route E10 during the tests, the experimental programme was developed for loading in span 2-3, with associated monitoring in spans 1-4 (Figure 2). The experimental programme can be summarised by the following, chronological steps:

1. Non-destructive determination of residual post-tensioned forces in cables in span 2-3 (May 27-28, 2014).
2. Preloading Test Schedule 1, of unstrengthened bridge girders, including destructive determination of residual post-tensioned forces in cables in span 2-3 (June 15-16, 2014).
3. Preloading Test Schedule 2, of strengthened bridge girders (June 25, 2014).
4. Failure test of the bridge girders (June 26, 2014).
5. Failure test of the bridge slab (June 27, 2014).
6. Complementary non-destructive determination of residual post-tensioned forces in cables in midspans 1-4 (June 27 and August 25, 2014).
7. Material tests of concrete, reinforcing steel and post-tensioned steel.
8. Condition assessment of post-tensioned cables.

Steps 1-6 were carried out at the Kiruna Bridge, with the test dates in parenthesis. However, steps 7-8 are planned to take place in the Complab laboratory at LTU after demolition of the bridge.

3.2 Strengthening

The experimental programme included tests of two separate systems for strengthening concrete structures using carbon fibre reinforcing polymers, which were attached to the lower sides of the central and southern girders in span 2-3 (see Figure 4 and Figure 10). However, the northern girder remained unstrengthened.

A system of three near-surface mounted (NSM) 10x10 mm^2 CFRP rods was installed in the concrete cover of the central girder [6, 28]. The bar lengths were limited to 10.00 m, due to transportation constraints, thus several overlaps (1.00 m) were required to apply the strengthening over the entire span length. A set of full-length CFRP rods was installed centrically in span 2-3, with sets of 5.80 and 5.74 m CFRP rods on the western and eastern sides, respectively

To strengthen the southern girder, a system of three 1.4x80 mm^2 prestressed CFRP laminates was applied to the blasted concrete surface [29-30]. The lengths of the middle and outer laminates were 14.17 and 18.91 m, respectively, in order to provide space for the anchorage device at each end. Each laminate was tensioned to 100 kN at the eastern end, controlled with a load cell, as the force was applied using a manually operated hydraulic jack. The force was gradually transferred to the concrete by the anchorage device. In this manner no force is expected to be transferred at the end, while it is fully transferred after 1.20 m. The anchorage devices were attached to the bridge until disassembly after Preloading Test Schedule 2. This experimental programme was the first reported full-scale installation and test of the strengthening method using prestressed laminates with these innovative anchor devices.

For the CFRP rods and laminates, denoted StoFRP Bar IM 10 C and, StoFRP Plate IM 80 C respectively, the mean modulus of elasticity and tensile strength were 210 GPa (200 GPa) and 3300 MPa (2900 MPa), respectively, with mean values specified in parenthesis. Epoxi StoPox SK41, a commercially available and CE-approved thixotropic epoxy adhesive, was used to bond both strengthening systems.

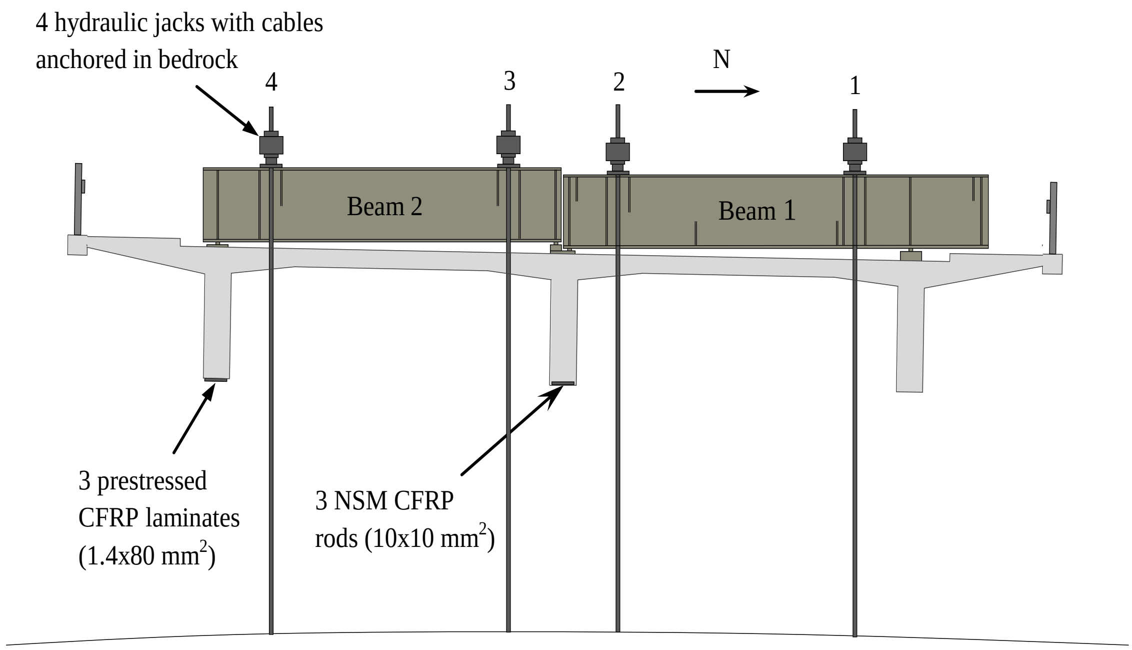

Figure 4 – Arrangement for loading the bridge girders in midspan 2-3.

3.3 Preloading

In span 2-3 two welded steel beams (outer dimensions 700x1180x5660 and 700x1180x6940 mm^3) were arranged horizontally to apply loads in the midspans of each girder, see Figure 2 and Figure 4. They consisted of a double web (thickness 15 mm) with flanges (thickness 30 mm) and were of the steel grade S355J0. The beams were supported by steel load distribution plates (steel grade S275JR), with areas of 700x700 mm^2 and total thicknesses ranging from 20 to 265 mm, due to the inclination of the bridge slab. A horizontal concrete surface was also cast locally under the plates. The bridge was loaded using four hydraulic jacks with cables, threaded through drilled holes in the bridge slab, anchored over a length of 14.60 m in the bedrock, as illustrated in Figure 4. The distances from the centre of the jacks and cables to the centre of the support of the transverse steel beam were 885 mm. The capacity of the jacks was approximately 7.0 MN, with a 150 mm stroke length. The piston cross-section area was 1282 cm^2 for jacks 1, 3 and 4, and 1284 cm^2 for jack 2. Each cable consisted of 31 wires with 15.7 mm diameter.

The bridge was preloaded by applying two schedules of incrementally increasing loads using the four jacks to both strengthened and unstrengthened girders in midspan 2-3, as listed in Table 1 and illustrated in Figure 4. The schedules were designed to reach the cracking load of the girders, as predicted by preliminary nonlinear finite element analysis. Before the force-controlled loading to a specified level, given by the actual load case, the bridge was unloaded. To ensure no drift in the measurements and stable loading, peak pressure was maintained for load cases 7, 9, 13 and 29. Load cases 15-18 in Schedule 1 were designed to determine the remaining forces in the post-tensioned cables (see Section 0).

3.4 Bridge girder failure test

Preloading was followed by a test to failure of the strengthened girders, according to the setup described in the previous section. Each girder was equally loaded to 12.0 MN in total (the approximate load-carrying capacity predicted by preliminary nonlinear finite element analysis): 4.0 MN delivered by the outer jacks and 2.0 MN by the inner jacks. The pressure in jack 4 was subsequently increased to failure of the southern girder and then the pressure in jacks 2 and 3 was increased to failure of the central girder, while the settings of the other jacks remained unchanged so they provided approximately constant loads. The jacks' grip positions were changed as necessary to accommodate deflections exceeding the stroke length.

3.5 Bridge slab failure test

The bridge slab in midspan 2-3 was tested to failure using an arrangement similar to load model 2 (LM 2) described in Eurocode 1 [31], with its centre located 880 mm from the outer side of the northern girder (Figure 5). By rotating steel beam 1 (Figure 4 and Figure 5), hydraulic jack 1 was reused to apply load on the slab, through two 350x600x100 mm^3 steel plates spaced 2.00 m apart. A horizontal concrete surface was also cast locally under the plates. Due to the widening of the bridge girders at the anchorages of the post-tensioned cables, the distances from the centres of the western and eastern load distribution plates to the inner sides of the girders were 470 and 330 mm, respectively. As in the previous tests, the loading was force-controlled.

3.6 Assessment of post-tensioned cables

The residual force in the post-tensioned cables was non-destructively determined by monitoring strains at the lower surface of each girder resulting from gradually cutting the concrete with a saw on both sides of a strain sensor [32] placed one-tenth of the span length west of midspan 2-3, before the bridge and slab failure tests. After the failure tests, the procedure was also applied to each girder in midspan 1-2, the northern girder in midspan 2-3 and the central and southern girders in midspan 3-4. In order to keep the reinforcing steel intact, the arrangements of sensors

Table 1 – Load cases for preloading the unstrengthened and strengthened bridge girders in midspan 2-3.

Load case	Jack 1 kN	Jack 2 kN	Jack 3 kN	Jack 4 kN
1[1,2]	500	250	250	500
2[1,2]	500	500	-	-
3[1,2]	-	-	500	500
4[1,2]	1000	1000	-	-
5[1,2]	-	-	1000	1000
6[1,2]	1500	1500	-	-
7[1,2]	1500	1500	-	-
8[1,2]	-	-	1500	1500
9[1,2]	-	-	1500	1500
10[1,2]	500	250	250	500
11[1,2]	1000	500	500	1000
12[1,2]	1500	750	750	1500
13[1,2]	1500	750	750	1500
14[1,2]	2000	1000	1000	2000
15[1,2]	2000	1000	1000	2000
16[1,2]	2000	1000	1000	2000
17[1]	2000	1000	1000	2000
18[1]	2000	1000	1000	2000
19[1]	500	500	-	-
20[1]	-	-	500	500
21[1]	1000	1000	-	-
22[1]	-	-	1000	1000
23[1]	1500	1500	-	-
24[1]	-	-	1500	1500
25[1]	500	250	250	500
26[1,2]	1000	500	500	1000
27[1,2]	1500	750	750	1500
28[2]	2000	1000	1000	2000
29[2]	2000	1000	1000	2000

[1] Load case for preloading the unstrengthened girder

[2] Load case for preloading the strengthened girder

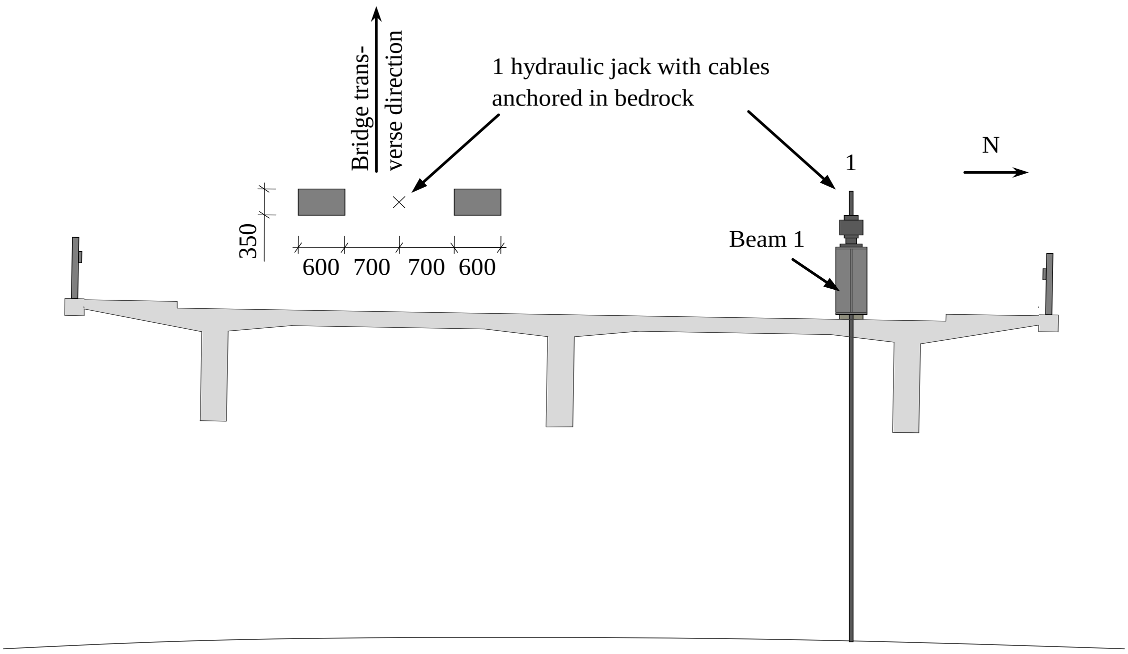

Figure 5 – Arrangement for loading the bridge slab in span 2-3.

and saw cutting lines were designed to avoid cutting either the stirrups or longitudinal reinforcing steel. The cutting proceeded to an approximate depth of 35 mm, or the actual depth of the longitudinal reinforcing steel. All the non-destructive tests were carried out without applying external loads.

As part of Preloading Test Schedule 1, the cracking moment test [33] was applied to calibrate the non-destructive test method. During load cases 1-14 cracks formed, and instruments described in the next section were used to monitor the behaviour of selected cracks and adjacent areas between load cases 14 and 15. Thus, the remaining force in the post-tensioned cables can be determined from data acquired from load cases 15-18, based on the sequence of reopening of the cracks.

Further laboratory assessments of the condition of both the cables and their grouting are planned.

3.7 Material tests

To determine characteristics of the bridge's materials, tests of the concrete, reinforcing steel and post-tensioned steel are also planned. Thus, before the tests described here at least six concrete cylinders were drilled out from the superstructure in both midspans 1-2 and 3-4, and each of the columns at support 4. In addition, during demolition of the bridge several 10, 16 and 25 mm diameter steel bars, and a specimen of the post-tensioned cables, were obtained for uniaxial tensile tests.

4 INSTRUMENTATION

4.1 General description

To evaluate the bridge's behaviour a comprehensive measuring programme was designed. This section summarises the instrumentation used to measure changes in monitored variables during the bridge girder and slab tests and the non-destructive tests with no external load. In addition, measurements during strengthening were carried out according to the description in previous section.

Before initiating any experimental investigation existing cracks in the entire span 2-3 and the half-spans 1-2 and 3-4 adjacent to the loaded span were mapped. The focus was on cracks in the girders, the crossing beams and the slab at the loading point. In order to follow the formation of cracks, the mapping was repeated after each test sequence. The cracks were mapped manually and their widths were not measured, apart from several cracks specified in the measuring programme.

In addition to the monitoring during bridge loading, long-term measurements were carried out during the nights before Preloading Test Schedules 1-2 and the failure test of the girders. The durations of the monitoring on these occasions were 22398, 21613 and 45558 s, respectively, and the same instrumentation was used as in the followed bridge loading, excluding manual measurements. Moreover, the bridge was examined when the anchorage device for the prestressed CFRP laminates was disassembled.

Most measurements of the bridge were generally acquired at a sampling frequency 5 Hz, except for the long-term measurements (1 Hz).

4.2 Girder test

A battery of instruments was installed before the tests of the longitudinal bridge girders to obtain as comprehensive measurements as possible, within budgetary constraints, of the resulting forces, displacements, curvatures, strains and temperatures. These measurements were complemented by monitoring using several video and still cameras. Data were acquired from all the instrumentation described in this section during the full programme of tests of the bridge girders unless otherwise stated.

Force
The applied load on the structure was measured by monitoring the oil pressure in each hydraulic jack (1-4), illustrated in Figure 4, using UNIK 5000 sensors (GE Measuring and Control; A5075-TB-A1-CA-H1-PA), which have a measuring range between 0 and 600 bar.

Displacement
Displacements of the bridge were measured using the following instruments. Draw-wire displacement sensors (MICRO-EPSILON; WDS-500(1000)-P60-CR-P) were installed to measure deflections at positions 1-10 and 13-15 (Figure 6): in midspan 2-3 on the lower sides of the girders, and lower sides of the crossing beams 500 mm from the outer columns (positions 4-5 and 9-10). All these sensors had a measuring length of 500 mm except those used at positions 6-8 (1000 mm). Twisted lines connected each sensor to a reference point on the ground or the basement.

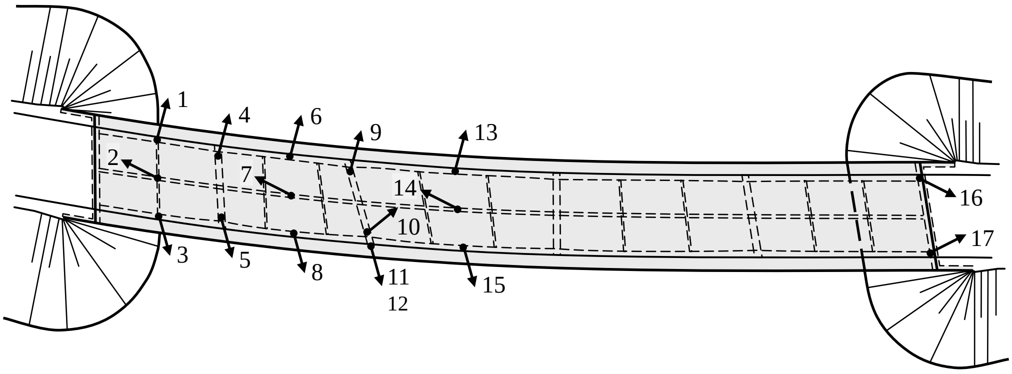

Figure 6 – Positions of bridge displacement sensors.

At positions 11 and 12 both the longitudinal and transverse displacement were monitored using Noptel PSM-200 sensor. The reference point for the horizontal displacement of the bridge slab at the centre line of support 3 was 150 mm perpendicularly away from the southern side of the basement. A transmitter was installed on the basement and a receiver on the lower side of the bridge slab, oriented vertically to the transmitter. The Noptel PSM-200 sensors were only active during the failure test of the bridge girders.In addition, the displacement of the basements' upper side at support 2-3 was manually measured during the girder failure test, 500 mm against the centre of the bridge in transverse direction, in relation to positions 4-5 and 9-10, and the reference point was an unaffected point beside the bridge. For safety reasons, the incremental monitoring proceeded until a certain load was reached, 9.0 MN in total.

At positions 16-17 (Figure 6) longitudinal displacements of the upper part of the rolling bearings, i.e. the lower side of the girders, was measured using linear displacement sensors (Micro-Measurements; HS 100) with a 102 mm measurement range, and positions in the abutment as reference points.

In Preloading Test Schedule 1, load cases 15-27, the width of one crack in the centre of the lower side of each girder (110, 910 and 1380 mm east of midspan for the northern, central and southern girders, respectively) was measured, using crack opening displacement sensors (EPSILON; 3541-010-150-ST) with the measuring range of 10 mm. Data were also acquired from the sensor on the girder strengthened with laminates during Preloading Test Schedule 2 and the bridge girder failure test.

Curvature

The curvature at support 2, support 3 and midspan 2-3 was measured over distances of 4.82, 5.08 and 5.00 m, respectively, using rigs composed of steel beams, supported at the ends, and five linear displacement sensors with 800 mm spacing based. At the supports the rigs were located on the bridge slab, while the midspan rig was located under the girder. Due to the discontinuities at the supports, i.e. changes in directions of the girder, and straightness of the curvature rigs, the instrumentation was installed along the line of the central girder in span 2-3. The sensors were HS 100, HS 50 and HS 25 instruments (Micro-Measurements) with measurement ranges of 102, 51.5 and 26 mm, respectively, set at the positions increasingly distant from the centre of the rigs.

Strain

Strain gauges supplied by Kyowa were installed on the longitudinal and vertical reinforcing steel bars, CFRP rods and laminates, and the concrete surfaces of both the columns at supports 2-3 and next to some major cracks during Preloading Test Schedule 1, load cases 15-27. All of these gauges had 120 ohm resistance, and those installed on the longitudinal reinforcing steel, stirrups or CFRPs and concrete had measuring lengths of 10, 5 and 60 mm, respectively (KFG-10-120-C1-11L1M3R, KFG-5-120-C1-11L1M3R, KC-60-120-A1-11L1M3R). In total, 35 strain gauges were systematically arranged on the longitudinal reinforcing steel bars: at sections A-K in Figure 7 and cross-section positions 1-12 in Figure 8, 1879 mm from the centre lines of the supports on each side (B and J), and 1433 and 2226 mm from each side of midspan 2-3. The locations of the sections were at angles of 45° to the centre line of the supports and 60° and 45°, respectively, to the load distribution plates. On the reinforcement bars they were installed in the corners of the closed stirrups (Figure 8) except at positions 2, 5, 8 and 11, where they were located 1248 mm from the lower side of the girders. All the strain gauges, apart from those in the bridge slab, were placed on the side of the girders.

The locations of the sensors for each section and cross-section position, are specified in Table 2. Due to the greater width of the girder in sections G-H and the corresponding increase in concrete cover strain gauges 25-28 were not used in the final measuring programme. Care was taken to avoid damaging the girder in any way that could potentially affect the quality of the

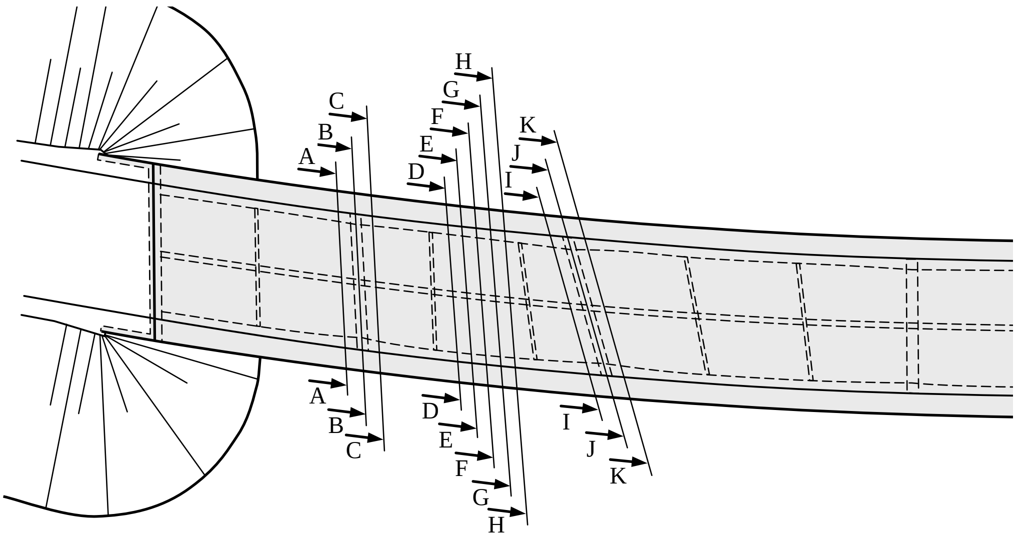

Figure 7 – Positions of strain gauges on longitudinal reinforcing steel.

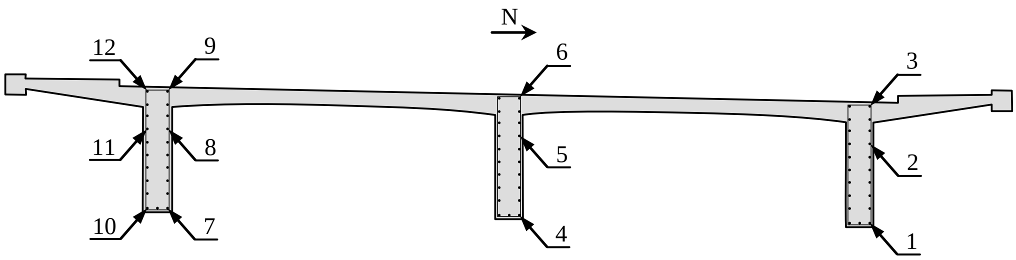

Figure 8 – Cross-section positions of strain gauges on longitudinal reinforcing steel.

Table 2 – Positions of strain measurements on longitudinal reinforcing steel.

No.	Position[1]	No.	Position[1]	No.	Position[1]
1	A6	14	E4	27	H4
2	A12	15	E7	28	H7
3	B1	16	F1	29	I6
4	B2	17	F2	30	I12
5	B3	18	F3	31	J1
6	B6	19	F4	32	J2
7	B10	20	F5	33	J3
8	B11	21	F6	34	J6
9	B12	22	F7	35	J10
10	C6	23	F8	36	J11
11	C12	24	F9	37	J12
12	D4	25	G4	38	K6
13	D7	26	G7	39	K12

[1] Section A-K in Figure 7 and cross-section position 1-12 in Figure 8

strengthening.

As illustrated in Figure 9, strain gauges were also installed in three lines on the vertical reinforcing steel on the northern side of the southern girder in span 2-3, at 900 mm spacing starting from the edge of the loading plate. Thus strain gauges 6-9 were located 1250 mm from the central point of the load application. Vertical distances from the bottom side of the girders to the sensors were 148, 548, 948 and 1348 mm, respectively.

In addition, an ARAMIS system in 5M configuration was used to optically record deformations of the surface on the southern girder on the opposite side to the instrumentation of the vertical reinforcing steel, and accompanying software was utilised to analyse the strains. The optical monitoring was based on a grid, centred 2.0 m west of midspan 2-3, from the bottom of the

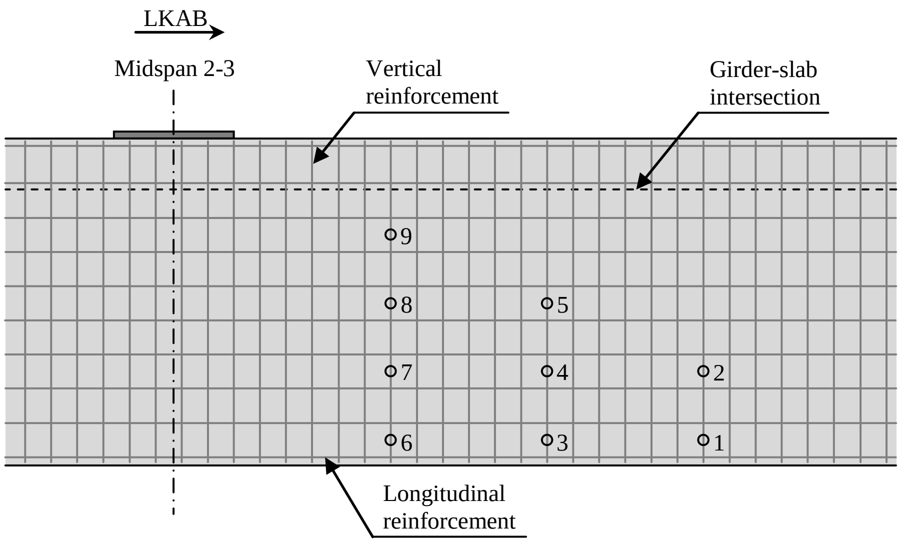

Figure 9 – Positions of strain gauges on vertical reinforcing steel.

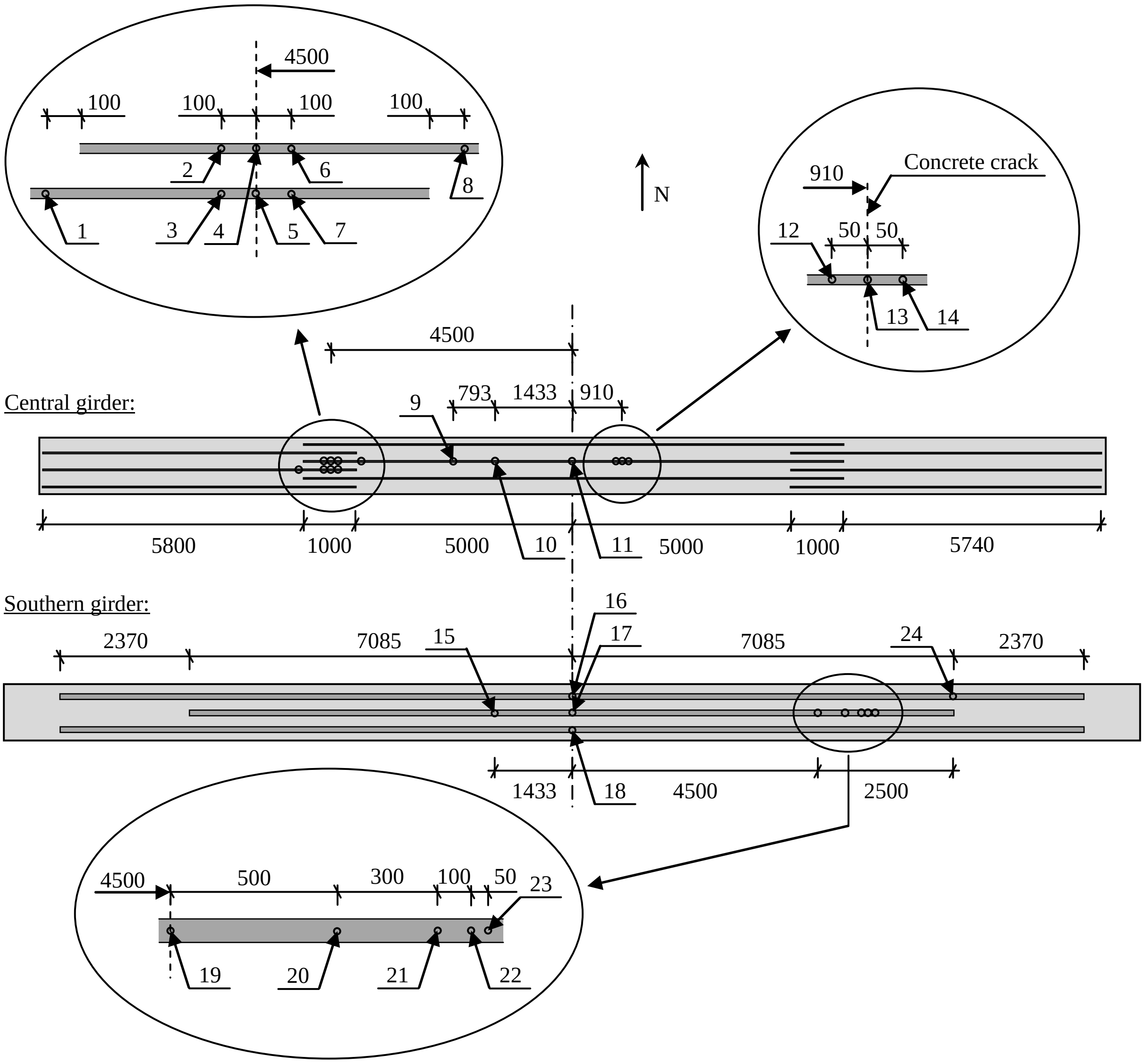

Figure 10 – Geometry of bridge strengthening systems in span 2-3 and positions of the strain gauges on the NSM CFRP rods and prestressed CFRP laminates.

girder. Thus, strain gauges 3-4 according to Figure 9, on the opposite side of the girder, were located within the monitored area, which theoretically covered 1050x880 mm.

In total 14 strain gauges were installed on the NSM CFRP rods and 10 on the prestressed CFRP laminates (Figure 10): gauges 1-8 recorded the strain at the western edge of the NSM strengthening; 9, 10 and 15 were located in the sections equipped with strain gauges on longitudinal reinforcing steel; 11 and 16-18 at midspan 2-3; 14-16 at major concrete cracks and 19-24 next to the anchorage of the laminates.

To obtain the reaction forces in the columns adjacent to the load application in midspan 2-3, i.e. supports 2 and 3, the concrete strains were measured by installing a sensor 800 mm above the bottom in the centre of each side of each column. Before the bridge tests, the methodology of using strain gauges to determine the reaction forces was validated using load cells, while preloading the column with hydraulic jacks and utilising the column's vertical adjustment device.

On each side of the cracks instrumented by crack opening displacement (COD) sensors as described above, the concrete strains were measured. Like the COD sensors, the strain gauges were located in the centre of the lower side of the girders. These sensors were only active in Preloading Test Schedule 1, load cases 15-27.

Temperature
During the experiments temperatures were measured at several locations in midspan 2-3 (Figure 11), using type T (04 N/N-24-TT) temperature wires inserted into holes to specified depths in relation to the concrete surface: 30 mm for positions 1, 3 and 6; 60 mm for positions 2 and 7; 50 mm for positions 4 and 8; and 80 mm for position 5.

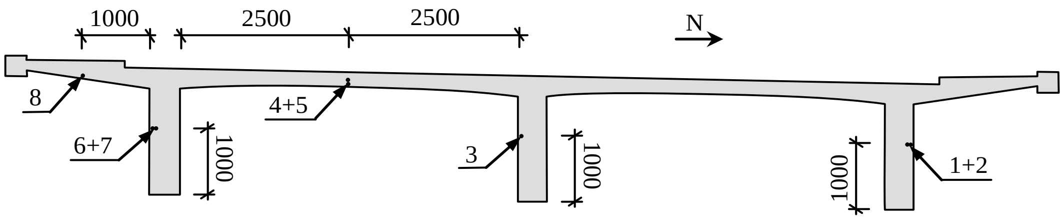

Figure 11 – Positions of temperature wires in the concrete at midspan 2-3.

4.3 Slab test

Relevant instrumentation that was still intact after the girder failure tests, and complementary instrumentation, was used to monitor the behaviour of the bridge during the following bridge slab failure test. The sensors still active during this test were:

- the oil pressure sensor for hydraulic jack 1, as shown in Figure 4 and Figure 5,
- draw-wire sensors 4-7 and 14-15, as shown in Figure 6;
- strain gauges 1-39 as specified in Table 2, excluding gauges 12, 17 and 24-28, which were not used for various reasons;
- strain gauges 1-9 as shown in Figure 9, except gauge 8, which was out of order;
- strain gauges 1-24 installed on the columns at supports 2 and 3;
- temperature wires 1-8, as shown in Figure 11.

Displacement
The above instrumentation was complemented with four draw-wire sensors, with similar specifications to the sensors utilised in previous tests. Two were located on the lower surface of the slab, at the centre of the load applications, to measure deflections, and two on the lower side of the northern longitudinal girder, in both cases 2.00 m on either side from midspan 2-3.

Curvature
To monitor curvature in the slab test the rigs used in the girder tests at supports 2 and 3 were installed on the top surface of the slab, parallel to the steel beam used for load application, 500 and 1000 mm southern to the centre of the loading plates. The midpoint of this instrumentation coincided with midspan 2-3.

4.4 Non-destructive test

Three strain gauges of the same type as previously specified for monitoring the concrete were used in the non-destructive tests to determine the residual forces in the post-tension cables, located in a line in the centre of the lower sides of each girder in span 2-3. In order to provide enough space to avoid damaging the sensors while cutting the concrete, the centre-centre distance was 120 mm, since the total length of the strain gauges was 74 mm with a 60 mm measuring length.

5 RESULTS

5.1 General description

In the experimental programme for the girder tests the bridge was instrumented with sensors at up to 141 positions in total, excluding the surface measurement using ARAMIS, and 93 sensors were used in the bridge slab failure test. General observations regarding the test procedures and the observed load-carrying capacity of the bridge are presented in this section.

5.2 Girder test

The loads applied in the preloading schedules and loading the bridge to failure, according to the recorded pressures in the hydraulic jacks, are illustrated in Figure 12 to Figure 14, which show that the preloading followed the schedules listed in Table 1, with minor deviations due to difficulties in manually controlling the oil pressure. In Preloading Test Schedule 1 (Figure 12) the complementary instrumentation used to determine the remaining forces in post-tensioned cables was installed after approximately 7400 s. The time spent installing it (about 5.5 hours including associated operations) is not shown in the graph, but no corrections have been applied to the force-time courses shown in Figure 13.

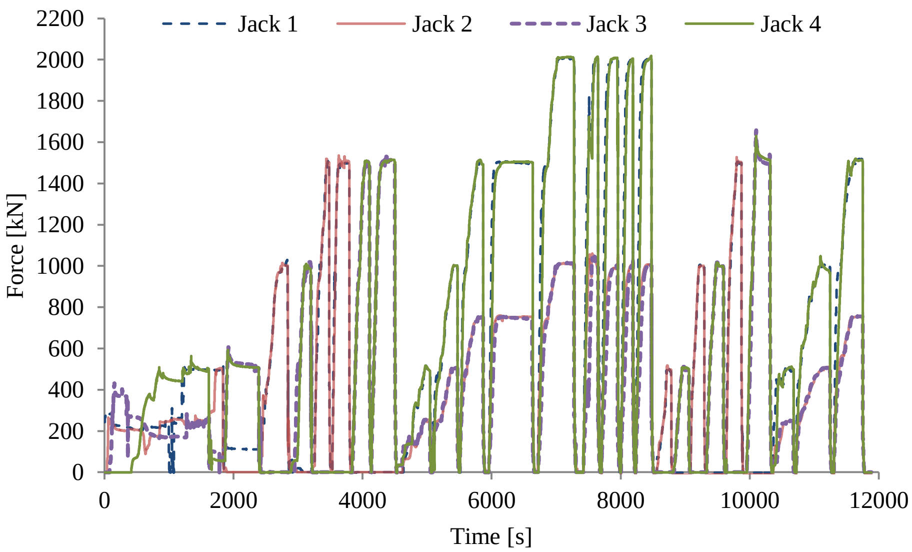

Figure 12 – Observed loadings during Preloading Test Schedule 1, unstrengthened girder.

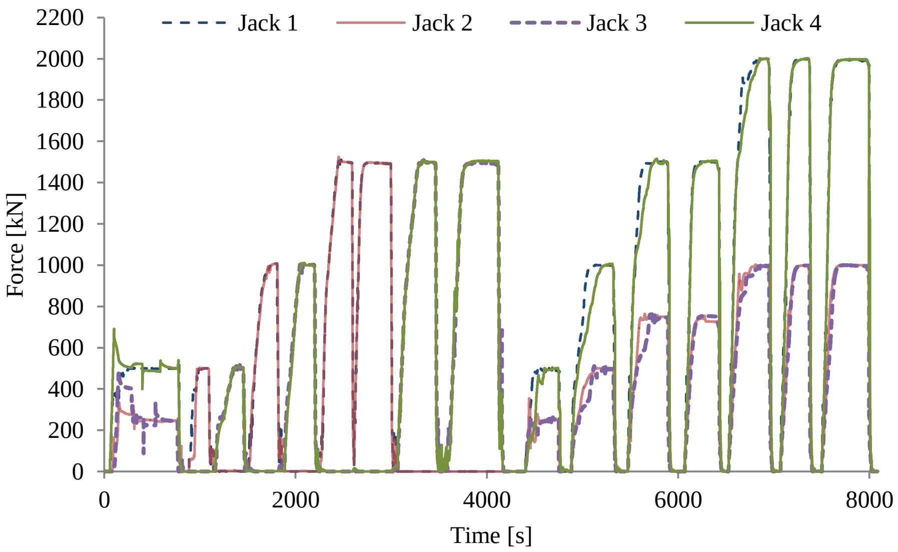

Figure 13 – Observed loadings during Preloading Test Schedule 2, strengthened girders.

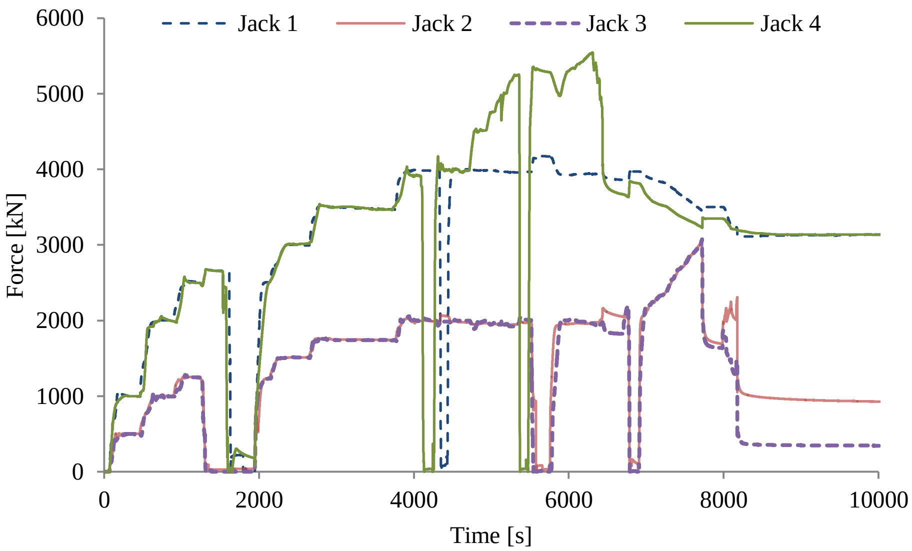

Figure 14 – Observed loadings during the bridge girders failure test.

In order to manually follow the basement settlements of the bridge safely, the loading was carried out stepwise up to a certain level, see Figure 14. Another reason for the irregularity in the loading procedure was the limited stroke length of the hydraulic jacks, which required the grip position to be changed several times to accommodate longer deflections of the bridge.

After applying a total load of 12.0 MN (4.0 MN for each girder), the pressure in jack 4 was increased to reach failure of the southern gird§er, while the pressure in the other jacks remained nearly constant. However, the pressure in jacks 1 and 4 slightly decreased as the central girder was loaded to failure using jacks 2-3, in responses related to the deformations of the bridge.

Deflections of the bridge are illustrated by the load-displacement curve in Figure 15, showing the relationship between the total load and midspan deflection of the central girder. Figure 15 also presents the behaviour according to finite element analysis with 2D and 3D idealisation in the software ATENA and ABAQUS, respectively. Unfortunately, draw-wire sensor 8 (Figure 6), was damaged during the test, so the midspan deflection of the southern girder is not available for the entire test. The highest loads the longitudinal southern and central girders were subjected to induced deflections of 136 and 159 mm, respectively (Figure 15). However, the bridge loading was further continued. The shapes of the girders after the test are shown in the photograph in Figure 16. The peak load at loading the southern girder to failure was 13.4 MN (5.5 MN in jack 1) and 12.7 MN for the central girder (6.1 MN in total in jacks 2-3).

5.3 Slab test

The data acquired from the specified test setup indicate that the load-carrying capacity of the bridge slab was 3.32 MN. Thus, the load transferred in each loading plate was 1.66 MN. The slab failed only at the western load distribution plate, displaying very brittle behaviour with no

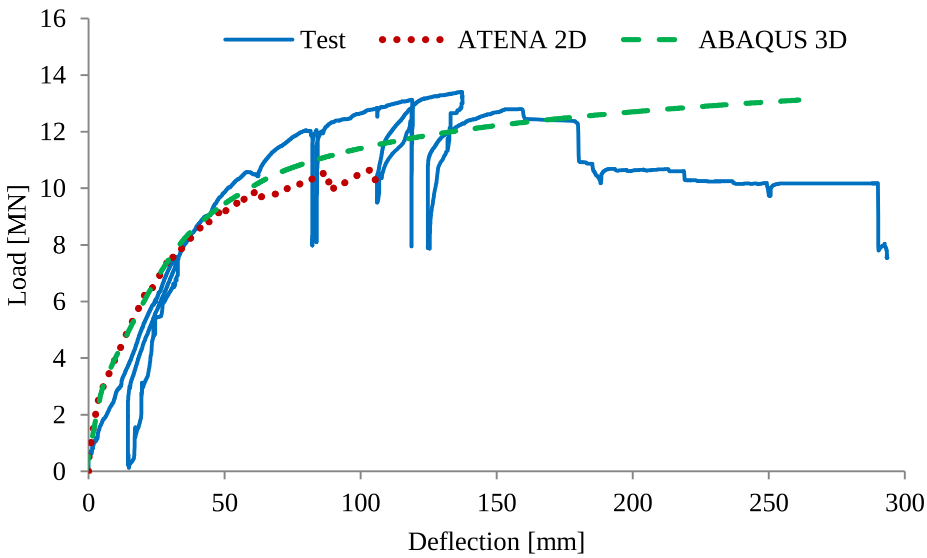

Figure 15 – Load-displacement relationship during the bridge girder failure test.

Figure 16 – Photograph of the bridge girders after failure, view from the south (2014-08-26).

Figure 17 – Photograph of the bridge slab failure, view from south (2014-06-27).

appreciable indication of the failure. In Figure 17 the final shape of the bridge slab is presented from underneath the bridge. Due to the arrangement of the test setup and type of failure it was not possible to further load the slab to achieve failure at the eastern load distribution plate.

6 CONCLUSIONS

Closure of the Kiruna Bridge provided a rare opportunity for LTU to monitor a post-tensioned concrete bridge during tests to failure using a wide array of instruments, from May to August 2014. The primary aim was to acquire relevant data for calibration and development of methods for assessing prestressed and post-tensioned concrete structures. The results acquired during the investigations reported in this paper suggest that the following parameters warrant further attention:

- Robustness, ductility and bridge behaviour;
- Shear resistance at ultimate limit state;
- The utility, behaviour and contributions to increases in capacity of strengthening methods using CFRPs;
- Punching resistance of bridge slabs;
- Condition of post-tensioned steel cables and non-destructive determination of residual forces;
- Reliability-based analysis of reinforced concrete structures;
- Finite element model updating.

Detailed analyses of these parameters would greatly facilitate improvements in models for assessing existing concrete structures and thus savings of costs, for bridge owners and managers.

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