# Full Scale Test of a PC Bridge to Calibrate Assessment Methods

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## Abstract

In this paper, experiences on the development of an assessment method for existing bridges are presented. The method is calibrated using the results of full-scale testing to failure of a prestressed bridge in Sweden. To evaluate the key parameters for the structural response, measured by deflections, strains in tendons and stirrups and crack openings, a sensitivity study based on the concept of fractional factorial design is incorporated to the assessment. Results showed that the most significant parameters are related to the tensile properties of the concrete (tensile strength and fracture energy) and the boundary conditions. A finite element (FE) model in which the results of the sensitivity analysis were applied, was able to predict accurately the load-carrying capacity of the bridge and its failure mode. Two additional existing prestressed concrete bridges, that will be used to improve further the method, are also described, and discussed.

**Keywords:** Assessment methods, calibration, boundary conditions, existing prestressed concrete bridges, tensile strength, fracture energy, full-scale testing, finite element modelling

## 1 Introduction

Demands to keep existing bridges in a healthy condition and able to carry increased loads make the ability to assess their true capacity an important task. Elaborate finite element (FE) methods are available for their capacity assessment, but the results obtained are heavily dependent on the input parameters and how well they describe the object they intend to model. Therefore, an incorrect definition of these parameters can generate large variations when numerical results are compared to real values obtained in tests, see Bagge et al. [1-2]. However, significant improvements on FE analysis can be achieved when in-situ results are available [3-5].

In this paper, a methodology to improve the assessment and FE models of an existing bridge is presented based on the experiences obtained from the full-scale test to failure of a prestressed concrete bridge [2, 6-9]. Assessment methods have been proposed at various levels with gradually increasing accuracy [10-14]. By increasing the efforts, the structural response and the load-carrying capacity can be more accurately estimated. An example of this process, applied to the analysis of the bond failure of Carbon Fibre Reinforced Polymer (CFRP) bars used to strengthen a RC bridge, can also be found in Puurula et al. [15].

As part of ongoing research, two additional bridges are presented. For these bridges, the methodology is planned to be implemented to ensure a good understanding of the structural behaviour which, for instance, is crucial for capacity estimates.

## 2 Testing the Kiruna Bridge

The prestressed concrete bridge in Kiruna (northern Sweden) shown in Figure 1 was taken out of service due to uneven ground settlements caused by the extension of an iron mine under the bridge. It was built in 1959 and had five continuous spans with a total length of 121,5 m and a width of 15,6 m (see Figures 2-3).

*Figure 1. Kiruna Bridge constructed in 1959, Nilimaa et al. [3]*

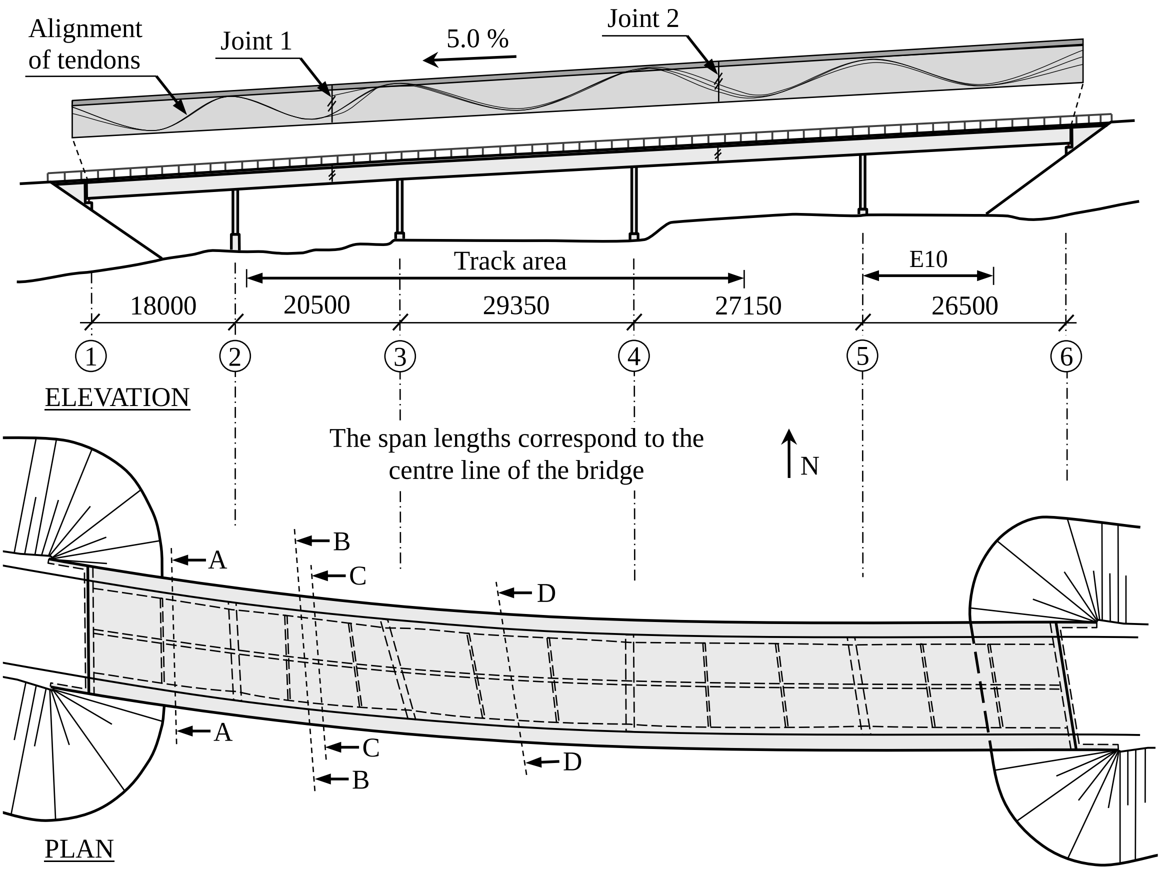



*Figure 2. Elevation and plan view of the Kiruna Bridge, Bagge [2].*

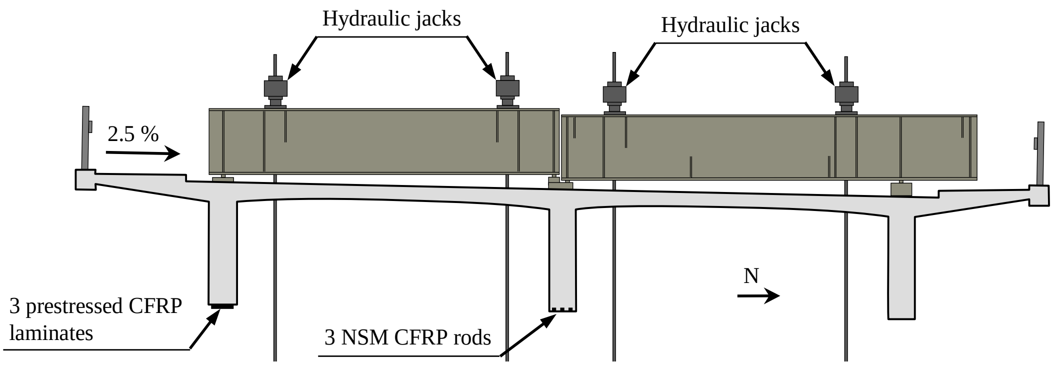



*Figure 3. Section view and loading arrangement, Bagge [2].*

The bridge carried road traffic from the centre of Kiruna to the mine, and it crossed the Euro route E10 and the iron ore railway between Luleå (on the Baltic Sea) and Narvik in Norway (on the Atlantic Ocean).

Before demolition, the bridge was loaded to failure in the summer of 2014 by applying concentrated loads in the middle of the second span. The witnessed failure mechanism consisted in a combined shear-bending failure in the girders and the slab as shown in Figure 4 [2, 6-9].

## 3 Simplified assessment versus in-situ tested results

In Figure 4, the load-deflection curves at mid-span of the three girders are shown. The experimental curves are also compared to those obtained through a standard structural analysis using a FE model with linear uncracked and cracked concrete response, respectively [2]. As it is clearly shown in Figure 4, there is an important variation in the true stiffness of the bridge, which illustrates the need of using a better calibrated non-linear analysis for a more correct modelling of the behaviour.

The final bending-shear failures of the south and central girders are also illustrated in Figure 4. As indicated, the loading of the bridge resulted in extensive concrete cracking at the midspan region of the girders, in addition to yielding of longitudinal reinforcement and stirrups and, consequently, large deformations. First, the southern girder was tested to failure and a peak load of 13,4 MN was achieved. Failure initiation was indicated in the slab and in the upper part of the girder where diagonal cracking was formed. Rupture of the stirrups crossing the cracks took place, first on the west side and then the east side. Thus, the failure mechanism incorporated, beside bending, both out-of-plane shear in the slab, where the loading plate punched through, and in-plane shear in the girder.

Subsequently, loading of the central girder was carried out. A similar failure mechanism was observed with a total external load of 12,7 MN. Here, the test showed a remarkable robustness of the bridge structure as it was able to keep carrying loads after the failure of the southern girder.

Due to the complexity of the measured failure mechanisms, there are difficulties in predicting the behaviour and the capacity using standard resistance methods [2]. This also indicate the need of more sophisticated methods to facilitate a more reliable and correct assessment.

## 4 Sensitivity study

### 4.1 Enhanced assessment

Nonlinear FE analysis was used to more accurately assess the structural response and the load-carrying capacity of the Kiruna Bridge. First an initial model was created based on existing modelling guidelines [18-19] to support the analyst in the choices of input parameters. In the second step a sensitivity study was carried out to investigate the importance of the parameters initially assumed. The parameters for further attention were thereby identified, thus, providing basis for updating the model. With this methodology for improving the FE model, it was possible to reduce the extent of uncertainties in the assessment.

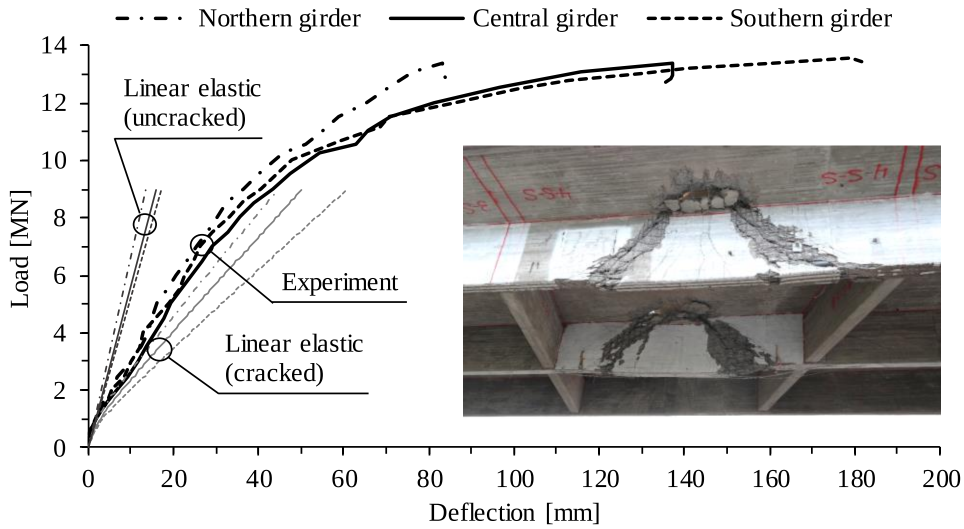



*Figure 4. Load-deflection relationship for the test of the bridge's girders and photo of south and central girders after shear-bending failure.*

## 4.2 Parameters studied

The concept of fractional factorial design at two levels (see Figure 5) with resolution IV was used in the sensitivity study [16]. Level A corresponds to the assumptions in the initial FE model, while level B gives the other extreme to be examined. In total, 32 simulations were carried out and 14 modelling parameters were investigated [2].

The effects were studied on midspan deflections of the northern, central, and southern girder ($\delta_{N,max}$, $\delta_{C,max}$ and $\delta_{S,max}$), maximal strain in tendons and stirrups ($\varepsilon_{sp,max}$ and $\varepsilon_{sw,max}$), and maximal crack opening ($w_{max}$).

The model was evaluated at a total load level of 12 MN, a load level where the loading procedure was changed. Although the response was just studied for this load level, the most important parameters and their influence on the results were expected to be identified.

Quantitative parameters related to the material model of concrete were mainly studied, but the following qualitative aspects were also included:

- Distribution of residual prestress forces (denoted $\sigma_{sp}$), either varied according to detailed analysis of prestress losses (level A) or assumed to be constant with prestress losses of 30 % (level B);
- Strengthening of the central girder with Carbon Fibre Reinforced Polymers (CFRP), either included (level A) or excluded (level B);
- Boundary conditions at the bottom of the columns (denoted BC), either freely rotational around all axes (level A) or fully restrained with no rotation (level B).

For the material model, most parameters included were studied. The levels specified for the modulus of elasticity ($E_c$) and compressive strength ($f_c$) were derived from cylinder tests (level A) and values given by the assessment code for the current concrete class specified in construction drawings (level B) [17], respectively.

In the initial FE model, both the tensile strength ($f_t$) and the fracture energy ($G_f$) were determined from the tested compressive strength (level A), which indicated a time-dependent increase not expected to be of the same magnitude for the tensile properties. Due to a potentially large variation in the tensile strength and fracture energy, these properties were also investigated at values half those initially calculated (level B).

Regarding the Poisson's ratio ($\nu$) and the limiting compressive strength reduction factor ($r^{lim}_{c}$ ), their values suggested by Rijkswaterstaat [18] (level A) and ATENA [19] (level B) were used. Moreover, the upper (level A) and lower (level B) limits recommended by [19] were investigated to find what level to set the crack rotation ($c_{fc}$), the tension stiffening ($c_{ts}$) and shear stiffness reduction factor ($s_F$). The predefined crack spacing ($s_{r,max}$) and the aggregate interlock parameter (denoted $a_g$) were set to values from the recommendations in [19] (level A), or taken as a half of the recommended value and zero (level B), respectively.

## 4.3 Results

Figure 5 shows how the bridge structural response is influenced by changes in the investigated modelling parameters. The change in response is scaled so that the increase (**blue**) or decrease (**red**) due to the studied parameters changes can be compared. Note that the influences of the different parameters shown in Figure 5 can only be compared for a certain response parameter, while the influence of a given modelling parameter on different response parameters cannot be directly compared.

Generally, the study showed that the assumed boundary condition at the bottom of the columns, the tensile strength, the fracture energy, and the crack spacing are parameters of high importance, particularly for the deflections and strains in the prestressed reinforcement. The strains in the stirrups were mainly affected by the tension stiffening, aggregate interlock, and the shear stiffness reduction factors, leading to reduced values when changing from level A to B, while the variation on fracture energy resulted in the opposite effect.

The sensitivity study indicated that the tested material properties (compressive strength and elastic modulus) were of minor importance for the bridge response. In contrast, the tensile properties (tensile strength and fracture energy) were shown to be important and, thus, preferably determined accurately through in-situ testing. Failure to do this might result in unnecessary uncertainties being built into the FE model if the relationships between

| Factors | Levels A | Levels B |
|---|---|---|
| $\sigma_{sp}$ | Varying | Constant |
| CFRP | Yes | No |
| BC | Hinged | Fixed |
| $E_c$ | 32.1 MPa | 34.0 MPa |
| $v$ | 0.15 | 0.20 |
| $f_t$ | 4.30 MPa | 2.15 MPa |
| $f_c$ | 62.2 MPa | 35.5 MPa |
| $G_f$ | 208 N/m | 104 N/m |
| $c_{fc}$ | 0.60 | 0.90 |
| $L_t$ | 200 mm | 100 mm |
| $c_{ts}$ | 0.01 | 0.05 |
| $a_g$ | Yes | No |
| $s_F$ | 20 | 200 |
| $r_c^{lim}$ | 0.40 | 0.80 |

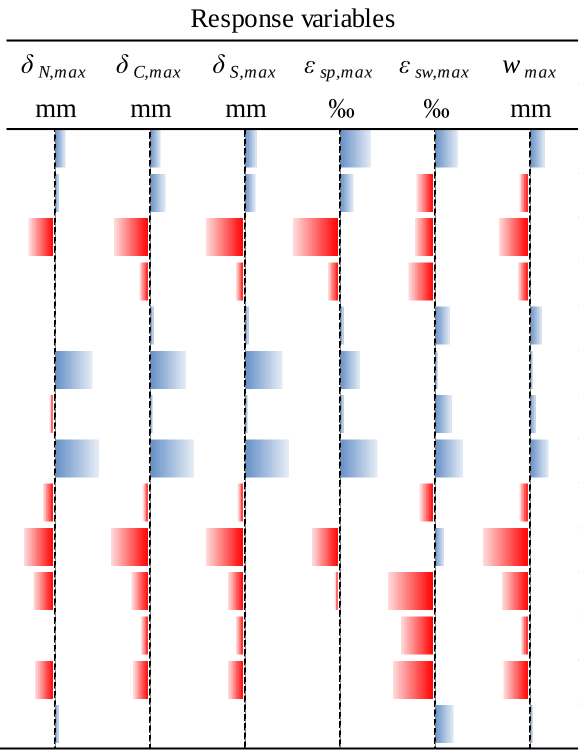



*Figure 5. Sensitivity study of the Kiruna Bridge externally loaded to 12 MN with the response variables being maximal deflections ($\delta_{i,max}$), maximal tendon and stirrup strains ($\varepsilon_{sp,max}$ and $\varepsilon_{sw,max}$) and maximal crack width ($w_{max}$). The levels correspond to modelling parameters assumed in the initial FE analysis (Level A) and in the sensitivity study (Level B). A **red** colour indicates a decrease in response when a factor goes from (A) to (B), while a **blue** colour indicates an increase. Bagge [2].*

compressive strength and other material properties, determined with relationships calibrated for new concrete, are not applicable for existing structures. In addition, the model was sensitive to other parameters in the concrete constitutive model that are not easily determined for an existing structure (e.g., tension stiffening, aggregate interlock, and shear stiffness reduction due to cracking). Here, it is necessary to rely on well established theories.

Reported results also indicates the importance of a sensitivity study in order to determine and understand the impacts of different parameters on the current problem. The use of fractional factorial design to examine a large number of modelling parameters with a limited number of simulations proved to be useful here.

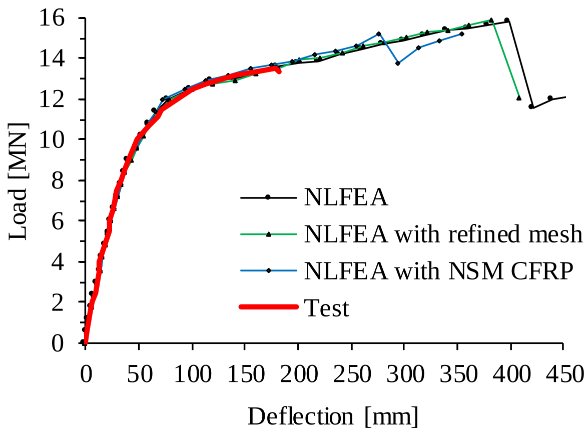
## 5 Updated nonlinear FE analysis

The initial model (level A) did not predict the correct failure mode and overestimated the load-carrying capacity, as shown in Figure 6a for the response of the south girder.

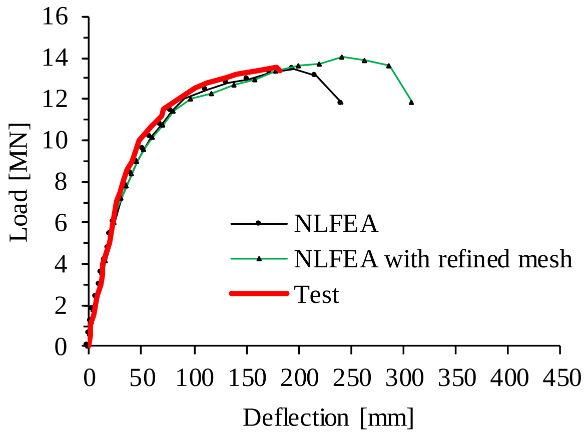
Consequently, in order to better reflect the behaviour and failure mode observed in the full-scale test of the bridge, the updated model was developed [2, 12]. In addition to the experience from the experiment, the knowledge from the sensitivity study was further utilised with a particular focus on the parameters shown to be of highest importance within the value ranges investigated. Figure 6b shows that the updated model predicts a closer value of load-carrying capacity and final deflection when compared to those attained in the original model.

## 6 Ongoing work

The importance of accurate assessment methods has recently been emphasized by Frangopol [21] and Täljsten et al. [22]. The fracture properties of concrete in old existing bridges in Sweden are being studied by Nilforoush et al. [23] and assessment procedures for railway bridges on the Iron Ore line by Coric et al. [24]. The procedure developed for the assessment of the Kiruna Bridge will be extended for the cases of existing bridges located in northern Sweden as part of ongoing work at LTU. At present two prestressed concrete bridges are studied: (1) Bridge over the Kalix River in Kalix, and (2) Bridge over the Torne River at Autio.




*(a)*




*(b)*

*Figure 6. Load-deflection curves for the south girder: (a) Initial NLFEA model; (b) updated NLFEA model.*

### 6.1 Bridge over Kalix River

The bridge was built in 1957 and has five spans of 43,85 m, 47,0 m, 84,0 m, 47,0 m, and 43,85 m for a total length of 275,7 m (see Figure 7). The bridge will be replaced by a new composite bridge as it has been assessed not to be fit to carry traffic with heavy trucks. Load testing, aimed to check its condition and function, is planned to take place during 2021/22.

As a first step, a linear-elastic analysis of the bridge was carried out using a 2-dimensional FE model.

*Figure 7. Bridge over Kalix River in Kalix, view towards south.*

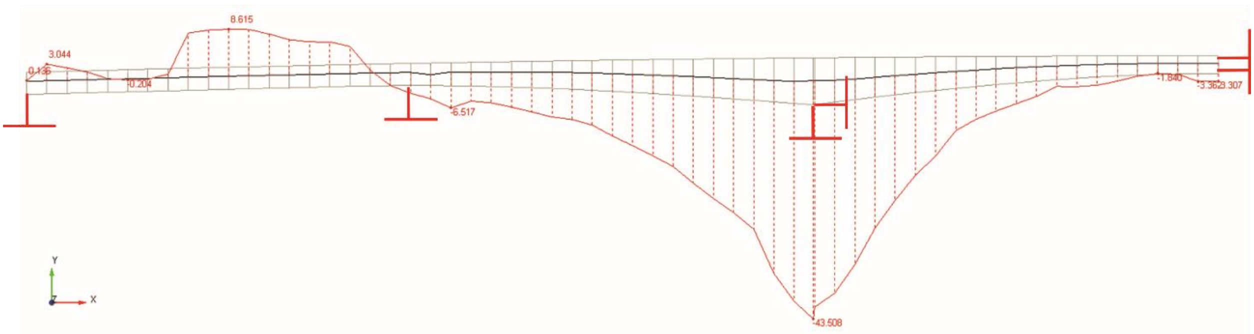



*Figure 8. Moment diagram of Kalix Bridge for a moving convoy load.*

The bridge was modelled using the available information (drawings, original design calculations, etc). Figure 8 shows the moment diagram along the bridge length for a moving convoy defined by the Swedish assessment code [20]. This preliminary analysis showed that a potential critical section in the superstructure is the one above pier 2 (second from right to left in Figure 7).

Further analysis is planned to be performed for this structure in order to provide thorough understanding of its behaviour. Following the methodology applied for the Kiruna Bridge (see section 4), the parameters that have the most significant influence on the structural behaviour can be determined also for the Kalix Bridge. Based on such an evaluation, in-situ data is collected and with reduced and/or eliminated uncertainties a more accurate structural assessment can be achieved.

### 6.2 Bridge over Torne River

This bridge was built in 1963 and has three spans of 36,4 m, 62,0 m, and 36,4 m for a total length of 134,8 m (see Figure 9). The width is 7,48 m. The prestressing was applied with BBRV cables of 32 threads with 6 mm in diameter.

A new mine has started in the neighborhood and heavy trucks are passing over the bridge. Many concrete cracks have been observed parallel to the tendons, and the bridge is continuously monitored to check its function. A new assessment is planned to estimate the remaining life length of the bridge and to provide basis for decisions for further actions. As experienced for the Kiruna Bridge, use of enhanced structural assessment methods is a possible measure to produce a reliable and precise assessment.

## 7 Discussion and Conclusions

The sensitivity study performed in this paper indicated that the tested material properties

*Figure 9. Bridge over Torne River at Autio (Sweden).*

(compressive strength and elastic modulus) were of minor importance for the bridge response, at least for the case in study. In contrast, the tensile properties (tensile strength and fracture energy) were shown to be important and, thus, preferably determined accurately through in-situ testing. Failure to do this might result in unnecessary uncertainties being built into model for assessment. Future research should focus on evaluating if the relationships between compressive strength and other material properties, determined for new concrete, are also applicable for existing structures

Furthermore, the study shows that the assumed boundary conditions at the bottom of the columns, is also of significant importance, particularly for the deflections of the girders and the strains in the prestressed reinforcement.

Finally, one can conclude that the assessment methodology implemented in the Kiruna bridge showed potential, and it can be applied and verified in other similar available bridges using full-scale testing. Associated to the assessment methodology, the importance of a rational treatment of the uncertainties is also demonstrated.

## 8 Acknowledgements

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