CHALMERS
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Punching Shear Capacity of a Tested RC Bridge Deck Slab in Kiruna, Sweden

Master's Thesis in the Master's Programme Structural Engineering and Building Technology

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Concrete Structures
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Cover:
Photo of the tested bridge in Kiruna. Photographed by Shu Jiangpeng in June 2014

Department of Architecture and Civil Engineering. Goteborg, Sweden, 2015

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ABSTRACT

To meet future transportation demands, there is need to improve existing transport infrastructures. One way to achieve this is by predicting the load carrying capacity of existing bridges more accurately to allow heavier traffic to pass. Reinforced Concrete Bridge Decks are among the most critical structural members that need checking. This study exemplifies a multi-level structural assessment with successively improved analysis of the punching shear capacity of an existing bridge deck slab.

In order to calibrate existing methods for structural assessment, a full-scale test to failure was conducted on a Bridge in Kiruna, Sweden, from May to August 2014. Preliminary calculations shows that the calculated punching shear capacity of the bridge deck was much lower than the capacity obtained in the test. Consequently, an improved approach for calculating the capacity of bridge decks are needed.

The results from this study show that with simplified methods, using linear analysis, the calculated load carrying capacity was 26.8% of the capacity obtained from the test. When instead using a non-linear 3D shell model in combination with the critical shear crack theory in fib. (2013), 70.5% of the test value was achieved.

Key words: Punching Shear Capacity, Bridge Deck Slab, Multi-Level Structural Assessment, Punching Shear, Load Carrying Capacity

Contents

Preface

This master thesis was carried out in the Department of Civil and Environmental Engineering (after re-organisation in 2017, Achitecture and Civil Engineering), Division of Structural Engineering at Chalmers University of Technology, Gothenburg, Sweden.

The work was conducted mainly during 2015, under the supervision of Jiangpeng Shu, Ph.D. student at Chalmers University of Technology, and Niklas Bagge, Ph.D. student at Luleå University of Technology. The examiner was Mario Plos, Associate Professor at the Division of Structural Engineering at Chalmers University of Technology. I would like to express my most sincere gratitude for all their tireless effort rendered to me during this research. All their guidance and available-to-help expressions were of great value in ensuring the outcome of the research.

This publication has been produced during my scholarship period at Chalmers University of Technology, thanks to the Swedish Institute scholarship.

1 Introduction

1.1 Background

Bridge infrastructure in Europe constitutes an important part of the transportation sector. Most of the existing bridges were constructed more than a half a century ago and need assessment. For example in railway sector alone Sustainable Bridges (2007), almost 70% of the current bridges in Europe are more than 50 years old. Some of these bridges have already passed their design life or are being used for loads higher than originally design for, and are not meeting the current and future traffic loads. Furthermore, the change in urban planning and population mobility increases the requirements on load carrying capacities of the bridges. Consequently, bridges need assessment to ensure that they still perform according to the design requirements.

Bridge deck slabs are often the most exposed and critical parts with respect to the load-carrying capacity of the bridge. It is therefore of importance that there are accurate assessment methods available for the bridge decks.

In an ongoing study at Chalmers University of Technology, an assessment strategy for reinforced concrete bridge deck slabs is being developed, Plos *et al.* (2015). This strategy is based on the assessment procedures for a systematic and cost-effective assessment and strengthening of existing railway bridges developed in the integrated European project Sustainable Bridges (2007)

Assessment based on this strategy is made on a bridge tested in a full-scale experiment in Kiruna, Sweden. The bridge was constructed in 1959, and its slab and girders were subjected to an experimental investigation in 2014, (Bagge *et al.*, 2014). The bridge had five continuous spans with a total overall length of 121.5metres. The bridge consisted of three longitudinal girders connected by the deck slab and cross beams. During the experiment, the bridge deck slab was loaded to failure using a force-controlled hydraulic jack located in the mid-span near one of the main girders.

The girders and the slab were monitored during the test with the force being applied using the hydraulic Jack. The deflections of both the slab and girder were being measured using draw wire sensors from the underside. At 500 and 1000 mm south of the centre line through the loading points, the curvature was measured along the bridge longitudinal direction. More detail about the experimental instrumentation is in (Bagge et al., 2014).

1.2 Aim

The aim of this research was to study the test of the bridge deck of the bridge in Kiruna and to evaluate the methods previously developed for assessment of bridge deck slabs by using them on the bridge tested.

1.3 Method

This project was initiated after the full-scale test was made on the bridge in Kiruna. It consisted of several parts. A literature review on punching of bridge deck slabs was made. The experimental results from the bridge test were evaluated in order to understand the behaviour of the bridge deck during the full-scale test, and to get a basis for comparison. A multi-level structural assessment was made and the results were compared to the test. The structural assessment was made with the following analysis methods:

- Yield line analysis for determination of the moment capacity
- Eurocode model for punching capacity on level 1
- Simplified assessment using a two-dimensional (2D) linear analysis method
- Enhanced assessment using three-dimensional (3D) non-linear shell FE analysis

1.4 Limitations

The study was limited to punching shear capacity of concrete bridge deck slabs. The Multi-level assessment strategy presented in Plos *et al.* (2015) was used in this study. In this paper assessment has been carried out only for level I, simplified analysis method and level III, 3D non-linear shell FE analysis.

1.5 Thesis outline

This section consists of an outline and summary of the research. *Chapter 1* contains an introduction to the study with the background, the Aim, the Methodology used during the process and the limitations associated with the thesis. *Chapter 2* provides a theoretical overview of bridge assessment, punching shear of reinforced concrete slabs, and of the multi-level structural assessment strategy presented by Plos *et al.* (2015). *Chapter 3* presents a general overview of the bridge in Kiruna while *Chapter 4* describes the punching shear test of the bridge. *Chapter 5* contains the bridge assessment and analysis at different assessment levels. The final research discussions, including the assessed load-carrying capacity of the bridge are summarized in *Chapter 6* while *Chapter 7* depicts further studies needed to enhance this research.

2 Structural assessment of bridges

2.1 The multi-level structural assessment approach

The need for assessment of an existing bridge begins with uncertainty whether the bridge fulfils its functional requirements, see e.g. Sustainable Bridges (2007) and Plos *et al.* (2015). The reason for the uncertainty may be a need for modification or reconstruction, or that the bridge suffers from deterioration or damage.

It proceeds with reviewing the available bridge information, sites visits and using simple analysis. The feasibility of further steps depends on the availability of both technical and financial resources. The level of the assessment detail and the extent of the required information can be determined depending on the particular bridge in consideration.

Figure 2.1 below shows the proposed phases of bridge assessment. In initial phase, the bridge is assessed using the available data at the initial stage; it includes previous calculations, any previous assessment report, and drawings. A simple calculation is also performed to check the deviation from the expected comparisons. The results from the initial stage determine whether further assessment is required.

Once the bridge is found to be sufficient in the initial phase, the assessment is terminated. Otherwise, the next step follows with an intermediate stage where the assessment is based on more advanced procedures with more accurate information. It involves obtaining the material properties or performing simple tests. The above-obtained information forms the input for the intermediate stage. If the intermediate results do not give satisfactory results, the assessment will have to move to a more improved level; the enhanced phase. In the enhanced stage, the bridge can be assessed using improved methods compared to the intermediate stage. Use of reliability based assessment method, system-level assessment, evaluation of site-specific loads are some of the methods to be employed. The analysis can be done using non-linear analysis and statistical modelling among others. All steps used focus on identifying the best possible way to assess the bridge performance. It is also at this stage where the bridge may be determined to be viable for use, have need for an upgrade or is ready for demolition.

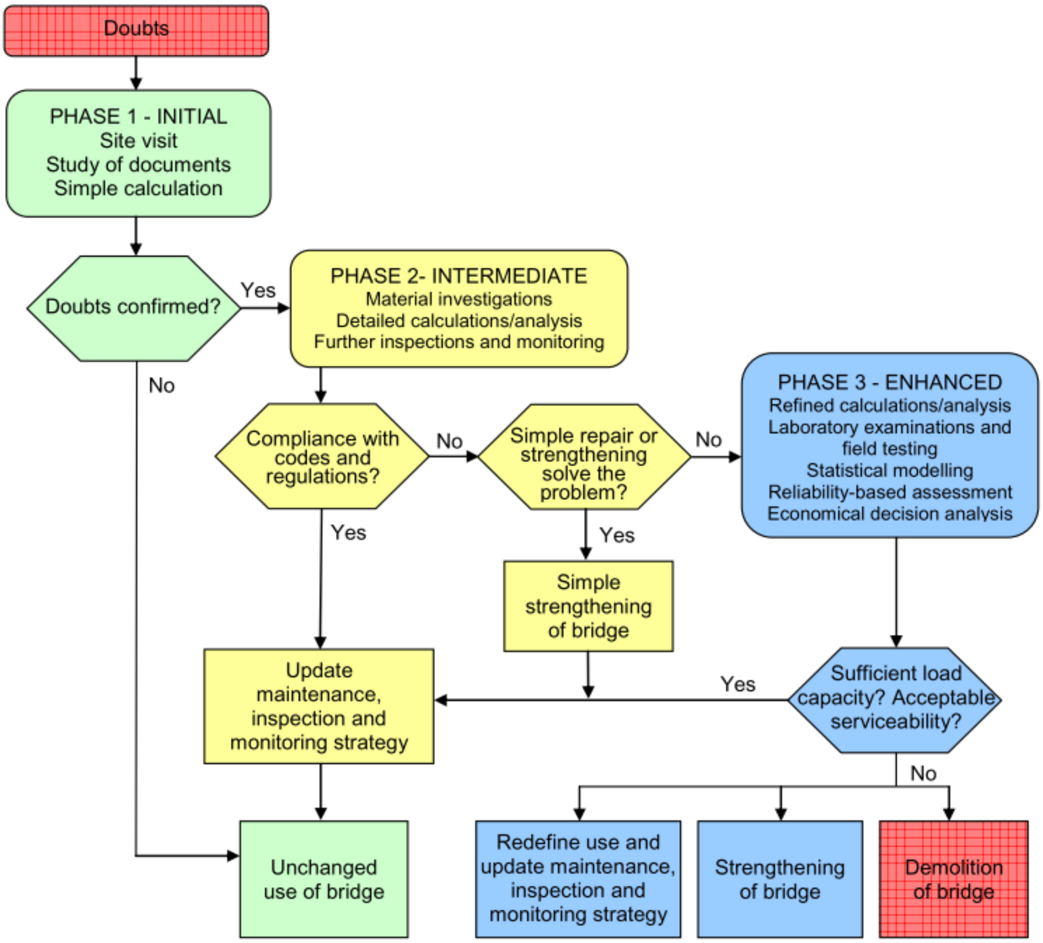

Figure 2.1 Flow chart for existing bridge assessment. Adapted from (Sustainable Bridges 2007)

In this study, a multi-level assessment strategy is employed Plos *et al.* (2015). The strategy proposes a successive refinement of the assessment by applying both linear and non-linear analysis of the structure through five levels. Level I is based on simplified 2D linear analysis followed by Level II, in which 3D linear analysis is used. Level III uses non-linear analysis with shell elements for the slab. Level IV uses 3D non-linear FE analysis with continuum elements and embedded reinforcement while the last level, Level V, uses 3D non-linear FE analysis with continuum elements and reinforcement bond. The levels corresponds to different accuracy in the analysis. Lower levels are corresponding to lower levels of details and accuracy while higher levels correspond to higher levels of details and precision.

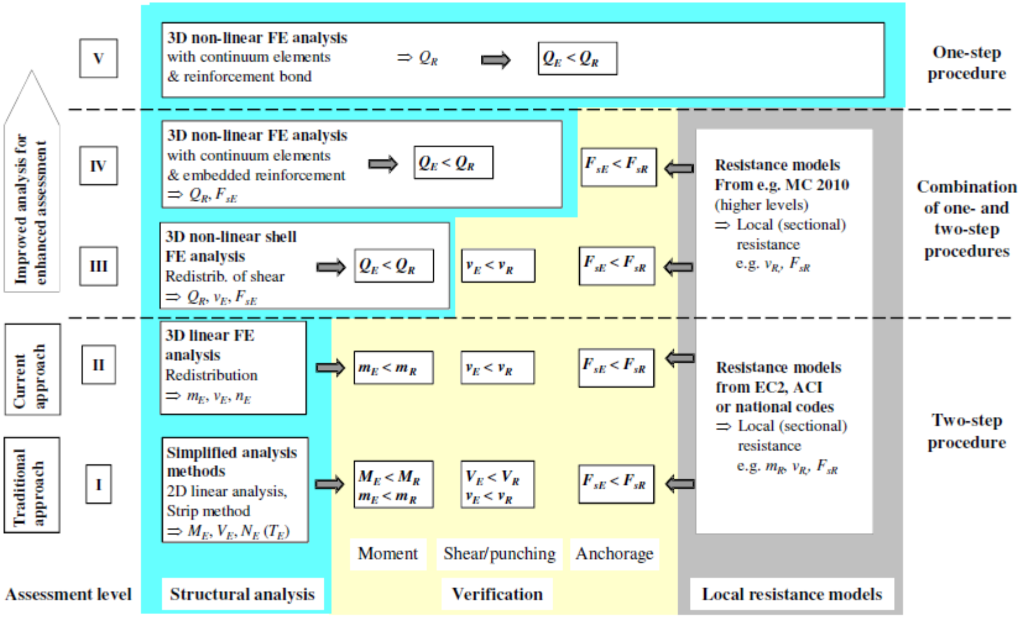

Figure 2.2. Scheme for multi-level structural assessment of RC Deck Slabs. From Plos *et al.* (2015).

2.2 Assessment of punching shear capacity

2.2.1 Introduction

When a concrete slab is subjected to a concentrated load larger than the slab capacity, the phenomenon is referred to as punching shear. The shear force per unit length may become high close to the area of loading. Punching shear failure may occur within the discontinuity regions if the ability of the slab punching shear is exceeded.

In 1985, Muttoni (Muttoni & Fernández Ruiz 2008) developed the concept of Critical Shear Crack Theory. The concept underwent development through experimental and theoretical studies in most countries. For the Swiss Code SIA 162 and fib Model code 2010 was based on the theory

2.2.2 Punching shear failure mechanism

A punching shear failure mechanism occurs when the compression region around the concentrated force fails as a result of the concrete strain attaining a critical level. The failure is as a result of combined action of both flexural and shear loading leading to a combination of flexural tangential and bending and inclined shear cracking. See *Figure 2.3* and *Figure 2.4* below. Flexural tangential and inclined shear cracking are designated tangential cracking.

From Hallgren in 1996 (Amir 2014), shear failure occurs when the tangential compression strain in the slab at the edge of the compressive load reaches a critical value. This weakens the concrete at the edge where the compressive load is acting until a critical value is reached leading to failure.

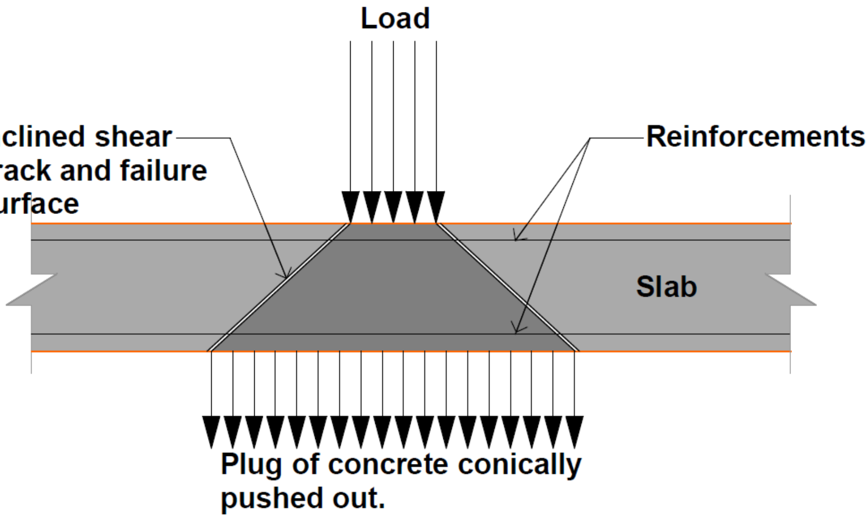

Figure 2.3 Punching shear failure section in concrete deck due to concentrated load

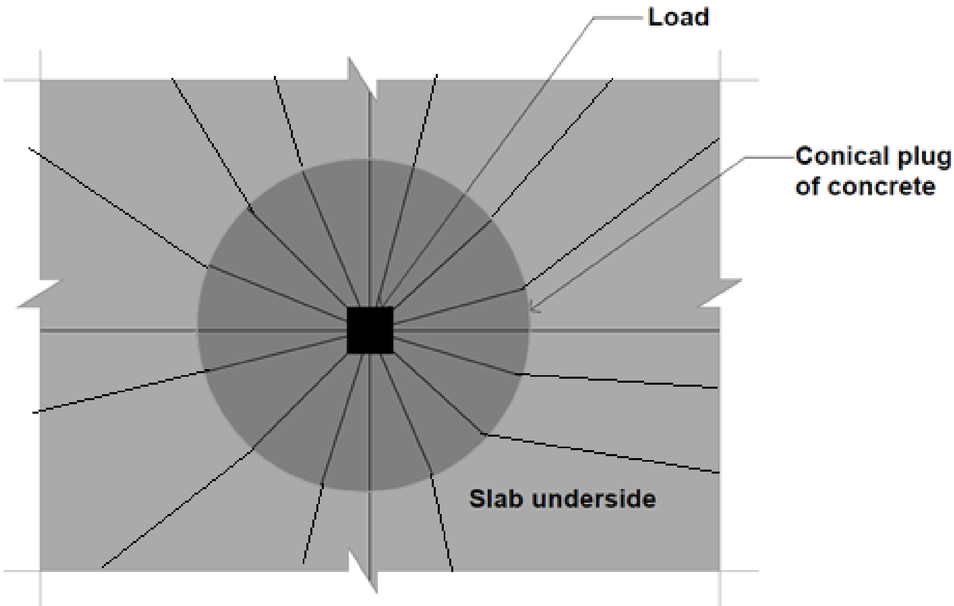

Figure 2.4 Punching shear failure section in concrete deck due to concentrated load

At low load levels, flexural cracks develop at the bottom of the slab directly under the projection of the load, within and around the loading perimeter. With load increase, radial cracks, caused by tangential moments, spread out from the perimeter of the load projection, breaking the slab into a fan-like fashion. At further loading, shear cracking starts from tangential bending cracks and start building up a cone-like plug.

At higher loads, the inclined cracks extend towards the slab edges and appear around the load perimeter. As the load keeps increasing, the crack widths are found to increase with few new cracks and then failure occurs suddenly in a brittle manner. The concrete cone subsequently pushes out of the slab at the ultimate punching load. For reinforced concrete slabs, the reinforcement acts as a hanger for the push out conical part, thus preventing complete removal of the concrete plug from the slab (Vaz Rodrigues 2007).

2.2.3 Punching shear failure in bridge decks

In bridge deck slabs, the punching shear mechanism may not be truly symmetrical since the flow of inner forces is different from that observed in slab column specimens. The transverse spans are much smaller than the longitudinal spans. And the radial cracking lines shown in Figure 2.1b can be longer and sometimes may not even be visible on the underside of the slab depending on the aspect ratio. Bridge deck slabs also differ from the regular slab-column isolated specimens as comprehensive membrane action can develop in the former due to external lateral restraint.

2.2.4 Punching shear strength according to building codes

2.2.4.1 Eurocode 2: EN 1992-1-1:2005

The punching shear according to Eurocode 2(Uni En 2005) is determined according to section 6.4 of the code as shown below.

$$V_{Rd,c} = C_{Rd,c}k(100\rho_1 f_{ck})^{1/3} + k_1\sigma_1 \geq v_{min} + k_1\sigma_{cp}$$

Where,

$f_{ck} =$ is the characteristic concrete strength in MPa,

$k = 1 + \sqrt{\dfrac{200}{d}} \leq 2.0$ d in mm

$\rho_1 = \sqrt{\rho_{ly} . \rho_{lz}} \leq 0.02$

ρ_{ly}, ρ_{lz} is related to the bonded tension steel in $y-$ and $z-$ directions respectively. The values ρ_{ly}, ρ_{lz} to be calculated as mean values taking into consideration the width of the slab equal to the column width plus 3d each side

$$\sigma_{cp} \geq \frac{(\sigma_{cy} + \sigma_{cz})}{2}$$

Where,

σ_{cy}, σ_{cz} are the normal stresses in the critical sectionin y $-$ and z $-$ directions(Mpa positive if compression

$\sigma_{cy} = \frac{N_{Ed,y}}{A_{cy}}$ and $\sigma_{cz} = \frac{N_{Ed,z}}{A_{cz}}$

$N_{Edy}, N_{Ed,z}$ are the longitudinal forces across the full bay for internal columns and the longitudinal forces across the control section for edge columns. The force may be from a load or prestressing action.

A_c is the area of concrete according to the definition of N_{Ed}

The values of $C_{Rd,c}, v_{min}$ and $k_1$can be found in National annex for specific countries. However the general recommended values as per EN is;

$C_{Rd,c} = \frac{0.18}{\gamma_c}, k_1 = 0.1$

2.2.4.2 ACI: 318-11

The punching shear capacity according to American Concrete Institute, ACI 318-11 is given by section 6.4 of the code as shown below.

$$V_{r,ACI} = \left(\beta_p\sqrt{f_c{}'} + 0.36\sigma_{cp}\right)b_0 d + V_p \geq (v_{min} + k_1\sigma_{cp}$$

Where,

$\beta_p = 0.29\ or\ 0.083(\frac{\alpha_s}{b_0} + 1.5)$

$\alpha_s = 40$ for internal columns, 30 for edge and 20 for corner columns

$f_c{}' =$ Compressive cylinder strength of concrete and not exceeding 35Mpa

$\sigma_{cp} =$ Average prestressing in each direction

$b =$ length of control perimeter at $\frac{d}{2}$ from the face of the column.

3 The Tested Bridge in Kiruna

3.1 Description of the bridge

Based on the construction drawings provided, the bridge had five spans, cast in situ concrete and post-tensioned girders and a pre-stressed bridge deck. The bridge description is mainly based on the construction drawings obtained and photos taken before, during and after the full-scale testing.

Figure 3.1 View of the Kiruna Bridge (Photo Shu Jiangpeng 2014).

3.1.1 Geometry

The bridge had five spans with an overall length of 121.5meters. The span lengths were 18.00, 20.50, 29.35, 27.15 and 26.5 meters long with intermediate columns in between them. *Figure 3.2*

The bridge deck slab had a thickness that varied from 300 mm at the girder slab intersection to 220 mm at 1.00 meter from the girder. The girders had an overall height of 1923 with a width varying from 410 mm at the mid-span, to 650 mm at 4.00 metres from the intermediate support. It reduces to 550 mm at the Anchorage location of post-tensioned cables, two-fifth of span length west, of support three and three-tenths of span length east of support four.

At the extreme ends of the bridge decks are edge beams of width 300 x 300 mm at the end of 1450 mm cantilever slabs and spans the entire bridge length. The three girders are centrally positioned on the bridge at 5000 mm distance apart and spanning the entire length of the bridge with intermediate supports at the external girders.

In the longitudinal direction, the slab is supported by secondary beams that run across the slab from North to West, and this supports the spanning bridge deck slab that spans in the longitudinal directions.

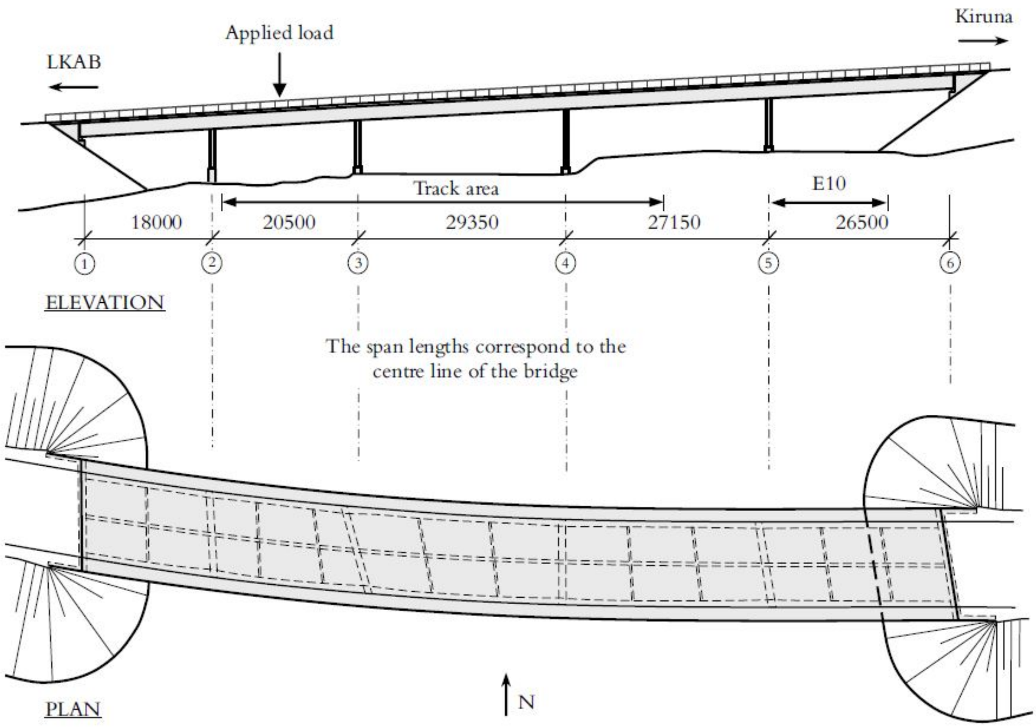

Figure 3.2 *Kiruna bridge elevation and plan adapted from* (Bagge & Blanksvärd n.d.).

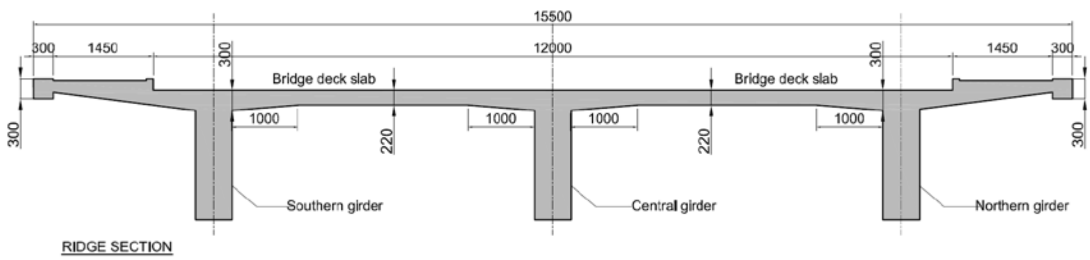

Figure 3.3 *Typical section through bridge deck slab*

3.1.2 Reinforcement

3.1.2.1 Girder reinforcement

The girders are reinforced with 3 16 mm diameter bars at the bottom, and 10 mm bars at the sides spaced at 150 mm for the central girder and 200 mm for the south and northern girders. The shear reinforcements consist of 10mm bars at 150 mm spacing. The concrete cover is 30 mm except for the 16mm reinforcement at 32 mm. The post-tensioned scheme was equipped using BBRV system and carried out in two stages. Six cables per girder were post-tensioned at each end of the central segment during the first phase. In the second phase, six cables per girder were post-tensioned from the free end of the western and eastern segments. Each cable composes of 32 wires with 6mm diameter.

3.1.2.2 Slab reinforcement

The slab reinforcement consisted of diameter 16 mm and 10 mm reinforcement at both top and bottom. The spacing of reinforcement ranged from 220 mm to 440 mm for both upper and lower longitudinal and transverse reinforcement. No shear reinforcement was provided according to the drawings and photos from the site. Due to the gradual change in thickness of the slab, part of the lower reinforcement at the girder support is bent and follows the slab profile.

3.1.3 Foundations

The foundations are of pile and founded on the rock beneath the ground. The abutments of the structure consist of reinforced concrete

3.1.4 Materials

The concrete and non-pre-stressing reinforcement have been tested by Luleå Technical University. Later, a more detail evaluation of the material was done at Luleå Technical University where a conventional material evaluation was compared to a statistical evaluation. To be able to compare the bridge response with the model, the same material properties used at Luleå Technical University have been used in this assessment. This is based on the assessment done by Lulea Technical University. Results from the bridge tests are shown in Table 3.1 and Table 3.2 below

Table 3.1 Material parameters for concrete used during the analysis

Young's Modulus E	Compressive strength, f_{cm} [MPa]	Tensile strength f_{ctm} [MPa]	Poison's ratio, v	Density σ [kg/m^3]
33	62.5	4.14	0.2	2400

Table 3.2 Material parameters for steel used during the analysis

	Young's Modulus,E, [GPa]	Yield strength f_m [MPa]	Ultimate strength f_{ck} [MPa]	*Poison's ratio, v*
Dia 10mm	200	557	-	0.3
Dia 16mm	200	584	-	0.3

4 Full scale test of the Bridge in Kiruna

4.1 Experimental procedures and load description

The Bridge in Kiruna, Sweden was subjected to an experimental investigation from May to September 2014. The bridge was constructed in 1959. The test programme encompassed tests on the bridge cables, the main girders and the bridge deck slab. The experimental program also included a test of two separate strengthening systems using carbon fibre reinforcing polymers. The polymers were attached to the lower side of the central and southern girders in span 2-3. The test experiments were performed in span 2-3 and accompanying monitoring in span 1-4(Bagge 2014)

4.1.1 Pre-loading

4.1.1.1 Loading Equipment

The loading was applied with the following equipment

- Two welded steel beams of 700 mm x 1180 mm x 5660 mm and 700 mm x 1180 mm x 6940 mm outside dimensions of steel grade s355J0
- Hydraulic Jacks, four in number, had cables arranged to pass through drilled holes in the bridge slab. The cables were anchored underneath the bridge to the rock at the bottom, a distance of 14.6metres below the bridge deck
- 700 x 700 mm^2 Steel Load Distribution plates of grade s275JR. The thickness of the plates ranges from 20 to 265 mm. The varying thickness was to cater for the inclined bridge deck slab
- The distance from the center of the Jacks and cables to the center of the transverse steel beam were 885 mm.
- The Jack had a capacity of 7.0 MN, with a 150 mm stroke length.

4.1.1.2 Pre-Loading Process

The failure loads of the bridge elements(Bagge 2014)were first preliminary estimated using preliminary calculation from Eurocode before the test enabling the failure load to be considered during the testing. The load-deformation format was applied. The loads on the bridge were applied using hydraulic jacks first to the girders before rotating it through 90 degrees to the deck slab. The load and deformation during the experiment were recorded for analysis and evaluation.

4.1.2 Bridge girder failure test

A total load of 12.0 MN load. 4.0 MN and 2.0MN were applied to the girder (Bagge & Blanksvärd n.d.) using the outer and inner Jack respectively. The set up followed the one during the preloading test. The loading was applied by increasing the pressures in the jacks until failure was reached in the girder. During the experiment, excessive deflections above the stroke length were always monitored and controlled by always adjusting the jack’s grip position.

4.1.3 Bridge slab failure test

The bridge slab between span 2 and 3 was tested by rotating the steel beam that was used previously for testing the girder by 90degrees. The load from the hydraulic jack was applied, through the load distributed to the slab using the load distribution steel plates. A horizontal concrete surface was also cast locally under the plates. The loading was deformation controlled, and the reaction forces were measured using strain gauges.

Figure 4.1 below, shows an experimental setup model for the slab test. The bridge deck was loaded to failure. A force controlled hydraulic Jack located in the steel beam mid-span was used to apply the load, through the steel beam and the two steel distribution plates. The load distribution plates were spaced at 2.0metres centres apart and located 470 mm and 330 mm on the inner side of the main northern girder (880 mm to the outside).

The girders and the slab deformation were monitored during the test during the hydraulic Jack force application. Draw wire sensors were instrumented in the mid-span of the girder for deflection measurements. When loading was taking place, corresponding deflection measurements were also being done simultaneously at loading plate positions and mid-spans between and adjacent to the plates using Draw wire sensors and LVDTs.

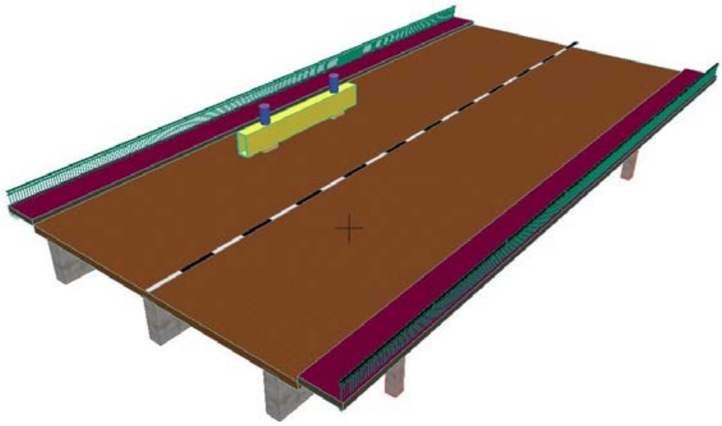

Figure 4.1Bride 3D model including the load distribution beam plate shown in yellow

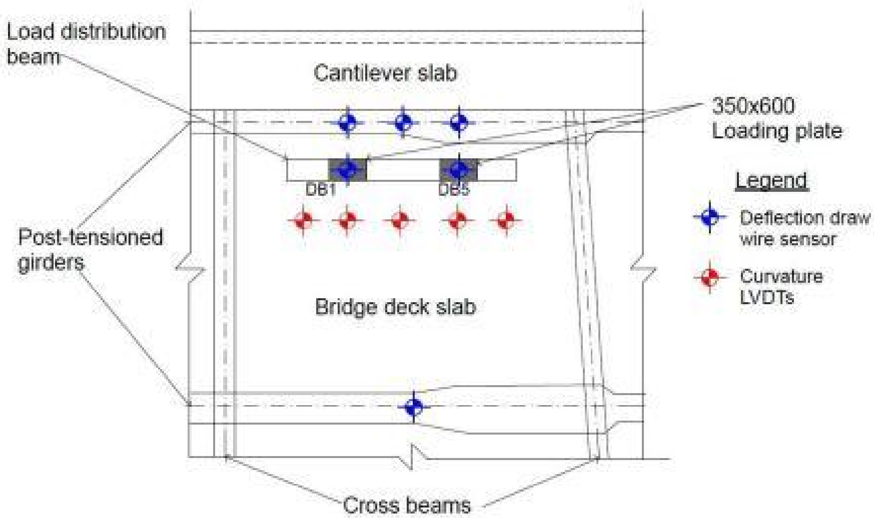

Figure 4.2. Bridge deck slab plan showing the slab full-scale test arrangement

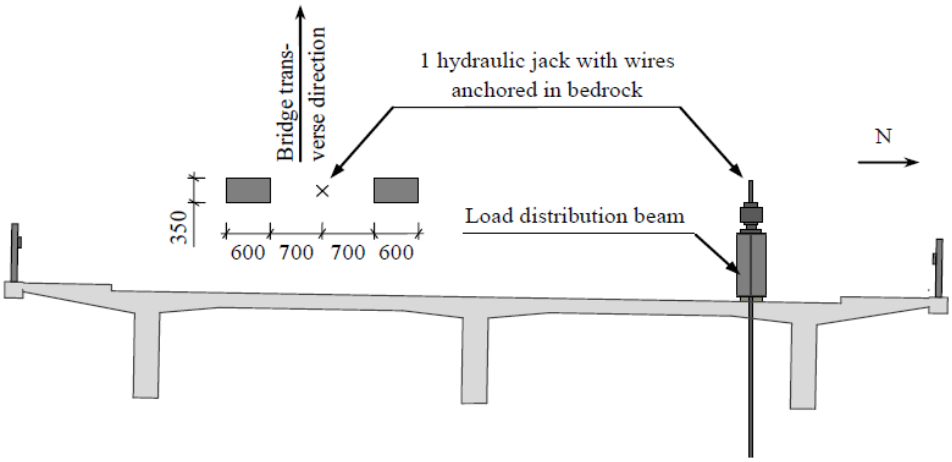

Figure 4.3. Bridge section showing the slab full-scale test arrangement. On the left is the loading plate plan. Adopted from (Bagge 2014)

The material properties of the bridge deck slab were determined through material testing. In-depth details about the reinforcement arrangements were determined using the available drawings and observations during testing.

4.2 Results and evaluation of the bridge slab failure test

During the experiment, the bridge deck slab was loaded to failure. See Figure 1 for the setup. The load applied was from the hydraulic Jack through two steel plates used for load distributing. The steel plates were 2.0m apart and were of dimensions 350x600 and located 470 and 330 mm on the inner side of the main northern girder (880 mm to the outer side). The girders and the slab were monitored during the test (Bagge et al., 2014).

Draw-wire sensors were set up at the mid-span of the north (D1) and Central (D2) girder to measure deflections. At the same time, deflections of the slab were measured underneath each loading plate (D3 and D4) and at the corresponding position of the northern girder (D5 and D6. At 500 and 1000 mm south of the center line through the loading points, the curvature was measured along the bridge longitudinal direction. Each curvature rig was composed by simply supported steel beams (length 4.82 and 5.08 mm) and five linear displacement sensors with 800 mm spacing. The midpoints of the curvature rigs coincided with the mid-span. The instrumentation is described more in detail in (Bagge et al., 2014).

From the test results recorded, a plot of the load against deflection was made to determine the behavior of the slab during loading. Figure 4.5. The graph indicated that the slab failed a total applied load of 3.32MN on both plates. At initial loading, no displacement response was registered until a load value of approximately 500 kN where a deflection increase appeared in both the draw wire sensors and the LVDTs. At a load of 2600kN, a temporary unloading was made before the slab was loaded to failure.

For the draw wire sensors, the maximum deflection during failure was recorded at DB11 at the left loading plate and the least at DB8 on the girder. The corresponding deflection value is 16.38 mm and 38.35 mm respectively (Bagge et al., 2014).

For the LVDT results, the maximum deflection during failure has been recorded at CB2 which is next to the loading plate while the least is at CB 2

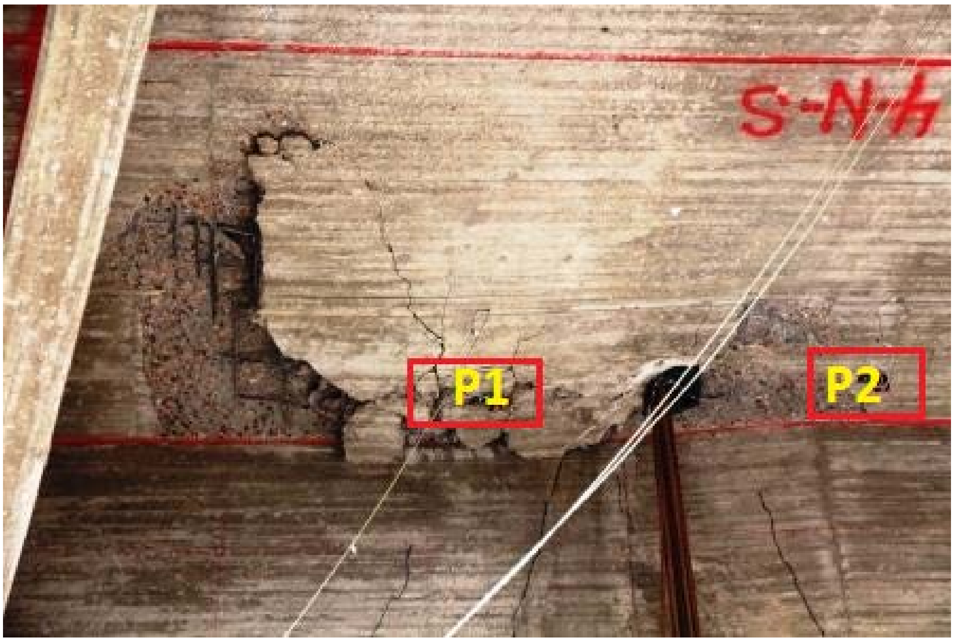

Figure 4.4. The failure pattern during the Deck slab loading test. P1 and P2 are the left and right loading plate respectively.

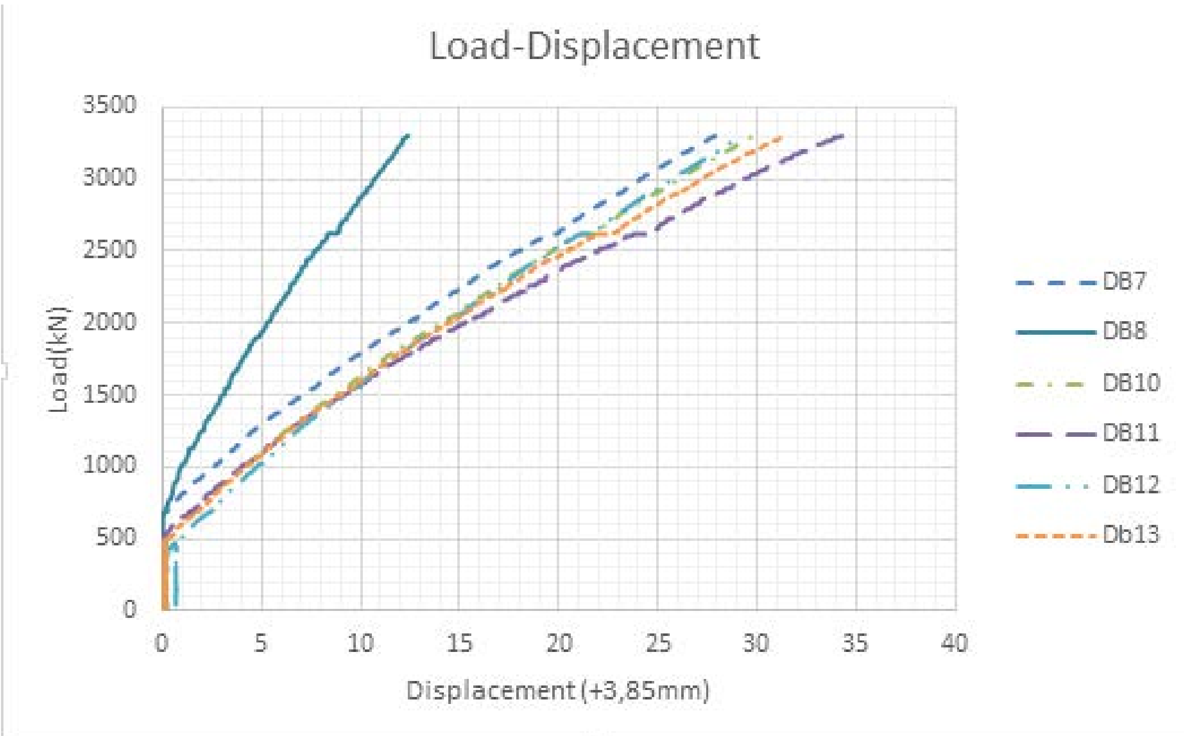

Figure 4.6 Load deflection plot for draw wire sensor

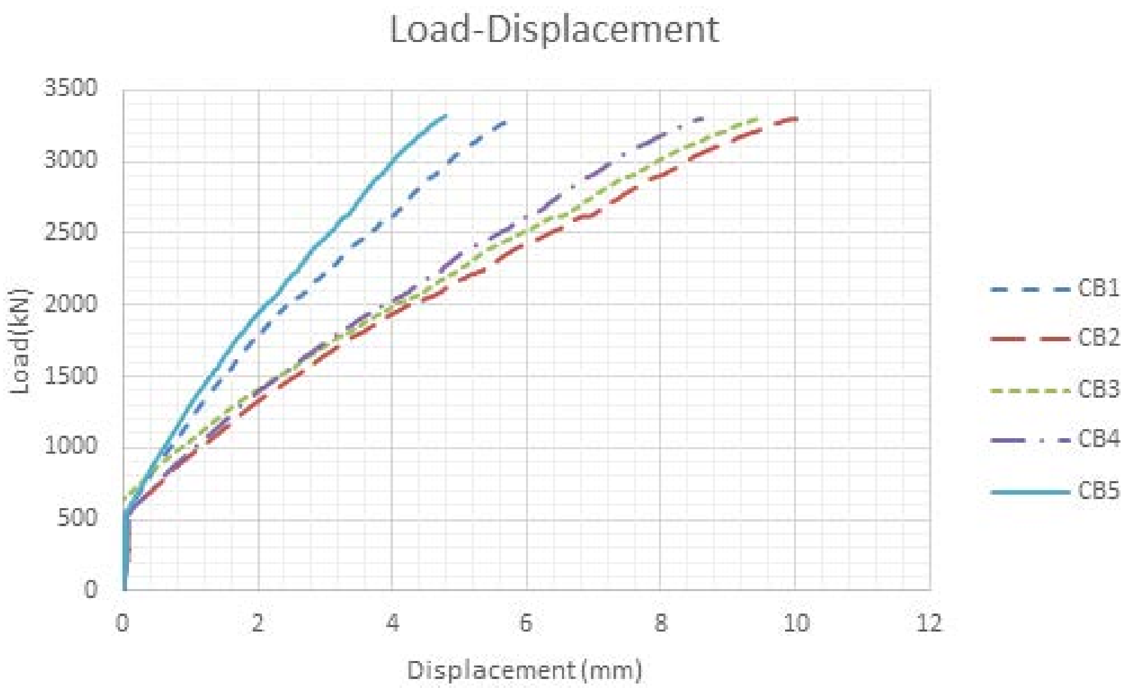

Figure 4.7. Load deflection graph for LVDT results

5 Bridge Assessment and analysis

To analyse the response of the bridge deck slab in relation to the test, the slab was investigated with both linear and non-linear analysis following the proposals described in the Multi-level structural analysis strategy (Shu et al. 2016). In Level I, the analysis was made using simplified resistance models from Euro code 2. The bending capacity of the slab was also checked using yield line method. The capoacity was not checked in level II since this was not expected to enhance the analysis results for the current case. In level III the critical crack method was employed in combination with finite element analysis with the softwares TNO Diana and Midas FX+ (Diana 2008). Analysis on Level IV and Level V was not included in this study.

5.1 Level I, simplified analysis

On the initial level for assessment, the load carrying capacity with respect to bending failure was estimated using the strip method(Hillerborg, 1996) for a lower bound value and the yield line method(Johansen, 1972) for an upper bound value; See Figure 5.1. The failure load with respect to one way shear and punching were checked according to euro code 2 (EN 1992-1-1, 2004). The results of the calculations are displayed in Table 5.1. The anchorage was also checked according to Eurocode 2(EN 1992-1-1, 2004) and proven to be non-critical.

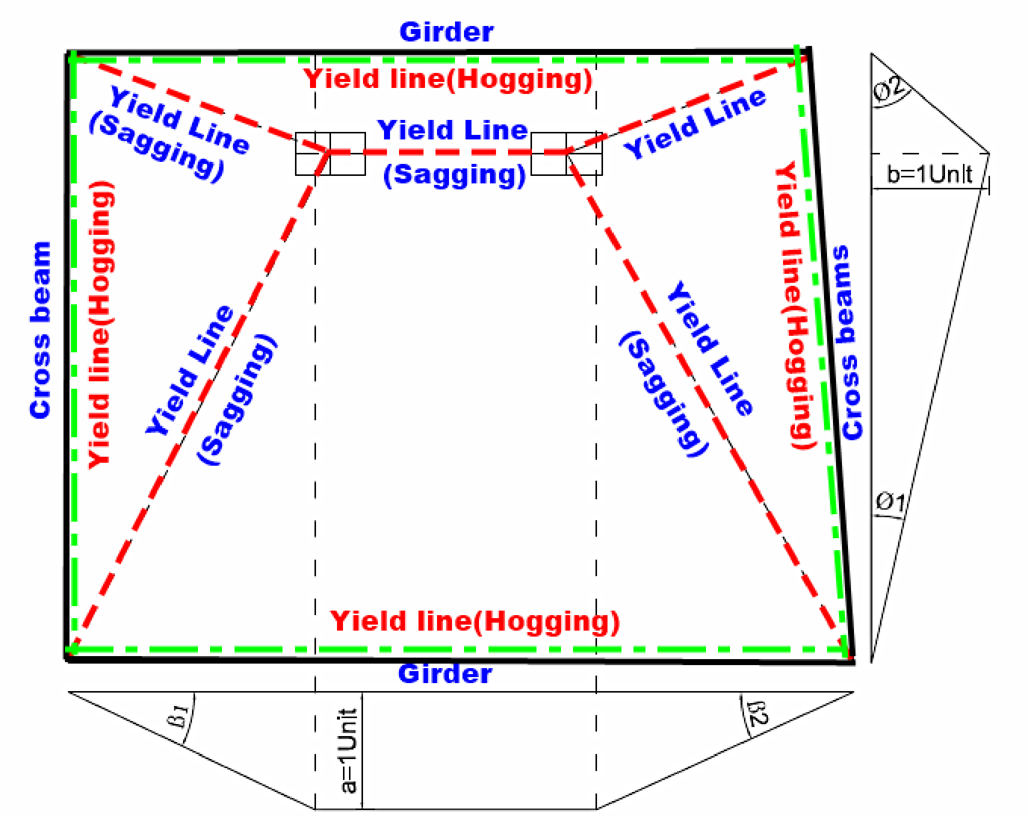

Figure 5.1 Yield line model used for bending capacity. The figure shows the loaded part of the bridge deck slab between longitudinal and transversal beams. Yield lines were assumed according to the figure.

Table 5.1 Multi-level assessment, level 1 results.

Resistance with respect to;	*Load carrying capacity,* Q_u [kN]
Bending capacity(Yield line method)	1006kN
Punching shear	888.89kN

5.2 Level II, linear 3D analysis

Since the punching capacity was found to be critical for the load carrying capacity of the slab, no analysis on Level II, based on 3D Linear shell FE analysis, was made. The resistance model for punching referred to in (reference to the Multi-level assessment strategy) is the same for level I and II, and an enhanced assessment would therefore not result in higher capacity.

5.3 Level III, non-linear 3D shell analysis

In level III, a combination of non-linear finite element analysis and the critical crack theory was used(Muttoni 2009). In the finite element analysis, TNO Diana (Diana 2008) was used to model the slab with shell element, see *Figure 5.4* below. In the critical crack theory, a criterion proposed by Muttoni which is based on the rotation of the slab during punching as a result of the load application was used.

5.3.1 Non-linear shell FE analysis

A FE model of the tested part of the bridge deck slab was made. Only the part of the slab in between adjacent longitudinal and transversal main beams was included. It was assumed that the boundary condition at the slab edges were fully restrained.

The steel reinforcement of the deck slab panels was modelled as a reinforcement grid embedded in the shell elements. In each layer, a thickness corresponding to the reinforcement areas shown in the drawings was provided in directions perpendicular to each other. A von Misses yield criterion was used for modelling the reinforcement material response. And initial yield strength equivalent to the average of the two determined during the test was used in the model.

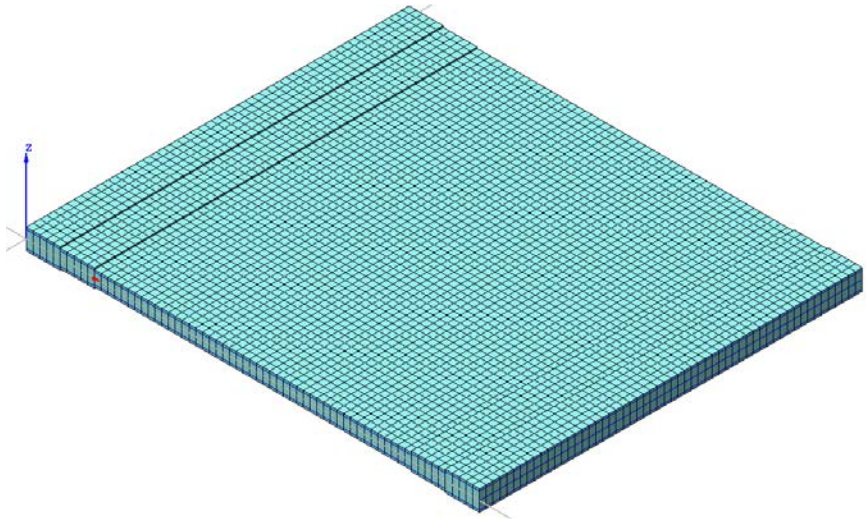

Figure 5.2 Shell element model with 100 mm mesh size

The modelling of the bridge was made in the pre and post processor FX+. The model files was then exported to TNO Diana for FE analysis before retrieved back to FX+ for extraction of results. A Simpson integration scheme with nine integration points over the thickness and 2 x 2 integration points over the shell area was used. The concrete was modelled with a total strain rotating crack model based on fracture energy. A concrete fracture energy of 154 N/m was used. In tension, an exponential softening function according to Hordyk was used as pre-defined in Diana. Refer to chapter 3 for material details.

The concrete compressive behaviour was modelled using the Thorenfeldt stress-strain response. This response was originally determined on 300 mm high specimen. In the current analysis, the Thorenfeldt response was modified to allow for 100 mm element sizes, as used in the model. The curve was modified according to Figure 5.4

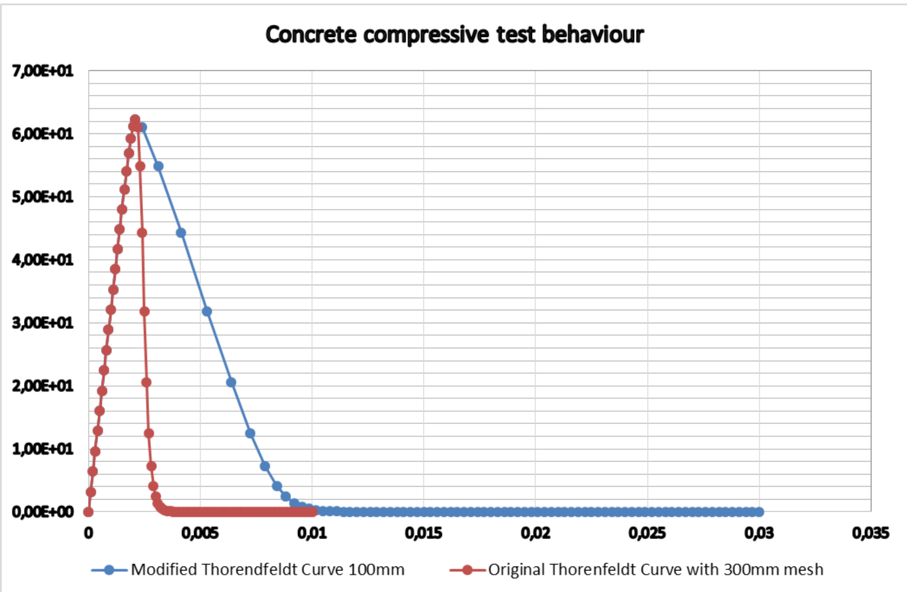

Figure 5.3. Modified stress-strain curve; 300 mesh size (Red), 100mm size (Maroon).

The reinforcement was modelled as grid layers. The total reinforcement area for the reinforcement in a particular section was calculated from the drawings and its equivalent thickness over a unit width was computed. The Von Mises plasticity model was used. The various input parameters for the steel properties (*Table 3.1* and *Table 3.2*) was obtained from the test and used for the analysis.

Figure 5.5 below shows the displacements in the model. The force output corresponding to the load step values were plotted against the corresponding displacement at the loading plates, see *Figure 5.6*.

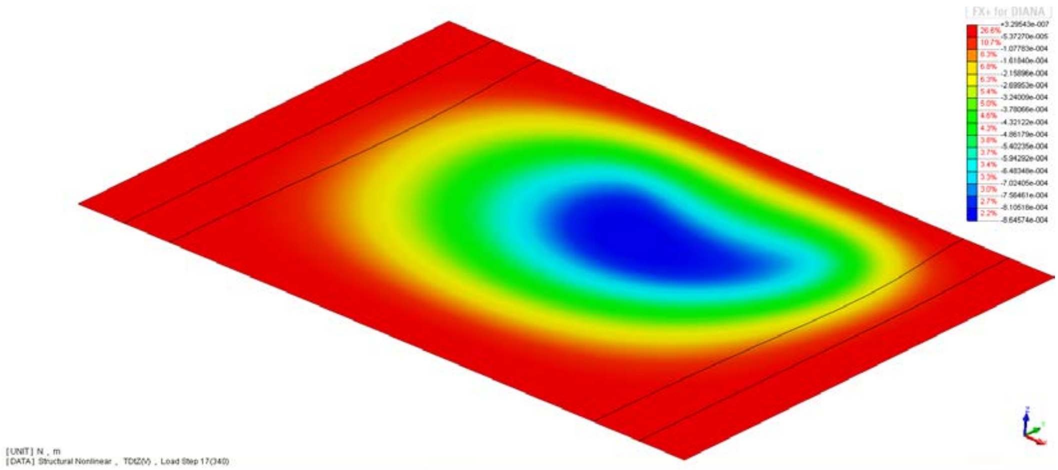

Figure 5.4 Typical figure of deflection at load step 17

5.3.2 Critical crack theory

According to the critical crack theory the punching shear strength depends on the rotation of the slab(Muttoni 2009). The load-rotation relation obtained from a non-linear analysis reflecting the bending response of the slab is compared to a rotation dependent failure criterion for punching failure of the slab. The load Q is made dimensionless:

$$Q = \frac{V_R}{b_o d\sqrt{f_c}}$$

Where

$V_R = applied\ shear\ force\ on\ a\ dimensionless\ form.$
$b_o = the\ distribution\ width$
$d = Average\ effective\ depth\ of\ concrete$
$f_c = Mean\ concrete\ compressive\ strength.$

The punching shear strength is determined as a function of the rotation of the slab:

$$Q = \frac{3/4}{1+15\frac{\omega d}{d_{go}+d_g}},$$

Where

$\omega = slab\ rotion\ at\ the\ point\ of\ load\ application\ and\ edge$
$d_{go} = Reference\ agregate\ size = 16mm$
$d_g = Maximum\ agregate\ size$

The intersection of the two graphs, when plotted in the same diagram, will give the punching shear capacity at their intersection.

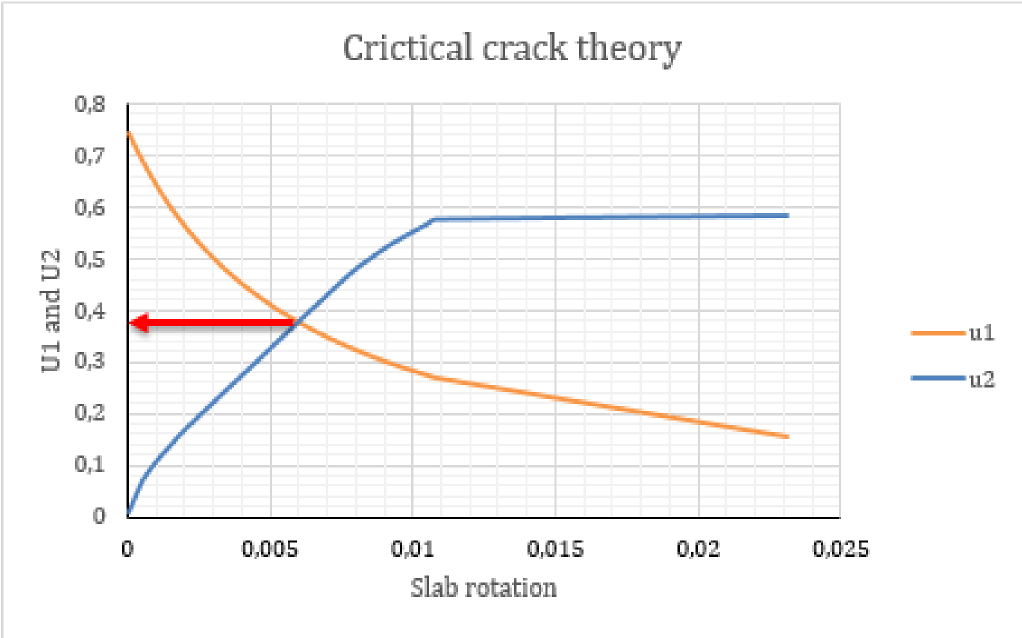

Figure 5.5. Failure load criterion

From the graph above, the punching shear capacity of the slab can be determined to be 2431kN. Table 5.2 below shows a summary of the critical shear crack theory results for Level III

Table 5.2 Multi-level assessment, level 3 results.

Resistance with respect to;	*Load carrying capacity, Q_u* [kN]
Bending capacity(Yield line method)	1006 kN
Punching shear capacity (Level 1)	888.89 kN
Punching shear capacity(Critical Crack Theory)	2431 kN

6 Conclusions

The purpose of this study was to determine the punching shear capacity of tested reinforced concrete bridge deck slab in Kiruna, Sweden. A full-scale punching test to failure was conducted at a reinforced concrete bridge in Kiruna Sweden. The multi-level structural assessment was carried out up to level III, the 3D Non-Linear Shell Finite Element analysis. A punching capacity of approximately 70.5% and 26.8% respectively of the test results was achieved using the design resistance according to the Eurocode 2 and the 3D Non-Linear Shall Finite Element Analysis respectively.

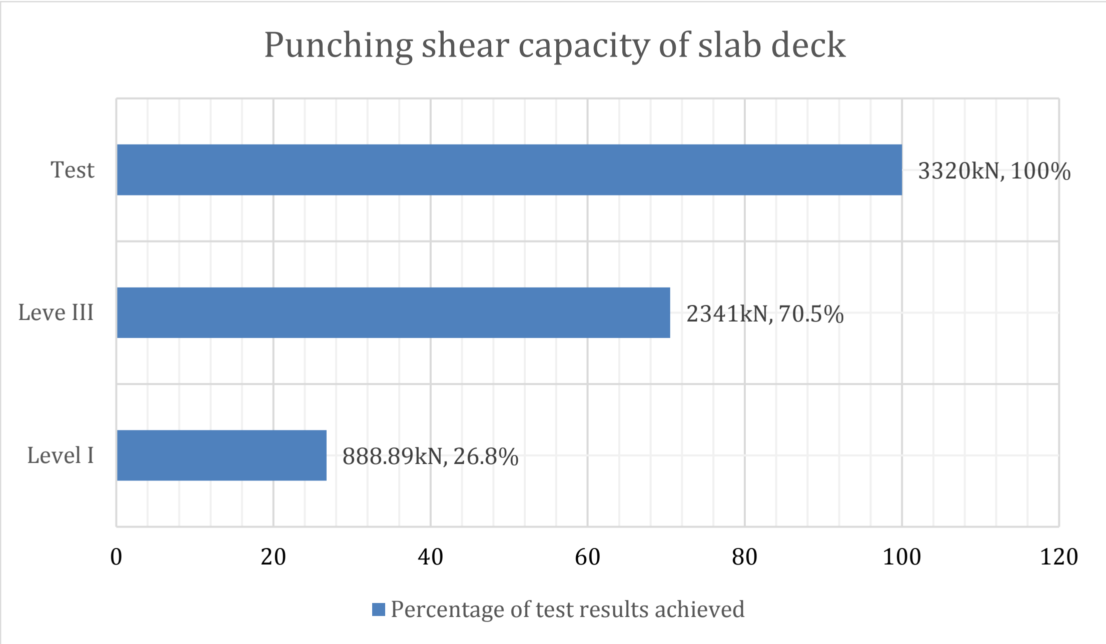

Figure 6.1. Punching shear capacity summary for the bridge deck slab.

7 Further studies

The structural assessment performed in this study included only assessment levels up to level III, according to the multi-level structural assessment strategy proposed by (reference). More refined assessment on level IV and V with 3D non-linear FE analysis with continuum elements, with embedded reinforcement and separate reinforcement elements with bond-slip connection to the concrete would have been interesting to include.

The reinforcement yield strength obtained through material tests for diameter 10 mm and diameter 16 mm were of different values. However during modelling for analysis in level III, the average of the reinforcement strengths were used. In Further studies it is preferable to assign the two different types of reinforcement separate yield strengths.

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