# CFRP strengthening of a reinforced concrete bridge

## - A case study done with Eurocode from an economic perspective


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# Strengthening of a reinforced concrete bridge
# - A case study done with Eurocode from an economic perspective

Kolfiberförstärkning av en armerad betongbro
- En fallstudie gjord med Eurocode ur ett ekonomiskt perspektiv

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# Sammanfattning


Syftet med examensarbetet är att analysera några vanliga förstärkningssystem med kolfiber för en befintlig vägbro. Analysen görs ur ett ekonomiskt perspektiv där olika kolfiberalternativ jämförs. Syftet är också att jämföra tillvägagångssätt och resultat av klassning samt förstärkningsbehov för den befintliga bron, enligt gamla och nuvarande svenska normer. Examensarbetet är teoretiskt och baseras på litteraturstudier och beräkningar för den befintliga bron. Resultaten från förstärkningsberäkningarna analyseras ur ett ekonomiskt perspektiv där material- och arbetskostnad för de möjliga förstärknings-alternativen uppskattas.

Den studerade bron är en armerad betongbro, som enligt Trafikverket är bevarandevärd. Klassning och förstärkning av bron har redan genomförts enligt "Allmän teknisk beskrivning för Klassningsberäkning av vägbroar" (Klassning 1998) respektive Bro2004, då bron skulle uppgraderas från Bärighetsklass 3 (BK3) till Bärighetsklass 1 (BK1). I examensarbetet har klassningen gjorts enligt "Metodbeskrivning 802 Bärighetsutredning av byggnadsverk" (Klassning 2009) och förstärkningsbehovet beräknats enligt TK Bro, d.v.s. enligt de nya normerna. Resultaten från Klassning 1998 har sedan jämförts med resultaten från Klassning 2009 och resultaten från Bro2004 har jämförts med resultaten från TK Bro. Förstärkning i böjstyvhet har analyserats för alternativ med kolfibersystemen laminat, väv, samt stavar. För tvärkraft har analysen enbart gjorts för alternativ med väv. De egenskaper som har varierats är kolfiberns elasticitetsmodul, töjning och tvärsnittsdimensioner, samt för tvärkraft även avstånd mellan remsor och lutning på fibrer.

Den enda skillnaden mellan klassningarna gjorda enligt Klassning 2009 och Klassning 1998 är två extra typfordon i Klassning 2009. Resultaten från klassningarna tyder på ett högre förstärkningsbehov för böjstyvhet, men ett likvärdigt förstärkningsbehov för tvärkraft, för fallstudien. Den kapacitetsanalys för böjstyvhet som är gjord enligt TK Bro resulterade i ett betydligt lägre förstärkningsbehov än för analysen gjord enligt Bro2004. Detta beror framförallt på de olika beräkningsfilosofierna gällande säkerhetsklassning. Kapacitetsanalysen för tvärkraft gjord enligt TK Bro resulterade i ett betydligt högre förstärkningsbehov än för Bro2004. Detta beror framförallt på att en mer konservativ beräkningsmodell används i TK Bro.

Kostnaderna för alternativen med laminat och väv ökar med en ökande elasticitetsmodul för både förstärkning i böjstyvhet och tvärkraft. För böjstyvhet är alternativ med väv generellt billigast. Det fungerande alternativet med stavar är billigare än laminaten och vissa alternativ med väv. Förankringslängden har ingen större inverkan på kostnaden, då alla alternativen kräver ungefär samma förankringslängd. Det är inte mer effektivt att applicera remsorna i 45° istället för 90°.

För den studerade bron skulle det utifrån gällande normer och ekonomiska uppskattningar rekommenderats att förstärkningen gjorts med väv och mekanisk förankring för både böjstyvhet och tvärkraft.



Nyckelord: Kolfiber, Betong, Förstärkning, Eurocode, TK Bro, Ekonomisk optimering

# Abstract

The purpose with this thesis is to analyze some common strengthening systems with CFRP for an existing road bridge. The analysis is done from an economic perspective, considering different material properties of the CFRP. The purpose is also to compare the procedures and results for the classification assessments and strengthening design of the existing bridge, according to old and present Swedish codes. The thesis is theoretic and based on literature studies and strengthening calculations for the case study. The result of the strengthening calculations is analyzed from an economic perspective where the costs of material and labor are estimated for different possible strengthening alternatives.

The studied bridge is a reinforced concrete bridge, which according to Trafikverket is important to preserve from a cultural historical perspective. The classification and strengthening has been performed in agreement with "Allmän teknisk beskrivning för Klassningsberäkning av vägbroar" (Classification 1998) and Bro2004, since the bridge had to be upgraded from capacity class 3 (BK3) to capacity class 1 (BK1). In the thesis, the calculations have been done according to Classification 2009 and TK Bro, according to the present codes. The results for the present codes have then been compared to the results for the old codes. The strengthening in flexure has been analyzed for alternatives with the CFRP systems; laminates, sheets and bars (NSMR). For shear, the analysis has been carried out only for alternatives with sheets. The properties of the CFRP that have been altered are the strain, moduli of elasticity, and cross-sectional dimensions. For shear, the spacing of the strips and the inclination of the fibres have also been altered.

The only difference for the classification assessments done in agreement with Classification 2009 and Classification 1998 is the additional design trucks in Classification 2009. The results of the classification assessments indicate a higher required flexural strengthening, but an essentially equal required shear strengthening. The capacity analysis in flexure, done according to TK Bro resulted in a significantly lower required strengthening than the analysis done according to Bro2004. The difference does mostly depend on the different design philosophies considering the safety classification. The calculation done with TK Bro resulted in a significantly higher required shear strengthening than the calculation done with Bro2004. This is mainly because TK Bro utilizes a more conservative model for shear.

The cost of the alternatives with sheets and laminates increases with the modulus of elasticity. The results show that generally the alternatives with sheets are cheaper. The alternative with bars is cheap compared to the laminates, and some of the alternatives with sheets. The anchorage length for the alternatives in flexure does not have a significant effect on the cost, since the variation of the anchorage lengths is small. It is not more effective to apply the strips with an inclination of 45° instead of 90°.

For the studied bridge, with regards to the present codes and the economical estimations, it would have been recommended to strengthen the bridge with sheets and mechanical anchorage in both flexure and shear.



Keywords: CFRP, Concrete, Strengthening, Eurocode, TK Bro, Economical optimization

# Preface

This thesis was completed in collaboration with Vectura Consulting AB and Sto Scandinavia AB, under the administration of the Division of Structural Engineering at the Faculty of Engineering, Lund University. The thesis was carried out during the fall of 2010 and was completed in February 2011.

We would like to thank our supervisors Dr. Tobias Larsson at Vectura Consulting AB, Dr. Thomas Blanksvärd at Sto Scandinavia AB, and Professor Sven Thelandersson at University of Lund for their support. We would also like to thank Peter Dalerstedt, Magnus Brommesson and Kent Kempengren for the help they have provided.

Malmö, February 2011
Emma Persson and Sofie Stigsson

## Notations

| | |
|---|---|
| $A_{allowed}$ | Allowed axle load for case $[kN]$ |
| $A_c$ | Cross-sectional area of the concrete $[mm^2]$ |
| $A_f$ | Cross-sectional area of CFRP $[mm^2]$ |
| $A_s$ | Cross-sectional area of the tensile steel reinforcement $[mm^2]$ |
| $A_{sw}$ | Cross-sectional area of shear reinforcement $[mm^2]$ |
| $A_{s0}$ | Minimum tensile reinforcement area $[mm^2]$ |
| $B_{allowed}$ | Allowed bogie load for case $[kN]$ |
| $E_a$ | Modulus of elasticity of the adhesive $[MPa]$ |
| $E_{cd}$ | Design value of modulus of elasticity of concrete $[MPa]$ |
| $E_{ck}$ | Characteristic value of the modulus of elasticity of concrete $[MPa]$ |
| $E_{cm}$ | Secant value of the modulus of elasticity of concrete $[MPa]$ |
| $E_d$ | Design value of the load $[kN]$ |
| $E_f$ | Design value of modulus of elasticity of CFRP $[MPa]$ |
| $E_s$ | Design value of the modulus of elasticity of tensile steel reinforcement $[MPa]$ |
| $F$ | Tensile force in the tensile steel reinforcement $[kN]$ |
| $F_{adj}$ | Adjusted tensile force in the tensile steel reinforcement $[kN]$ |
| $F_c$ | Compressive force in concrete $[kN]$ |
| $F_f$ | Tensile strength in the CFRP $[kN]$ |
| $F_{f,a}$ | Tensile force at the position of anchorage of the CFRP $[kN]$ |
| $F_{f,b}$ | Tensile force in the position of anchorage of the CFRP $[kN]$ |
| $F_{f,e}$ | Maximum effective tensile force in CFRP $[kN]$ |
| $F_{red}$ | Reduced load within the distance of $3d$ from the system line $[kN]$ |
| $F_s$ | Yield force in the tensile steel reinforcement $[kN]$ |
| $\Delta F$ | Required flexural strengthening force $[kN]$ |
| $G_a$ | Shear modulus of adhesive $[GPa]$ |
| $G_f$ | Fracture energy of concrete in the bonded zone $[MPa]$ |
| $G_{k,j}$ | Characteristic value of the total permanent load $[kN]$ |
| $G_{k,p}$ | Characteristic value of permanent load from the pavement $[kN]$ |
| $I_1$ | Moment of inertia of transformed uncracked cross-section $[mm^4]$ |
| $I_2$ | Moment of inertia of transformed cracked cross-section $[mm^4]$ |
| $L$ | Total length of the bridge $[m]$ |
| $L_{beam}$ | Length of beam $[mm]$ |
| $L_e$ | Minimum bond length for an NSMR bar $[mm]$ |
| $L_{ef}$ | Effective anchorage length $[mm]$ |
| $L_{per}$ | Perimeter of potential failure plane for an NSMR bar $[mm]$ |
| $L_{\Delta F}$ | Length that requires flexural strengthening $[mm]$ |
| $M_d$ | Design value of the moment in ULS $[kNm]$ |
| $M_{d0}$ | Design value of the moment during the strengthening $[kNm]$ |
| $M_x$ | Design value of the moment at position $x$ $[kNm]$ |
| $M_{xa}$ | Design value of the shifted moment at the position of anchorage $[kNm]$ |
| $M_{xcr}$ | Moment at the position of the last crack $[kNm]$ |
| $N_{Ed}$ | Design value of the normal force $[kN]$ |
| $N_d$ | Design value of the normal force in ULS $[kN]$ |
| $N_x$ | Design value of the normal force at position $x$ $[kN]$ |
| $N_{xa}$ | Design value of the shifted normal force at the position of anchorage $[kN]$ |
| $P_{eff}$ | Corresponding point load $[kN]$ |

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| $P_{(\max)L}$ | Maximum bond strength of NSMR $[kN]$ |
| $Q_{1k}$ | Value of vertical axle load $[kN]$ |
| $Q_{k,1}$ | Characteristic value of the main live load $[kN]$ |
| $Q_{k,i}$ | Characteristic value of an interacting live load $[kN]$ |
| $Q_{lk}$ | Value of characteristic longitudinal forces (braking and acceleration forces) |
| $V_c$ | Shear capacity of concrete $[kN]$ |
| $V_{d,red}$ | The shear force calculated with reduced load $[kN]$ |
| $V_{d0}$ | The shear force for the actual load $[kN]$ |
| $V_{Ed}$ | The design shear in considered cross-section due to external loads $[kN]$ |
| $V_i$ | Varying effective depth $[kN]$ |
| $V_r$ | Required strengthening in shear $[kN]$ |
| $V_{Rd}$ | Design capacity of shear for members with shear reinforcement $[kN]$ |
| $V_{Rd,c}$ | Design capacity of shear for members without shear reinforcement $[kN]$ |
| $V_{Rd,f}$ | Contribution of the CFRP to the shear capacity $[kN]$ |
| $V_{Rd,max}$ | Maximum design capacity for the concrete compression struts $[kN]$ |
| $V_{Rd,s}$ | Design capacity of shear limited to yielding of shear reinforcement $[kN]$ |
| $V_s$ | Shear capacity of shear reinforcement $[kN]$ |
| $V_{Sd}$ | Total design value of shear load $[kN]$ |
| $V_{sw}$ | Design load of shear, for self-weight $[kN]$ |
| $V_{ss}$ | Design load of shear, for superstructure $[kN]$ |
| $V_{traffic\_A}$ | Design load of shear, for design truck with the axle load A $[kN]$ |
| $V_{traffic\_B}$ | Design load of shear, for design trucks with the bogie load B $[kN]$ |
| $W_c$ | Flexural resistance $[mm^3]$ |

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| $a$ | Distance from node to end of CFRP $[mm]$ |
| $a_d$ | The distance from the system line to where the wheel load is placed $[m]$ |
| $a_l$ | Displacement of tensile force curve $[mm]$ |
| $a_v$ | Distance from the end of the column to the wheel load $[m]$ |
| $b$ | Width of cross-section $[mm]$ |
| $b_f$ | Width of CFRP $[mm]$ |
| $b_g$ | Width of the groove for NSMR bar $[mm]$ |
| $c$ | Depth of cover $[mm]$ |
| $d$ | Effective depth of beam $[mm]$ |
| $d_{ef}$ | Available bond length $[mm]$ |
| $d_f$ | Height of shear strengthening above tensile steel reinforcement $[mm]$ |
| $f_{cd}$ | Design value of the compressive strength of concrete $[MPa]$ |
| $f_{ck}$ | Characteristic value of the compressive strength of concrete $[MPa]$ |
| $f_{ctd}$ | Design value of tensile strength of concrete $[MPa]$ |
| $f_{ctk}$ | Characteristic value of the tensile strength of concrete $[MPa]$ |
| $f_{ctm}$ | Average value of the tensile strength of concrete $[MPa]$ |
| $f_v$ | Shear strength of concrete $[MPa]$ |
| $f_{vr}$ | Increased value of the shear strength of concrete $[MPa]$ |
| $f_{yd}$ | Design value of yield strength of tensile steel reinforcement $[MPa]$ |
| $f_{yk}$ | Characteristic value of yield strength of tensile steel reinforcement $[MPa]$ |
| $f_{ywd}$ | Design value of yield strength of shear reinforcement $[MPa]$ |
| $f_{ywk}$ | Characteristic value of yield strength of shear reinforcement $[MPa]$ |

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| $h$ | Height of cross-section $[mm]$ |
| $h_f$ | Height of flange $[mm]$ |
| $k_1$ | Coefficient $[-]$ |
| $k_b$ | Size factor $[-]$ |
| $l_a$ | Anchorage length of CFRP $[mm]$ |
| $l_{cr}$ | Critical anchorage length of CFRP $[mm]$ |
| $l_e$ | Minimum bond length in flexure $[mm]$ |
| $l_{ef}$ | Minimum bond length in shear $[mm]$ |
| $l_{\mathrm{frp}}$ | Total length of CFRP in flexure $[mm]$ |
| $l_p$ | Half of the length of the CFRP in flexure $[mm]$ |
| $n$ | Number of layers (sheet or laminate) or number of NSMR bars $[-]$ |
| $q_{1k}$ | Value of uniformly distributed load $[kN/m]$ |
| $r$ | Spacing of the CFRP sheets $[mm]$ |
| $s$ | Spacing of the shear reinforcement units $[mm]$ |
| $s_a$ | Thickness of adhesive $[mm]$ |
| $s_f$ | Spacing of the sheets measured in a horizontal direction $[mm]$ |
| $t_f$ | Thickness of CFRP $[mm]$ |
| $t_g$ | Thickness of groove $[mm]$ |
| $\nu_{min}$ | Factor that considers the compressive strength of the concrete $[MPa]$ |
| $w_1$ | Width of one lane on a road bridge $[m]$ |
| $x_{cg}$ | Position of the center of gravity in strengthened cross-section $[mm]$ |
| $x_{cg0}$ | Position of the center of gravity in unstrengthened cross-section $[mm]$ |
| $x_{cr}$ | Position of the last crack along the beam $[mm]$ |
| $x_{N.A.}$ | Position of the neutral axis in unstrengthened cross-section $[mm]$ |
| $z$ | Inner lever arm of tensile steel reinforcement $[mm]$ |
| $z_0$ | Inner lever arm of CFRP at the anchorage point $[mm]$ |
| $z_f$ | Inner lever arm of CFRP $[mm]$ |
| | |
| $\alpha$ | Angle of stirrup with respect to the member axis $[rad]$ |
| $\alpha_{cc}$ | Coefficient to consider long time effects for compressive concrete $[-]$ |
| $\alpha_{ct}$ | Coefficient to consider long time effects for tensile concrete $[-]$ |
| $\alpha_{cw}$ | Coefficient that regards the influences of possible compressive stresses $[-]$ |
| $\alpha_f$ | Proportionality factor of concrete and CFRP $[-]$ |
| $\alpha_s$ | Proportionality factor of concrete and tensile steel reinforcement $[-]$ |
| $\alpha_{Q1}$ | Adjustment factor for some load models $[-]$ |
| $\alpha_{q1}$ | Adjustment factor for some load models $[-]$ |
| $\beta$ | Inclination of fibres $[rad]$ |
| $\beta_{red}$ | Reduction factor for shear $[-]$ |
| $\delta_f$ | Interfacial bond-slip at final fracture $[mm]$ |
| $\varepsilon$ | Dynamic contribution [%] |
| $\varepsilon_c$ | Compressive strain in concrete for strengthened cross-section [%] |
| $\varepsilon_{cbot}$ | Tensile strain in concrete for unstrengthened cross-section [%] |
| $\varepsilon_{cu}$ | Characteristic ultimate limit strain in concrete [%] |
| $\varepsilon_f$ | Design value of tensile strain in CFRP [%] |
| $\varepsilon_{fb,d}$ | Tensile strain in CFRP due to debonding [%] |
| $\varepsilon_{fd,ic}$ | Tensile strain in CFRP due to risk of intermediate cracking [%] |

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|---|---|
| $\varepsilon_{fu}$ | Design value of ultimate strain in CFRP [%] |
| $\varepsilon_{fuk}$ | Characteristic ultimate strain in CFRP [%] |
| $\varepsilon_{fx}$ | Maximum allowed tensile strain in anchorage[%] |
| $\varepsilon_{max}$ | Maximum tensile strain in NSMR [%] |
| $\varepsilon_{s}$ | Strain in yielding tensile steel reinforcement [%] |
| $\varepsilon_{s0}$ | Strain in not yielding tensile steel reinforcement [%] |
| $\varepsilon_{sy}$ | Ultimate strain for yielding of tensile steel reinforcement [%] |
| $\varepsilon_{u0}$ | Initial strain in the bottom of the cross-section[%] |
| $\gamma_{c}$ | Partial coefficient for strength in concrete $[-]$ |
| $\gamma_{cE}$ | Partial coefficient for the modulus of elasticity of concrete $[-]$ |
| $\gamma_{d}$ | Partial coefficient for safety classification $[-]$ |
| $\gamma_{frp}$ | Partial coefficient for the modulus of elasticity of CFRP $[-]$ |
| $\gamma_{n}$ | Partial coefficient for safety classification $[-]$ |
| $\gamma_{s}$ | Partial coefficient for strength of tensile steel reinforcement $[-]$ |
| $\lambda$ | Rectangular distribution factor of compressed concrete $[-]$ |
| $\lambda_{n}$ | Factor for the minimum bond length for a NSMR bar $[-]$ |
| $\lambda_{\tau}$ | Factor for the maximum shear stress at the end of the CFRP $[-]$ |
| $\eta$ | Strength factor of compression block in the compressed concrete zone $[-]$ |
| $\omega$ | Reinforcement factor $[-]$ |
| $\omega_{bal}$ | Balanced reinforcement factor $[-]$ |
| $\sigma_{1}$ | Main stress in the end of the CFRP $[MPa]$ |
| $\sigma_{cbot}$ | Stress in compressed concrete for unstrengthened cross-section $[MPa]$ |
| $\sigma_{cp}$ | Compressive stress in the concrete due to normal force $[MPa]$ |
| $\sigma_{s}$ | Stress in the steel reinforcement for unstrengthened cross-section $[MPa]$ |
| $\sigma_{x}$ | Normal stress in the x-direction at the end of the CFRP $[MPa]$ |
| $\sigma_{y}$ | Normal stress in the y-direction at the end of the CFRP $[MPa]$ |
| $\tau_{f}$ | Bond shear stress in NSMR $[MPa]$ |
| $\tau_{max}$ | Maximum shear stress at the end of the CFRP $[MPa]$ |
| $\theta$ | Angle of concrete compression struts/crack inclination angle $[rad]$ |
| $\nu$ | Poisson's ratio $[-]$ |
| $\nu$ | Reduction factor for concrete with shear cracks $[-]$ |
| $\rho$ | Coefficient $[-]$ |
| $\xi$ | Coefficient $[-]$ |
| $\phi$ | Diameter of tensile steel reinforcement $[mm]$ |
| $\phi_{s}$ | Diameter of shear reinforcement $[mm]$ |
| $\Psi_{0,1}$ | Factor for interacting live loads $[-]$ |
| $\Psi_{0,i}$ | Factor for interacting live loads $[-]$ |

## Table of contents

# 1 Introduction

## 1.1 Background

During the last years the need to strengthen existing concrete structures in Sweden has increased. The reason is mostly because of deterioration or because the bearing capacity of the structure is not high enough for current traffic demands (Betongföreningen, 2002). Unless more advanced assessment methods fail to verify its safety, the structures must be either improved or rebuilt (Vägverket, 2000).

The most common ways of strengthening concrete structures are by increasing the dimensions of the cross-section, installing external post tensioning or applying external reinforcement. Earlier, the most common external reinforcement method was externally bonded steel plates. However, installations of Carbon Fibre Reinforced Polymers (CFRP) are getting more common (Vägverket, 2000).

CFRP consists of carbon fibres embedded in a thermosetting plastic, and can be applied with an adhesive on a concrete surface (Betongföreningen, 2002). The most commonly used CFRP strengthening systems are externally bonded laminates or sheets, as well as bars (Near Surface Mounted Reinforcing, NSMR) (fib, 2001). Strengthening with CFRP has become popular due to the high strength and stiffness of the fibres, and also because of the light weight of the fibres compared to other materials (Al-Mahmoud et al., 2010). An advantage is also that strengthening can be done without changing the appearance of the structure too much (Smith and Teng, 2000).

Within the field of civil engineering, the use of CFRP composites is still seen as a new technique which is in progress (fib, 2001). More knowledge is therefore needed about the long-term behavior of CFRP strengthening, its moisture effects on concrete structures, environmental effects, behavior in extreme temperatures and so on (fib, 2001). One drawback with CFRP strengthening is that most failure modes of CFRP strengthened beams are brittle, and give no indication of failure (Maalej and Leong, 2005). Another major drawback with the CFRP strengthening is the cost of the material, which is high compared to steel and concrete (Täljsten, 2006). Even though the material cost is high, other costs, for example the labor cost can be lower for CFRP strengthening than for other strengthening methods.

Since the interest in performing CFRP strengthening has increased, the interest in understanding the strengthening technique from an economic perspective has also increased, especially to choose best suited materials and systems for the right purpose.

Figure 1 presents the calculation procedure for an existing bridge. At first, a classification assessment is done to determine if the bridge requires strengthening. In case strengthening is required, a strengthening design is carried out.

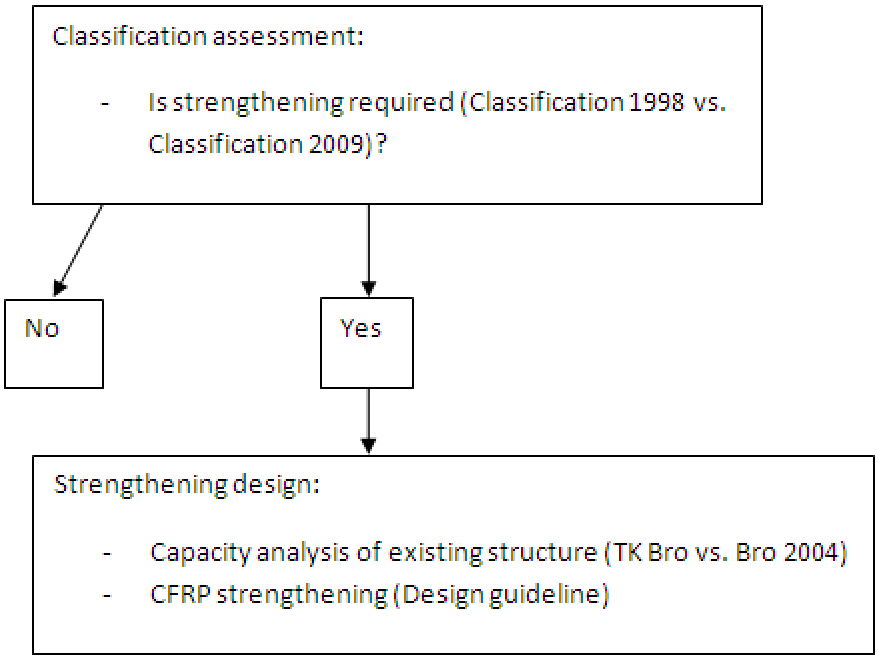



**Figure 1. Calculation procedure for an existing bridge.**

Earlier, classification assessments of Swedish road bridges have been carried out according to Vägverket's "Allmän teknisk beskrivning för Klassningsberäkning av vägbroar; 1998:78" (Classification 1998), and strenghtening design has been done according to "Vägverkets allmänna tekniska beskrivning för nybyggande och förbättring av broar, Bro2004; 2004:56" (Bro2004). In July 2009 the European building code, Eurocode became the governing code for Swedish bridge design. Consequently, Vägverket created national documents on how to apply Eurocode for Swedish bridge design. Since 2009, classification of bridges should be based on the document "Metodbeskrivning 802 Bärighetsutredning av byggnadsverk; 2009:61" (Classification 2009) instead of Classification 1998. The strengthening designs should be done in agreement with Vägverket's "TK Bro; 2009:27" (TK Bro) instead of Bro2004.

## 1.2 Purpose and goal

The purpose with this thesis is to with help of a case study increase the understanding of the differences between the earlier Swedish codes and the new, existing ones, regarding classification assessments and strengthening design of a reinforced concrete bridge.

The purpose is also to analyze some common CFRP strengthening systems for a reinforced concrete bridge. The analysis is done from an economic perspective, considering different alternatives of material properties, such as the modulus of elasticity and the cross-sectional dimensions of the CFRP.

## 1.3 Objectives and limitations

The following objectives and limitations are dealt with in this thesis:

- For the studied bridge, how does the classification assessment differ when based on Classification 2009 compared to Classification 1998? Also, what are the differences between the strengthening design based on TK Bro and Bro2004?

- Which are the most cost effective ways to strengthen the bridge with Carbon Fibre Reinforced Polymers in flexure and in shear?

The thesis will be limited to an analysis in flexure and in shear for one specific beam of the studied bridge, and will only be done for the Ultimate Limit State (ULS). Only strengthening systems with CFRP will be considered, and the systems that will be analyzed are externally applied laminates, sheets, and bars. The analysis is also limited to CFRP systems from one manufacturer.

## 1.4 Outline of the thesis

The thesis is theoretic and based on literature studies and strengthening calculations for a case study. The result of the strengthening calculations is thereafter analyzed from an economic perspective.

The outline of the thesis follows the calculation procedure presented in Figure 1. The classification assessment is carried out in agreement with Classification 2009 and is compared to a classification assessment performed according to Classification 1998.

The strengthening design is carried out by calculating the capacity of the existing structure in agreement with TK Bro (based on Eurocode), and is compared to the results from a capacity calculation done in agreement with Bro2004 (based on BBK).

The strengthening calculations with CFRP are performed with the design guideline "Dimensioneringshandbok för förstärkning av betongkonstruktioner: Teknisk rapport" (2011) from Luleå University of Technology. The calculations are performed with help of the computer software MathCAD and the loads and load combinations are derived with the computer software Stripstep. Finally, the economic analysis is done for the possible strengthening alternatives with estimations of costs for material and labor (including equipment).

# 2 Carbon Fibre Reinforced Polymers

Fibre Reinforced Polymers (FRP) is a composite material that consists of fibres in a resin matrix, bonded to a surface with an adhesive. The material used for the fibres are usually carbon, aramid or glass. The physical and mechanical properties of these different fibre materials, as well as within the fibre materials can differ greatly. The most commonly used fibre material in the construction industry is carbon, which will exclusively be mentioned from now on. FRP based on carbon fibres is defined as Carbon Fibre Reinforced Polymers (CFRP). The advantages with the CFRP are its high tensile strength, high stiffness, low weight and large deformation capacity compared to other strengthening materials. It is also less affected by aggressive environments, as the CFRP does not corrode to the same extent as ordinary, untreated steel reinforcements (fib, 2001).

## 2.1 CFRP systems

The externally bonded CFRP systems are separated into prefabricated systems and "wet lay-up" systems. The suitability of the system depends on factors such as the structure in need of strengthening and the material components of the CFRP (fib, 2001).

Prefabricated systems are usually called strips or laminates (fib, 2001). The prefabricated systems are fully cured and have their final strength and stiffness prior to the application (fib, 2001). They are installed on a straight concrete surface with help of the adhesive, as seen in Figure 2 (fib, 2001). For a good bonding result, it is optimal to only apply one layer of laminates, although it is acceptable to use two layers (Blanksvärd, 2011). The laminates are usually used for flexural strengthening, but can also be used in specific cases of shear strengthening (Blanksvärd, 2011).

**Figure 2. Application of laminate (Sto Scandinavia AB, 2011).**

Wet-lay-up systems include unidirectional fibre sheets, semi-unidirectional fibre sheets, or multidirectional fabrics (see Figure 3). The systems are either applied directly on a resin saturated concrete surface, or initially saturated with the resin in a machine and afterwards applied wet to the surface (fib, 2001). For the wet lay-up systems, the adhesive aims not only to bond the reinforcement to the concrete surface, but also to impregnate the fibres (fib, 2001). The systems are applicable on structures with all types of shapes and are often applied with multiple layers (fib, 2001). In extreme cases ten layers of sheets are acceptable, but increasing the number of layers will make it harder to ensure good quality (Blanksvärd, 2011). Usually sheets are used for strengthening in shear or for openings, but are also possible to use for flexural strengthening of shorter spans (Blanksvärd, 2011).

**Figure 3. Sheets (Sto Scandinavia AB, 2011).**

Near Surface Mounted Reinforcing systems (NSMR) consist of bars that are placed in sawed slots in the concrete cover, and bonded with an epoxy adhesive (see Figure 4). The system is usually used for flexural strengthening when the strengthening system needs to be protected from surface damage, or when the surface is uneven. It is important that the concrete cover has a sufficient depth of at least 20-25 mm for this system (Täljsten, 2006). Due to the design of the NSMR strengthening system, it has better abilities to anchor the stresses compared to an externally bonded strengthening system, such as laminates or sheets (Al-Mahmoud et al., 2010).

**Figure 4. Application of NSMR (Sto Scandinavia AB, 2011).**

A finishing system is usually applied on the surface of all types of CFRP-systems to protect the structure from fire, wearing or UV-light (Täljsten, 2006).

## 2.2 Workmanship

During the strengthening process, it is important to have the right temperature, humidity and a not too uneven concrete surface. The temperature on the concrete surface should for example not be below 10° C, for a good result of the hardening process of the thermosetting (fib, 2001). The quality control accordingly has to be thorough during the application and the strengthening method requires good workmanship and education (fib, 2001). Special precaution has to be made when working with the thermosetting epoxy since working with the thermosetting might induce allergic reactions (Arbetsmiljöverket, 2005).

When strengthening a structure with CFRP it is also important to consider the impact it might have on the concrete structure. Even though the CFRP elements themselves are not easily affected by moisture, the concrete structure that is strengthened or the adhesive might be affected. For example if an excessive amount of moisture is localized at the bonding line, between the CFRP and the concrete substrate.  If the excessive moisture freezes it will then expand and may cause delamination of the concrete. Excessive moisture at the bonding line can also result in corrosion on the internal steel reinforcement, and the CFRP should therefore not cover the whole concrete surface (fib, 2001).

## 2.3 Cost effectiveness

The material cost for CFRP is high compared to other strengthening materials, but the cost based on the strength is more favorable for the CFRP (fib, 2007). It is thus important to minimize the quantity of the material to make it a cost effective strengthening method (Hollaway & Head, 2001). An advantage with the CFRP is its light weight, which makes it easier to handle during the construction work and less expensive to transport (fib, 2007). The strengthening work can moreover usually be done during a short period of time, resulting in a lower labor cost (Täljsten, 2006). The design cost is though higher because of a complicated design process and a need to optimize the use of the material (Hollaway & Head, 2001).

## 2.4 Important assumptions for strengthening with CFRP

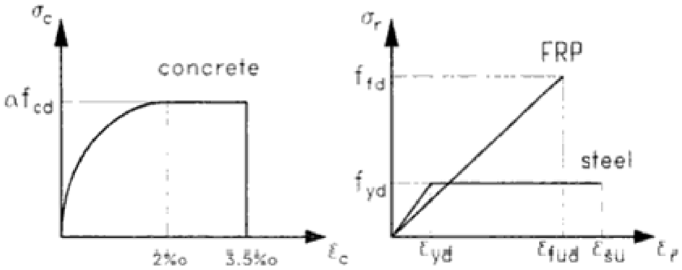



**Figure 5. The stress-strain curve for concrete, steel and FRP in ULS (fib, 2001).**

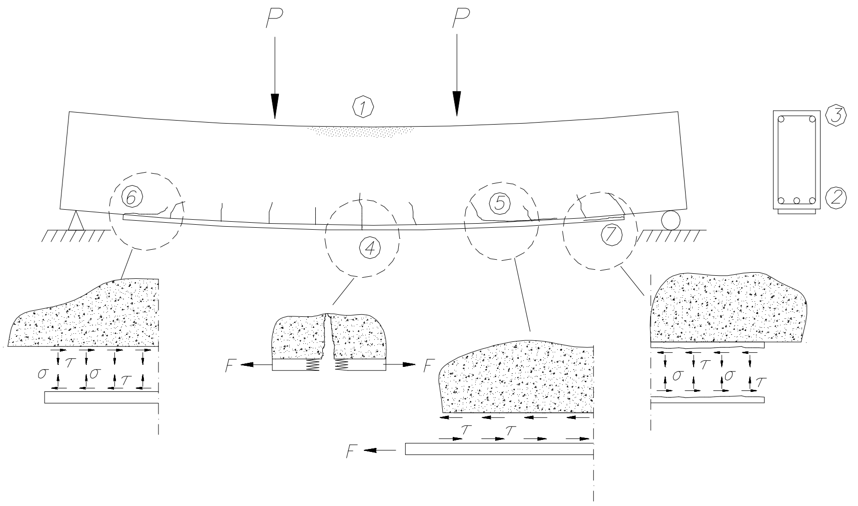
The stress-strain curve for concrete in compression is parabolic-rectangular, while the steel reinforcement has a bilinear stress-strain relationship, as seen in Figure 5. The FRP, on the other hand, has a linear elastic relationship (fib, 2001).

Most of the research done on shear strengthening by externally bonded CFRP, is based on the assumption that the strengthening can be equalized to a strengthening done with internal steel stirrups and thereby contribute to the total shear capacity of the structure (fib, 2001).

## 2.5 Failure modes for flexural strengthening

It is important to understand how a strengthened structure will react when reaching failure. Therefore, it is necessary to study the different possible failure modes, to be able to strengthen the structure in a proper way (Buyukozturk & Hearing, 1998). For a section strengthened with CFRP there are some additional failure modes that can occur for the reinforced concrete structure. As the CFRP behaves linearly elastic until failure, caution must be made to the possible failure modes in the strengthened structure. This means that there are no significant signs when approaching failure in CFRP, since there are no signs of yielding or plastic deformation of the CFRP (Täljsten, 2006).

The possible failure modes for flexural strengthening are presented in Figure 6 (Täljsten, 2006).



1. Compressive failure in concrete
2. Yielding of steel reinforcement in tension
3. Yielding of steel reinforcement in compression
4. Tensile failure in CFRP
5. Anchorage failure in the bond zone (intermediate crack debonding)
6. Peeling failure at the end of CFRP
7. Delamination of CFRP

**Figure 6. Failure modes in flexural strengthening (Täljsten, 2006).**

Some of the presented failure modes in Figure 6 are brittle and some are ductile. It is preferable to design for a ductile failure, as the brittle failure modes are more unpredictable and can occur unexpectedly. A ductile failure, on the other hand, usually starts with a premature deformation, before the becoming failure (Täljsten, 2006).

Concrete structures that are not strengthened with CFRP, can fail either in crushing of concrete or yielding of steel reinforcement. The preferable failure mode for this structure is yielding of steel, as it is a ductile failure mode while concrete crushing is a brittle failure mode (Isaksson et al., 2010).

The ductile or mostly ductile failure modes are; yielding of steel in tension, yielding of steel in compression, and anchorage failure in the bond zone. The preferable failure mode to design for is yielding of steel reinforcement in tension followed by concrete crushing. Compressive failure in concrete, tensile failure in CFRP, peeling failure at the end of CFRP, and delamination of CFRP, are all brittle failures that must be avoided (Täljsten, 2006).

If the ratio of steel and CFRP reinforcement is large, a section may fail due to compression in concrete (Buyukozturk & Hearing, 1998). If the section fails in concrete crushing, and the CFRP has not yet failed, the section is over-reinforced (Täljsten, 2006). Therefore, it is important to optimize the CFRP and steel reinforcement ratio (Buyukozturk & Hearing, 1998).

Intermediate crack debonding develops if a crack propagates along the CFRP, and is thereby difficult to detect (Täljsten et al., 2011). When the crack is originated in the concrete, tensile stresses in the concrete are transferred to the CFRP. Thereafter, high interfacial stresses arise between the concrete and the CFRP close to the crack that result in debonding (Teng, 2003). To avoid intermediate crack debonding the strain in the CFRP should be limited (Täljsten et al., 2011).

Peeling failure at the end of CFRP, also called peeling-off failure, occurs when the bond between concrete and CFRP is lost in such a way that the CFRP cannot take any loads. The failure will be brittle and sudden if the stresses are not able to be redistributed to the internal steel reinforcement of the structure. The peeling-off failures in a CFRP strengthened concrete structure can occur due to several reasons and at several positions such as in an uncracked zone, at flexural cracks, at shear cracks or caused by unevenness of the concrete surface (fib, 2001).

The bond between the CFRP and the concrete surface is important when the forces should be transferred from the concrete to the CFRP, i.e. anchorage stresses. Anchorage failure means that the composite action between the concrete and the CFRP is lost so that the force transfer cannot take place (fib, 2001).

The anchorage of the CFRP is important to achieve an effective design (Täljsten, 2006). If mechanical anchorage is required to resist peeling and shear stresses, there are mainly three alternatives recommended to anchor the CFRP; plates, U-shaped sheets or L-shaped strips (fib, 2001). Plates can be used together with bolts as shown in Figure 7 (Vectura, 2010).

**Figure 7. Mechanical anchorage with plates (Vectura, 2010).**

## 2.6 Failure modes for shear strengthening

Normally, a concrete structure is designed to cope with large deformations before failure, which means that the structure usually fails in flexure. The failure in shear is generally a brittle failure, and should therefore be avoided.

Shear failures are complex mechanisms that depend on shearing, giving a bi-axial stress-state in the beam. The shear design models are mostly based on empirical studies, such as experiments on beams (Täljsten, 2006).

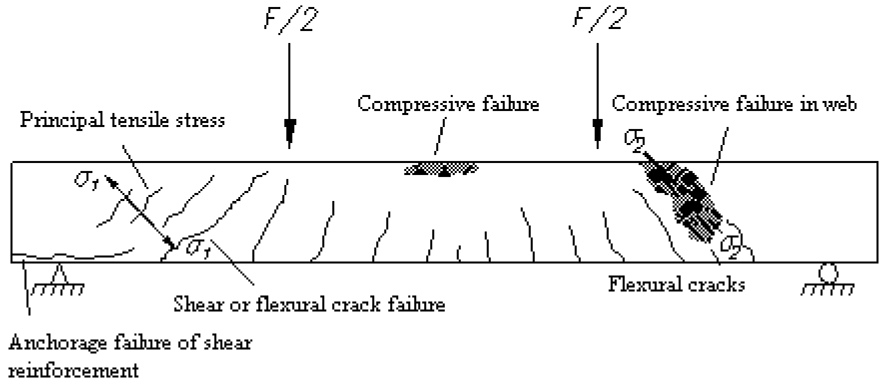



**Figure 8. Failure modes in shear for RC beam (Täljsten et al., 2011).**

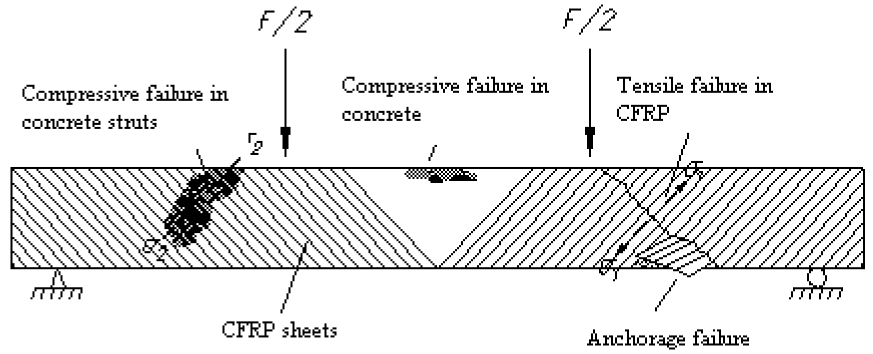



**Figure 9. Failure modes for strengthening in shear with CFRP sheets (Täljsten et al., 2011).**

Some possible failure modes in shear are presented in Figure 8 and Figure 9 and are described below (Täljsten, 2006).

- Web shear failure – Web shear failure occurs where the beam is not affected by flexural cracks so that the principal tensile stress exceeds the tensile strength of the concrete. It is often a result of insufficient shear reinforcement.

- Flexural shear failure – Starts in bending cracks, and grows from the tensile zone of the structure to the compressive zone.

- Compressive failure in concrete struts – Occurs when the shear reinforcement is over-dimensioned; moreover, the steel does not reach the yield limit until the concrete in compression has crushed.

- Anchorage failure of shear reinforcement – Normally the inclined shear cracks are situated near the compression zone of the beam. At every point where the crack intersects one of the stirrups the stresses reach yielding and it is therefore important that the stirrups have enough anchorage.

- Tensile failure in CFRP – Failure occurs when the strain capacity in the fibre is exceeded; usually the fabric gradually fails due to a propagating failure.

- Anchorage failure – When the area where the fabrics are anchored is too small to transfer the shear forces from the concrete structure, or external strength of the concrete is not high enough, anchorage failure will occur.

# 3 Codes and design guidelines

A classification assessment of an existing structure is done to determine its load-carrying capacity, and to find out if strengthening is required. Before July 2009, the classification assessments were carried out according to Vägverket's "Allmän teknisk beskrivning för Klassningsberäkning av vägbroar; 1998:78" (Classification 1998). If strengthening was required, the strengthening design was done according to "Vägverkets allmänna tekniska beskrivning för nybyggande och förbättring av broar" (Bro2004).

For both the classification assessment and the strengthening design, the capacity analyses were done according to "Boverkets handbok om betongkonstruktioner" (BBK 04). Classification 1998 and Bro2004 were both governed by "Vägverkets föreskrifter (VVFS 2004:31) om bärförmåga, stadga och beständighet hos byggnadsverk vid byggande av vägar och gator" (VVFS 2004:31). The hierarchy of the codes can be seen in Figure 10.

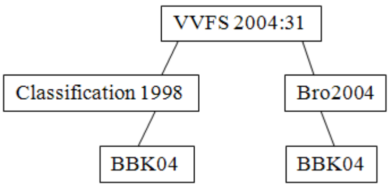



**Figure 10. The hierarchy of the codes that were used before July 2009.**

Since July 2009, new codes are in use. For the classification assessment "Metodbeskrivning 802 Bärighetsutredning av byggnadsverk; 2009:61" (Classification 2009) is used. If strengthening is required, the strengthening design is done in agreement with TK Bro; 2009:27 (TK Bro).

For the classification assessment, the capacity analysis is done according to BBK04. For the strengthening design Eurocode is used for the capacity analysis, as seen in Figure 11. The governing code is still VVFS 2004:31, but the national decisions of Eurocode are presented in "Vägverkets föreskrifter (VVFS 2004:43) om tillämpningen av europeiska beräkningsstandarder" (VVFS 2004:43).

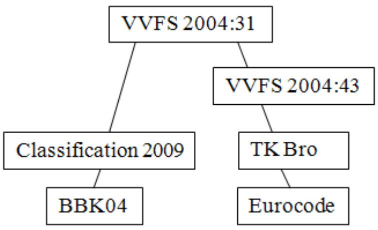



**Figure 11. The hierarchy of the present codes.**

According to "TR Bro; 2009:28" (TR Bro) and Bro2004, an improvement of the capacity of a reinforced concrete bridge in ULS can be done with CFRP in agreement with the earlier publication of the strengthening design guideline called "FRP Strengthening of Existing Concrete Structures - Design Guidelines" (2006). In 2011, a new version of the guideline will be published.

## 3.1 *Loads* and load combinations

### 3.1.1 Classification assessment

The load combinations are defined in the same way for Classification 1998 and for Classification 2009. The load combination for ULS is considered as in Eq.1, where $E_d$ represents the design value of the load.

$$E_d = 1.0G_{k,j} + 1.2G_{k,p} + 1.3Q_{k,1} + 0.7Q_{k,i} \tag{1}$$

$G_k$ is defined as the characteristic value of the permanent load and consists of the self-weight of the primary structure including railings and the fill material etc. ($G_{k,j}$), and the weight of the pavement ($G_{k,p}$) (Vägverket, 2009a).

$Q_{k,1}$ is the characteristic value of the most unfavorable live load while $Q_{k,i}$ is the characteristic value of other live loads. $Q_{k,1}$ will be given the load coefficient 1.3, while other live loads ($Q_{k,i}$) will be given the load coefficient 0.7 (Vägverket, 2009a).

The live load consists of the traffic load and the braking force. When performing the classification assessment for a bridge, the traffic loads that the bridge should be able to carry are defined as an axle load ($A$) and a bogie load ($B$). These values are usually mentioned as the “A/B values”. The A/B values are combined into different load combinations to form design trucks as seen in Appendix 1. According to Classification 1998, the first 12 design trucks in Appendix 1 should be considered. In Classification 2009, there are two additional design trucks, meaning that all 14 design trucks in Appendix 1 are to be considered.

Due to braking, horizontal forces are induced. These braking forces are according to Vägverket (2009a):

- $70\ kN$ for a bridge with a total length of maximum $20\ m$.
- $170\ kN$ for a bridge with a total length of maximum $40\ m$.
- $470\ kN$ for a bridge with a total length greater than, or equal to $170\ m$.

For bridges with lengths in between the specified lengths above, the braking force can be determined with a linear interpolation (Vägverket, 2009a).

The dynamic contribution, defined in Eq. 2 should be added to all concentrated loads (Vägverket, 2009a).

$$\varepsilon = \frac{740}{20+L_{beam}} \qquad (\varepsilon\ in\ \%,\ L_{beam}\ in\ m) \qquad 2$$

### 3.1.2 Strengthening design

The loads and load combinations for the strengthening design done in agreement with Bro2004 are identical to those described in Section 3.1.1, for the classification assessments.

The load combinations for TK Bro differ from the ones described in Section 3.1.1, as they are based on load combinations from Eurocode. The load combinations are defined in SS-EN 1990, where STR is the load combination for a load-bearing member in ULS. The design factors for the different loads in load combination STR are defined in VVFS 2004:43, and are presented in Eq. 3 and 4. In Eq. 3, the permanent load is the dominating load while in Eq. 4 the live load is the dominating load (SS-EN 1990, 2004).

$$E_d = \gamma_d 1.35 G_{k,j} + \gamma_d 1.5 \psi_{0,1} Q_{k,1} + \gamma_d 1.5 \psi_{0,i} Q_{k,i} \qquad 3$$

$$E_d = \gamma_d 0.89 \cdot 1.35 G_{k,j} + \gamma_d 1.5 Q_{k,1} + \gamma_d 1.5 \psi_{0,i} Q_{k,i} \qquad 4$$

$G_{k,j}$ is the characteristic value of the total permanent load, $Q_{k,1}$ is the characteristic value of the main live load, and $\psi_{0,1}$is a factor for the dominating live load (SS-EN 1990, 2004). $Q_{k,i}$ is the characteristic value of an interacting live load that is not the main live load. $\psi_{0,i}$ is thus a combination factor for the interacting live loads (SS-EN 1990, 2004).

$\gamma_d$ is a partial safety factor to consider the safety classification. The safety factor is in Eurocode reducing the load, while in the classification assessments, the reduction factor ($\gamma_n$) does instead reduce the capacity of the structure (Vägverket, 2009a). The partial safety factors for the different safety classifications are defined as (VVFS 2004:43, 2009):

-safety classification 1: $\gamma_d = 0.83$

-safety classification 2: $\gamma_d = 0.91$

-safety classification 3: $\gamma_d = 1.0$

The permanent load ($G_{k,j}$) in TK Bro is defined in the same way as in Section 3.1.1, and can be found in SS-EN 1991-1-1. Furthermore, the weight of the pavement should be increased with 10 %, due to possible future changes (VVFS 2004:43, 2009).

According to VVFS 2004:43 (2009), the first 12 design trucks in Appendix 1 should be checked. The traffic load and braking force will both be included in the main live load.

$$Q_{lk} = 0.6\alpha_{Q1}(2Q_{1k}) + 0.10\alpha_{q1}q_{1k}w_1L \quad 5$$

$$180\alpha_{Q1}(kN) \leq Q_{lk} \leq 900(kN) \quad 6$$

$Q_{lk}$, defined in Eq. 5 and Eq. 6, is the value of the characteristic longitudinal forces (braking and acceleration forces), $Q_{1k}$ is the characteristic value of the vertical axle load, and $q_{1k}$ is the value of the uniformly distributed load. $\alpha_{Q1}$ and $\alpha_{q1}$ are adjustment factors for the load models, and $w_1$ is the width of one lane on a road bridge.

The dynamic factor applied to the axle and bogie loads of the design trucks in Classification 2009 are described in Section 3.1.1 and are limited to 35%.

## 3.2 Flexural capacity

Some assumptions have to be made to calculate the flexural capacity of the structure; these assumptions can be found in Isaksson et al. (2010).

The stress-strain distribution curve of the tensile steel reinforcement is defined as in Figure 12 where the steel is assumed to have an elastic behavior until the yielding point and thereafter have a plastic behavior with a constant stress.

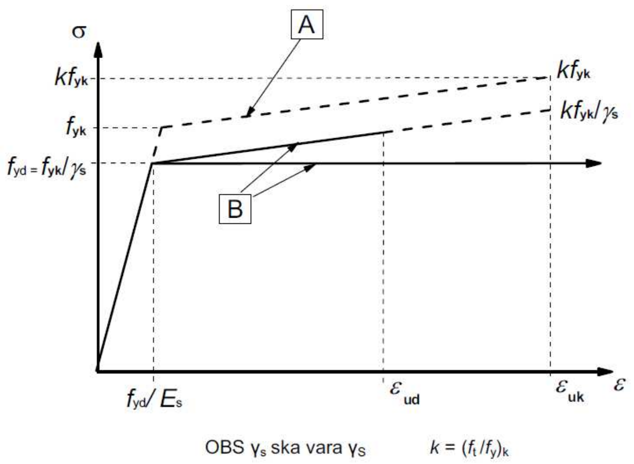



**Figure 12. The stress-strain distribution curve for tensile steel reinforcement (SS-EN 1992-1-1:2005, 2008).**

### 3.2.1 Classification assessment

The procedure to calculate the capacity of an existing structure is the same for Classification 2009 and Classification 1998, as both of them are based on BBK 04. Note that this does not apply for the loads and load combinations, as there are two more design trucks that must be considered in the design load for Classification 2009, as described earlier. Therefore, the outcome of the classification assessments might be different.

### Safety classification

$\gamma_n$ is a partial safety factor that considers the safety classification, and reduces the capacity of the structure depending on the safety classification of the structure (BBK 04, 2004).

- Safety class 1: $\gamma_n = 1.0$
- Safety class 2: $\gamma_n = 1.1$
- Safety class 3: $\gamma_n = 1.2$

### Material properties

The design values for tensile strength of concrete ($f_{ctd}$) and compressive strength of concrete ($f_{cd}$) are calculated according to Eq. 7 and 8 with the partial coefficient ($\gamma_c$), and the partial coefficient for safety classification ($\gamma_n$) (Boverket, 2004). The characteristic values $f_{ctk}$ and $f_{ctk}$ are defined in BBK 94.

$$f_{ctd} = \frac{f_{ctk}}{\gamma_n \gamma_c} \qquad 7$$

$$f_{cd} = \frac{f_{ck}}{\gamma_n \gamma_c} \qquad 8$$

The design value of the modulus of elasticity of concrete ($E_{cd}$) is calculated with Eq. 9, where $\gamma_{cE}$ and $\gamma_n$ are defined in BBK 04 and the characteristic value $E_{ck}$ is based on test results. The ultimate limit strain ($\varepsilon_{cu}$) for concrete is also found in BBK 04 and should not be adjusted with any partial coefficient for concrete.

$$E_{cd} = \frac{E_{ck}}{\gamma_n \gamma_{cE}} \qquad 9$$

The yield strength of the tensile steel reinforcement ($f_{yk}$) should be adjusted with the partial coefficient for tensile steel reinforcement ($\gamma_s$), and the partial coefficient for safety classification ($\gamma_n$ ) to get the design value ($f_{yd}$), see Eq. 10.

$$f_{yd} = \frac{f_{yk}}{\gamma_n \gamma_s} \qquad 10$$

### Tensile force and capacity of the tensile steel reinforcement

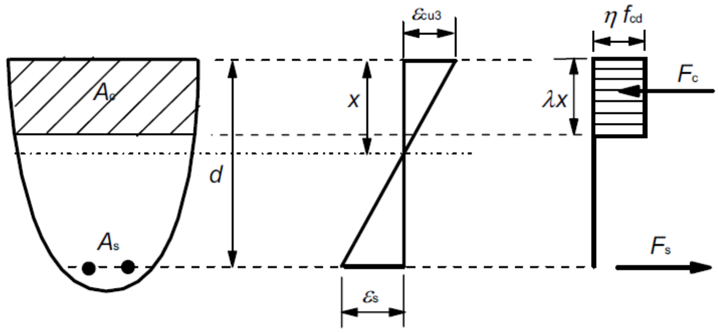



**Figure 13. The stress-strain distribution of the compressed concrete (SS-EN 1992-1-1:2005, 2008).**

The distance to the neutral layer ($x_{N.A.}$) for the unstrengthened cross-section is calculated with help of equilibrium of the bending moment (Eq. 11), see Figure 13.

$$M_d = f_{cd}\eta\lambda x_{N.A.}b(d - 0.5\lambda x_{N.A.}) + N_d(d - 0.5h) \qquad 11$$

In case a normal force is present in the cross-section, the structure is subjected to a bending moment because of the transverse load as well as a bending moment created by the non-centric normal force. The normal force ($N_d$) is usually assumed to be situated at half the height of the cross-section.

The tensile force in the tensile steel reinforcement ($F$) is found with help of force equilibrium (see Eq.12). Where $F_c$ is the force in the concrete and $N_d$ is the normal force.

$$F = F_c + N_d \qquad 12$$

The curve for the tensile force in the steel reinforcement along the structure has to be adjusted with the shifting distance $a_l = 0.5d$ toward the supports, because of the impact of shear, as seen in Figure 14 (Boverket, 2004). The adjusted tensile force is denoted as $F_{adj}$.

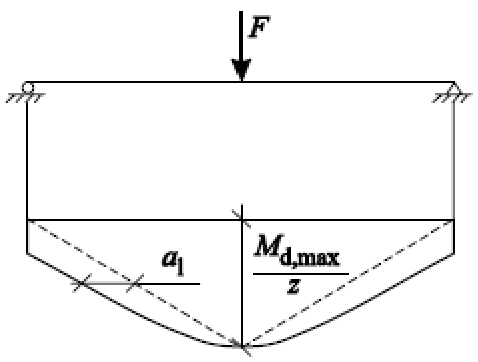



**Figure 14. Adjustment of tensile force due to impact of shear (Boverket, 2004).**

The tensile capacity in the steel reinforcement, also called the yield force ($F_s$), is based on information about the material properties of the steel and the positions and anchorage lengths of the reinforcing bars.

**Determine if strengthening is required**

Finally, the difference between the capacity ($F_s$) and the load ($F_{adj}$) is resulting in a conclusion of whether the structure has to be strengthened or not, to manage the required classification. In case the difference ($\Delta F$) is relatively high when calculated according to Eq.13, flexural strengthening should be considered. The strengthening design is then done in agreement with TK Bro, and was earlier done according to Bro2004.

$$\Delta F = F_{adj} - F_s \qquad 13$$

### 3.2.2 Strengthening design

The capacity analysis done according to Bro2004 is performed similarly to the one in Classification 1998. The analysis will give the same result of required strengthening as the maximum value of $\Delta F$ derived in Section 3.2.1, in agreement with Classification 1998.

Since TK Bro is based on Eurocode, the capacity analysis does not look exactly the same for the strengthening design as described in Section 3.2.1. The basic differences are however the way the material properties, as well as the shifting distance ($a_l$) are defined.

The design values ($f_{ctd}$) and ($f_{cd}$) are calculated with Eq. 14 and 15. The partial coefficient ($\gamma_c$), the characteristic tensile strength ($f_{ctk}$), and the characteristic compressive strength ($f_{ck}$) are found in SS-EN 1992-1-1:2005. Note that the partial safety factor ($\gamma_n$) is not included. As all calculations are done for ULS, the coefficients of the long term effect for both the compressive strength ($\alpha_{cc}$) and the tensile strength ($\alpha_{ct}$) equals 1.0, according to VVFS 2004:43.

$$f_{ctd} = \frac{\alpha_{ct} f_{ctk}}{\gamma_c} \qquad 14$$

$$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c} \qquad 15$$

The design value $E_{cd}$ is calculated with Eq. 16, where the characteristic value of the modulus of elasticity of concrete ($E_{ck}$) and the partial coefficient ($\gamma_{cE}$) are found in SS-EN 1992-1-1:2005.

$$E_{cd} = \frac{E_{ck}}{\gamma_{cE}} \qquad 16$$

The characteristic yield strength of the tensile steel reinforcement ($f_{yk}$) should be adjusted with the partial coefficient ($\gamma_s$) to get the design value of the yield strength ($f_{yd}$), see Eq.17. The design value of the modulus of elasticity for the tensile steel reinforcement ($E_s$) is defined in Eq. 18, in agreement with SS-EN 1992-1-1:2005.

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \qquad 17$$

$$E_s = 200\text{MPa} \qquad 18$$

The tensile force distribution is adjusted due to the shear reinforcement with the distance $a_l$ according to SS-EN 1992-1-1:2005.

$$a_l = \frac{z(\cot(\theta) - \cot(\alpha))}{2} \qquad 19$$

In Eq. 19, $z$ is the inner lever arm for the tensile steel reinforcement, $\theta$ is the angle of concrete compression struts, and $\alpha$ is the angle of the shear reinforcement with respect to the member axis (SS-EN 1992-1-1:2005, 2008).

The procedure to calculate the flexural capacity of the structure prior to strengthening is otherwise done as in the classification assessment with help of Eq. 11-13.

## 3.3 Shear capacity

When designing a reinforced concrete structure in shear, it needs to be taken in consideration that there are two main materials that are interacting, and that the concrete may be cracked in some parts of the structure. It is usually more difficult to determine the shear capacity than the tensile and compressive capacity, as shear forces usually do not appear in a material on its own, but together with compressive and tensile stresses. The methods for determining the shear strength of a concrete structure are empirically compiled and based on test results, as the behavior of elements subjected to shear is not yet known. The failure in a beam due to shear is usually brittle, and it is therefore important to avoid this type of failure.

### 3.3.1 Classification assessment

For Classification 1998 and Classification 2009, the capacity analyses are the same for bridges built before 2002, as the shear strength can be calculated according to “the addition principle”. The addition principle is based on a superposition of the capacity of the steel reinforcement and the concrete (Boverket, 2004).

For bridges built after 2002, "the alternative model" is to be used to calculate the capacity of the structure in shear (Boverket, 2004). "The alternative model" is the same as "the truss model" used in SS-EN 1992-1-1:2005. The capacity of the truss model is defined as the lesser one of the capacity of the shear reinforcement and the capacity of the concrete compression strut (Vägverket, 2009a). The capacity calculated according to the truss model will be described in Section 3.3.2, while in this section only the addition principle is described. The equations in this section are from BBK 04, if nothing else is stated.

#### Reduction near support

If a concentrated load is situated near the support, some of that load will be transferred to the support (Engström, 2004). Therefore, if the wheel load is placed within the distance of $3d$ from the system line, the capacity of the structure may be increased with a value of the shear strength of concrete, $f_{vr}$ (Eq. 20) (Avén, 1985).

$$f_{vr} = f_v \frac{V_{d0}}{V_{d,red}} \leq f_{ctd} \qquad 20$$

$V_{d0}$ is the shear force for actual load, $V_{d,red}$ is the shear force calculated with reduced load, and $f_v$ is the shear strength of the concrete. The reduced shear force ($V_{d,red}$) is calculated by reducing the load ($F_{red}$) situated closer to the system line than $3d$, see Eq. 21. $a_d$ is the distance from the system line to where the wheel load is placed (Avén, 1985).

$$F_{red} = F \frac{a_d}{3d} \qquad 21$$

The shear strength of concrete ($f_v$) is calculated as in Eq. 22 with the coefficients $\xi$ and $\rho$ as defined in Eq. 23 and 24.

$$f_v = 0.30\xi(1 + 50\rho)f_{ctd} \qquad 22$$

$\xi$ is recommended to: 23

| | | |
|---|---|---|
| $1.4$ | for | $d \leq 0.2m$ |
| $1.6 - d$ | for | $0.2m < d < 0.5m$ |
| $1.3 - 0.4d$ | for | $0.5m < d < 1.0m$ |

$$\rho = \frac{A_{s0}}{b_w d} \leq 0.02 \qquad 24$$

$f_{ctd}$ can then be calculated according to Eq. 14, and $A_{s0}$ is the minimum tensile reinforcement area in the considered part of the beam.

The capacity of the beam can be calculated with or without the shear reinforcement included. If Eq. 25 is satisfied, the shear reinforcement should be included.

$$V_s \geq 0.2 b_w d f_{ctd} \tag{25}$$

**Member without shear reinforcement**

The shear capacity of a beam without statically active shear reinforcement ($V_{sd}$) is based on the concrete capacity ($V_c$) and the influence on the shear capacity of the varying effective depth ($V_i$), Eq. 26.

$$V_{sd} \leq V_c + V_i \tag{26}$$

The concrete capacity ($V_c$) is defined in Eq. 27, with $f_v$ calculated according to Eq. 22.

$$V_c = b_w d f_v \tag{27}$$

**Member with shear reinforcement**

For reinforced concrete with statically active reinforcement, the capacity of the steel ($V_s$) is included in the calculation, see Eq. 28.

$$V_{Sd} \leq V_{Rd} = V_c + V_i + V_s \tag{28}$$

$V_c$ is defined in Eq. 27 and $V_s$ in Eq. 29, where $A_{sw}$ is the cross-sectional area of the shear reinforcement, $s$ is the spacing of the shear reinforcement units, and $\alpha$ is the angle of inclination of the shear reinforcement.

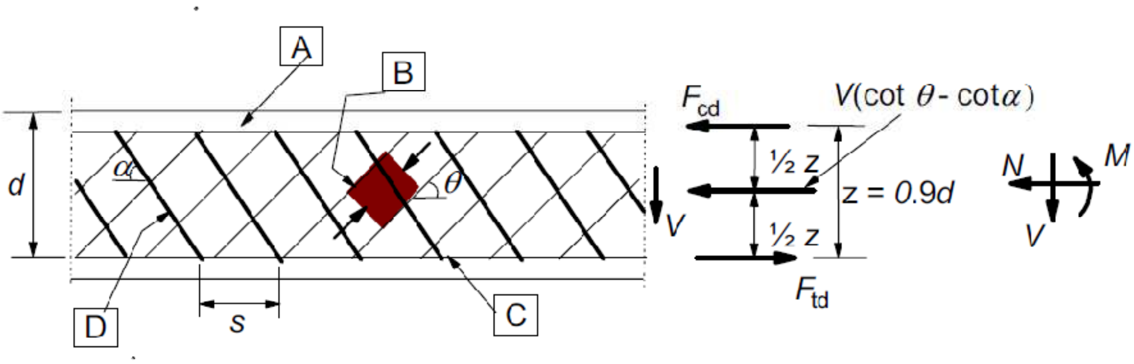
$$V_s = A_{sw} f_{ywd} \frac{0.9d}{s} (sin\alpha + cos\alpha) \tag{29}$$

The design value of the tensile strength of the shear reinforcing steel ($f_{ywd}$) is defined in Eq. 30. $f_{ywk}$ is a reduction of the characteristic value of the strength, $\gamma_n$ is the partial safety factor for safety classification, and $\gamma_s$ is the partial coefficient for steel reinforcement.

$$f_{ywd} = \frac{f_{ywk}}{\gamma_n \gamma_s} \tag{30}$$

### 3.3.2 Strengthening design

The capacity analysis done in agreement with Bro2004 is done in the same way as for the classification assessments, described in Section 3.3.1. For TK Bro, though, the capacity analysis is very different. The equations in this section are from SS-EN 1992-1-1:2005, if nothing else is indicated.




A – Compressed longitudinal reinforcement
B – Concrete compression struts
C – Tensioned longitudinal reinforcement
D – Shear reinforcement

**Figure 15. The truss model which the shear capacity is based on (SS-EN 1992-1-1:2005, 2008).**

The truss model, described in Figure 15 can be used to study the equilibrium in a reinforced structure with inclined shear cracks. In the assumed truss, the compressed concrete will form compression struts, and the steel reinforcement will form tension bars (Engström, 2004).

#### Reduction near support

The supports will take care of some of the concentrated load that is situated near a support, as the load can be transferred to the support by skew compression struts. The shorter the distance is to the support, the steeper the compression struts will be (Engström, 2004). Furthermore, if the wheel load is placed within a distance $a_v < 2d$ from the support, the load may be reduced by multiplying the load with a factor $\beta_{red}$ (see Eq. 31). A presumption of this reduction is that the longitudinal reinforcement is fully anchored.

$$\beta_{red} = \frac{a_v}{2d} \qquad (31)$$

If the distance $a_v$ is lesser than $0.5d$, $a_v$ can be set to $0.5d$.

The capacity of the load-bearing members may be calculated either with or without the shear reinforcement included. In this thesis the case for a member without shear reinforcement is not relevant.

**Member with shear reinforcement**

If the capacity of the beam without shear reinforcement is not sufficient ($V_{Ed} > V_{Rd,c}$), the shear reinforcement needs to be included in the calculations. An angle of the concrete compression struts ($\theta$) must be assumed and limited as in Eq. 32.

$$1 \leq \cot\theta \leq 2.5 \qquad (32)$$

When calculating the capacity of the load-bearing member ($V_{Rd}$), the lesser one of the capacity of the shear reinforcement ($V_{Rd,s}$) and the capacity of the concrete compression struts ($V_{Rd,max}$) is chosen, see Eq. 33. If $V_{Rd} < V_{Ed}$ for the analyzed case, strengthening is required.

$$V_{Rd} = min\begin{cases} V_{Rd,s} \\ V_{Rd,max} \end{cases} \qquad (33)$$

The capacity of the shear reinforcement with vertical stirrups is calculated as in Eq.34.

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} cot\theta \qquad (34)$$

The design value of yield strength of shear reinforcement ($f_{ywd}$) is calculated according to Eq. 35.

$$f_{ywd} = \frac{f_{ywk}}{\gamma_s} \qquad (35)$$

The capacity of the concrete compression struts ($V_{Rd,max}$) is calculated according to Eq. 36.

$$V_{Rd,max} = \frac{\alpha_{cw} b z v_1 f_{cd}}{cot\theta + \tan\theta} \qquad (36)$$

$\alpha_{cw}$ is recommended to: (37)

| | | |
|---|---|---|
| 1 | for | Structure without prestressing |
| $(1 + \sigma_{cp}/f_{cd})$ | for | $0 < \sigma_{cp} \leq 0.25 f_{cd}$ |
| 1.25 | for | $0.25 f_{cd} < \sigma_{cp} \leq 0.5 f_{cd}$ |
| $2.5(1 - \sigma_{cp}/f_{cd})$ | for | $0.5 f_{cd} < \sigma_{cp} \leq 1.0 f_{cd}$ |

## 3.4 Flexural Strengthening

The failure modes that can occur in the cross-section with the governing bending moment and normal force are failure in the CFRP, concrete crushing, or yielding in the tensile steel reinforcement. For the calculation procedure in Täljsten et al. (2011), yielding in the tensile steel reinforcement is preceding the failure of the CFRP. If nothing else is told, the equations in this section are based on Täljsten et al. (2011). The equations from Täljsten et al (2011) are not considering normal forces in the structure, so some of the equations have been modified to deal with the impact of a normal force.

### Material properties of CFRP

The characteristic modulus of elasticity for the CFRP ($E_{fk}$) is modified with the partial coefficient $\gamma_{frp}$ to get the design value of the modulus of elasticity ($E_f$), see Eq. 38.

$$E_f = \frac{E_{fk}}{\gamma_{frp}} \qquad 38$$

The design value for the ultimate strain ($\varepsilon_{fu}$) is based on the characteristic ultimate strain, $\varepsilon_{fuk}$ according to Eq. 39.

$$\varepsilon_{fu} = \frac{\varepsilon_{fuk}}{\gamma_{frp}} \qquad 39$$

### Tensile Failure in the CFRP

The procedure to control that CFRP failure will not be the failure mode is the same for any strengthening system. To start with, the cross-sectional area of the CFRP is assumed. For laminates or sheets it is based on the thickness of each layer ($t_f$), the number of layers ($n$), and the width of the CFRP ($b_f$). For NSMR, only the cross-sectional area of the bars and the number of bars ($n$) are considered. Other important material properties are the ultimate strain in the CFRP ($\varepsilon_{fu}$) and the modulus of elasticity of the CFRP ($E_f$).

Täljsten et al. (2011) suggest that the cross-sectional area of the CFRP ($A_f$) should be derived from equilibrium in bending moment. That is however not possible when the cross-section is dealing with bending moment as well as a normal force. The required area of the CFRP is then found with help of the force equilibrium in the cross-section exposed to the maximum bending moment (Eq. 40). $A_f$ has to be higher than the required area.

$$A_f \geq \frac{\Delta F}{\varepsilon_f E_f} \qquad 40$$

For NSMR-systems, the strain ($\varepsilon_f$) is limited because of the risk of fibre rupture and is therefore equal to the design value of the ultimate strain in the CFRP ($\varepsilon_{fu}$). The strain in the CFRP ($\varepsilon_f$) for laminates and sheets is limited, not primarily because of the risk of fibre rupture, but also because of the risk of intermediate crack debonding ($\varepsilon_{fd,ic}$). Eq. 41 presents the latter restriction which is based on a model used in the American Standard (ACI-440, 2002). The lesser one of $\varepsilon_{fu}$ and $\varepsilon_{fd,ic}$ will then give the governing value of $\varepsilon_f$ for strengthening alternatives with laminates or sheets.

$$\varepsilon_{fd,ic} = 0.41\sqrt{\frac{f_{cd}}{nE_f t_f}} \qquad 41$$

The tensile strength in the CFRP ($F_f$) is calculated in Eq. 42 and should be higher than the required strengthening ($\Delta F$), derived in Section 3.2.2.

$$F_f = \varepsilon_f E_f A_f \qquad 42$$

### Compressive failure in concrete

A control is done to confirm that the cross-section is normally reinforced after the strengthening (Eq. 43).

$$\omega_{bal} \geq \omega \qquad 43$$

$\omega_{bal}$ and $\omega$ are defined in Eq. 44 and 45.

$$\omega_{bal} = \frac{\lambda}{1 + \frac{\varepsilon_f + \varepsilon_{u0}}{\varepsilon_{cu}}} \qquad 44$$

$$\omega = \frac{F_s + F_f}{bhf_{cd}} \qquad 45$$

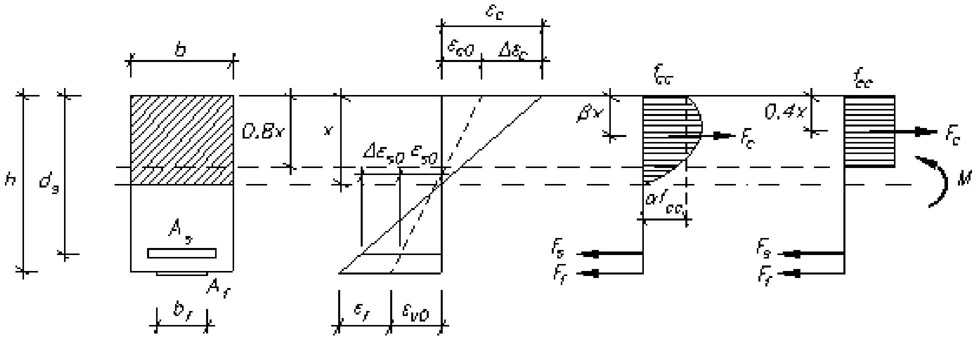


**Figure 16. Cross-section with strengthening (Täljsten et al., 2011).**

$F_s$ is the yield force in the tensile steel reinforcement at the analyzed cross-section as defined in Section 3.2.1. As seen in Figure 16, an initial strain occurs before the strengthening due to the permanent load of the structure. The initial strain in the bottom of the cross-section ($\varepsilon_{u0}$), is found with its linear strain-relationship with the strain in the tensile steel reinforcement ($\varepsilon_{s0}$). Since $\varepsilon_{s0}$ is unknown, the assumption must be made that the tensile steel reinforcement is not yielding. Based on that assumption, $\varepsilon_{s0}$ can be calculated with help of Hooke's law, where the stress in the steel reinforcement ($\sigma_s$) is calculated with Eq. 46 for an unstrengthened cross-section.

$$\sigma_s = \alpha_s \frac{M_{d0}}{I_2}\left(d - x_{cg0}\right) + \alpha_s \frac{N_{d0}(x_{cg0} - \frac{h}{2})}{I_2}\left(d - x_{cg0}\right) + \alpha_s \frac{N_{d0}}{bh} \qquad 46$$

As the design is done in ULS, the cross-section is assumed to be cracked. $I_2$ is the moment of inertia for a cracked section, meaning that only the compressed area of the concrete is included. The bending moment and normal force are only based on permanent loads, as the stress is considered for an unstrengthened cross-section.

The proportionality factors for the tensile steel reinforcement ($\alpha_s$) is calculated according to Eq. 47 (SS-EN 1992-1-1:2005, 2008).

$$\alpha_s = \frac{E_s}{E_{cd}} \qquad 47$$

In case the cross-section is not normally reinforced, strengthening with the chosen alternative of CFRP-strengthening is not an option.

Concrete crushing also has to be checked for the strengthened cross-section. The maximum strain in the concrete after strengthening ($\varepsilon_c$) is calculated with help of its relation with the strain in the CFRP ($\varepsilon_f$) in addition to the strain in the compressed concrete in advance of the strengthening ($\varepsilon_{cbot}$). $\varepsilon_{cbot}$ is calculated with help of Hooke's law, where the stress $\sigma_{cbot}$ is calculated according to Eq. 48, for an unstrengthened and cracked section, subjected to only permanent loads.

$$\sigma_{cbot} = \frac{M_{d0}}{I_2}\left(h - x_{cg0}\right) + \frac{N_{d0}(x_{cg0} - \frac{h}{2})}{I_2}\left(h - x_{cg0}\right) + \frac{N_{d0}}{bh} \quad (48)$$

The concrete will not crush as long as the strain in the compressed concrete ($\varepsilon_c$) does not exceed the ultimate strain of concrete ($\varepsilon_{cu}$).

### Yielding of the tensile steel reinforcement

When the steel is yielding, the stress in the tensile steel reinforcement ($\sigma_s$) will be equal to its tensile strength ($f_{yd}$). So far, the tensile steel reinforcement has been assumed not to yield. Knowing the maximum strain in the compressed concrete ($\varepsilon_c$) for a strengthened cross-section, the strain in the tensile steel reinforcement ($\varepsilon_s$) can be calculated with help of geometry (see Figure 16). The strain in the tensile steel reinforcement is then compared to the strain at which yielding will occur ($\varepsilon_{sy}$).

### Anchorage failure in the bond zone because of cracked concrete

Anchorage without mechanical help can only be guaranteed on uncracked concrete as the empirically derived equations for anchorage are based on laboratory tests for uncracked concrete. It is furthermore safer to anchor the CFRP on uncracked concrete. The bending moment along the beam is therefore compared to the bending moment where the last crack will occur. The moment at the position of the last crack ($M_{xcr}$) is calculated according to Eq. 49.

$$M_{xcr} = W_c f_{ctm} \quad (49)$$

$W_c$ is the flexural resistance for an uncracked, strengthened cross-section. $f_{ctm}$ is the average value of concrete axial tensile strength, which is calculated with Eq. 50 (SS-EN 1992-1-1:2005, 2008).

$$f_{ctm} = 0.3\sqrt[3]{{f_{ck}}^2} \qquad (f_{ck}\ in\ MPa;\ f_{ctm}\ in\ MPa) \quad (50)$$

Anchorage has to be done beyond the point of intersection ($x_{cr}$) between the bending moment where the concrete is cracked, and the bending moment that the beam is exposed to (see Figure 17). If there is no room for anchorage beyond the point $x_{cr}$, mechanical anchorage is the only option to anchor the CFRP.

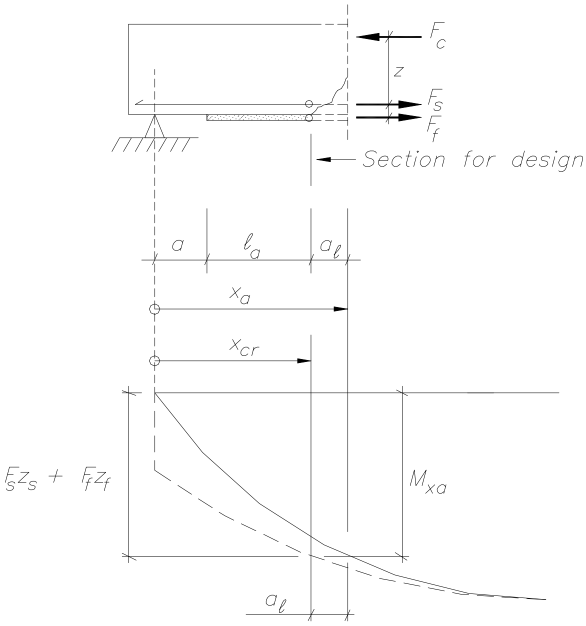



**Figure 17. Anchorage of CFRP and shifted bending moment and the position of the last crack (Täljsten et al., 2011).**

To consider the additional tensile force because of shear, the bending moment curve is shifted the distance $a_l$ (Figure 17). The bending moment ($M_{xa}$) is therefore found for the points $a_l$ from the points where the beam is cracked. The shifting distance is determined according to Eq. 19 in Section 3.2.2.

## Required anchorage length – sheets or laminates

The anchorage length is the critical length from which the strengthening capacity will not increase if the anchorage length increases (Täljsten, 2006). The anchorage length ($l_a$), as seen in Figure 17, is the length beyond the section for strengthening design (Eq. 51). The minimum recommended anchorage length ($l_{cr}$) is $250\ mm$. The anchorage length might also be governed by the empirically derived distance $l_e$ , calculated with Eq. 52. $k_b$ is a size factor, calculated according to Eq. 53. Finally, $l_a$ should be compared to the possible anchorage length within the supports (see Figure 17).

$$l_a = max\begin{cases} l_e \\ l_{cr} \end{cases} \quad (51)$$

$$l_e = 0.6\sqrt{\frac{E_f t_f}{\sqrt{k_b f_{ctm}}}} \qquad (E_f\ in\ MPa;\ t_f\ in\ mm;\ f_{ctm}\ in\ MPa;\ l_e\ in\ mm) \quad (52)$$

$$k_b = \sqrt{\frac{2 - \frac{b_f}{b}}{1 + \frac{b_f}{b}}} \geq 1.0 \qquad 53$$

**Maximum allowed tensile force at position of the last crack – sheets or laminates**

The force at the position of the last crack, where the anchorage zone begins has to be checked to make sure that the CFRP does not take any load within the anchorage zone. The maximum allowed tensile force ($F_{f,e}$) is compared to the real tensile force in the CFRP at the position of the last crack. $F_{f,e}$ is calculated with Eq. 54, where the maximum allowed tensile strain ($\varepsilon_{fx}$) is calculated with Eq. 55. The fracture energy of concrete in the bonded zone ($G_f$) is defined in Eq. 56.

$$F_{f,e} = \varepsilon_{fx} A_f E_f \qquad 54$$

$$\varepsilon_{fx} = \sqrt{\frac{2G_f}{E_f t_f}} \qquad 55$$

$$G_f = 0.03 k_b \sqrt{f_{ck} f_{ctm}} \qquad (G_f \ in \ N/mm) \qquad 56$$

The real tensile force is the largest of $F_{fa}$ and $F_{fb}$, which are calculated with help of the moment equilibrium at the compressed concrete (Eq. 57 and 58).

$$F_{fa} = \frac{M_{xa}}{z_f} - A_s f_{yd} \frac{z}{z_f} + N_{xa} \frac{z_f - 0.5h}{z_f} \qquad 57$$

$$F_{fb} = \frac{\frac{M_{xa}}{z_f} + N_{xa} \frac{z_f - 0.5h}{z_f}}{1 + \frac{E_S A_S}{E_f A_f} \left(\frac{z}{z_f}\right)^2} \qquad 58$$

In Eq. 57 for $F_{fa}$the tensile steel reinforcement is assumed to be yielding at the position of the last crack. On the other hand, in Eq. 58, for $F_{fb}$, the tensile steel reinforcement is assumed not to be yielding at the position of the last crack. If $F_{f,e} \leq F_{fa} \ and \ F_{fb}$, the anchorage has to be moved closer to the supports. Otherwise, the force at the position of the last crack can be transferred and the anchorage is sufficient. $z_f$ is the inner lever arm of the CFRP, while $z$ is the inner lever arm of the tensile steel reinforcement. $M_{xa}$ is the bending moment at the position of the last crack with regards to the shifted distance $a_l$.

**Peeling failure at the end of CFRP – sheets or laminates**

A peeling stress ($\sigma_1$) occurs between the concrete and the adhesive of the CFRP (Eq. 59). $\sigma_1$ is acceptable if it is lesser than the mean value of the tensile strength of concrete ($f_{ctm}$), otherwise the position of anchorage has to be moved closer to the supports, so that peeling-off will not occur at the end of the CFRP.

$$\sigma_1 = \frac{\sigma_x+\sigma_y}{2}\sqrt{\left(\frac{\sigma_x-\sigma_y}{2}\right)^2+\tau_{xy}{}^2} \tag{59}$$

$\sigma_x$ is calculated with Eq. 60 where the moment of inertia ($I_1$) and the distance to the centre of gravity ($x_{cg}$) are calculated for a strengthened and uncracked cross-section. $M_x$ is the bending moment and $N_x$ is the normal force at the position of the end of the CFRP.

$$\sigma_x = \frac{M_x}{I_1}\left(h-x_{cg}\right)+\frac{N_x\left(x_{cg}-\frac{h}{2}\right)}{I_1}\left(h-x_{cg}\right)+\frac{N_x}{bh} \tag{60}$$

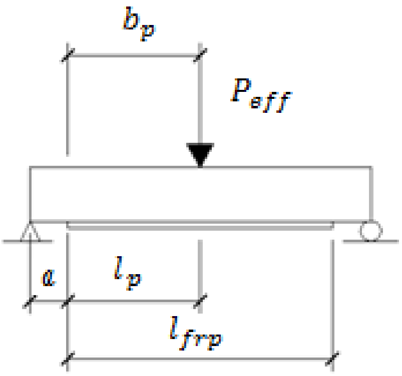



**Figure 18. Corresponding concentrated load and distances for calculations of the shear stress.**

The maximum shear stress ($\tau_{max}$) is calculated with Eq. 61, for an arbitrary located point load. As seen in Figure 18, $P_{eff}$ is the corresponding point load at the middle of the beam that gives the same design moment as the actual moment at the middle of the structure. $P_{eff}$ is therefore seen as the concentrated load in the middle of a simple supported beam. $l_p$ is half of the length of the CFRP and $b_p$ is the length from the end of the CFRP to the concentrated load.

$$\tau_{max} = \left(\frac{P_{eff}}{2}\right)\left(\frac{G_a}{s_a E_{cd} W_c}\right)\left(\frac{2l_p+a+b_p}{a+l_p}\right)\left(\frac{a\lambda_\tau+1}{\lambda_\tau{}^2}\right) \tag{61}$$

According to Figure 18 the distance between the support and the end of the CFRP is defined as $a$. $G_a$ is calculated with Eq. 62, where $E_a$ is the modulus of elasticity for the adhesive, and $\nu$ is Poisson's ratio for the adhesive.

$$G_a = \frac{E_a}{2(1+\nu)} \quad 62$$

$\lambda_\tau$ is calculated with Eq. 63, where $z_0$ is the inner lever arm between the CFRP and the neutral axis at the position of the anchorage for the strengthened cross-section.

$$\lambda_\tau = \sqrt{\frac{G_a b_f}{s_a}\left(\frac{1}{E_f A_f} + \frac{1}{E_{cd} A_c} + \frac{z_0}{E_{cd} W_c}\right)} \quad 63$$

Since it is complicated to find the actual shear stress ($\tau_{xy}$) and the normal stress in the y-direction ($\sigma_y$), the simplification is made that the normal stress in the y-direction is equal to the maximum shear stress ($\tau_{max}$).

### Possible anchorage length – NSMR bars

The minimum bond length ($L_e$) required to achieve the maximum load (see Figure19) for an NSMR bar is calculated with Eq. 64. The factor $\lambda_n$ is calculated with Eq. 65.

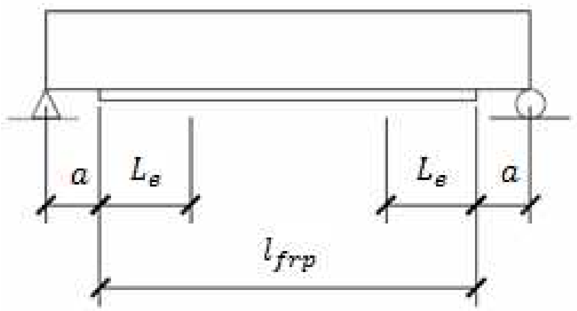



**Figure19. Anchorage length for NSMR**

$$L_e = \frac{\pi}{2\lambda_n} \qquad (\lambda_n\ in\ mm^{-1};\ L_e\ in\ mm) \quad 64$$

$$\lambda_n = \sqrt{\tau_f \frac{L_{per}}{\delta_f E_f A_f}} \qquad (\lambda_n\ in\ mm^{-1}) \quad 65$$

The perimeter of potential failure plane ($L_{per}$) is the contact surface between the concrete and the NSMR bar in a cut (see Figure 20). In Eq. 66, $b_g$ is the width of the groove and $t_g$ is the thickness of the groove.

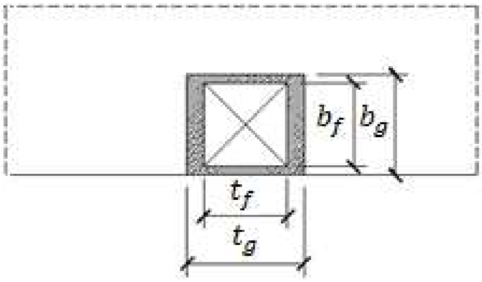


**Figure 20. Geometry of an NSMR-bar.**

$$L_{per} = 2b_g + t_g \qquad 66$$

The bond shear stress ($\tau_f$) and the interfacial bond slip at final fracture ($\delta_f$) (Eq. 67 and 68) are empirically compiled, where $f_{cd}$ is the design value of the compressive strength for concrete.

$$\tau_f = 0.54\sqrt{f_{cd}}{b_f}^{0.4}{t_f}^{0.3} \quad (f_{cd}\ in\ MPa;\ b_f\ in\ mm;\ t_f\ in\ mm;\ \tau_f\ in\ MPa) \qquad 67$$

$$\delta_f = 0.78\frac{{f_{cd}}^{0.27}}{{t_g}^{0.3}} \qquad (f_{cd}\ in\ MPa;\ t_g in\ mm;\ \delta_f in\ mm) \qquad 68$$

## Maximum debonding strength – NSMR bars

If the available bonding length is longer than the required bond length ($L_e$) for an NSMR bar, the force $P_{(\max)L}$ that has to be transferred between the CFRP and the concrete surface without debonding, is calculated according to Eq. 69. If the available bonding length is not long enough, the force $P_{(\max)L}$ is calculated according to Eq. 70.

$$P_{(\max)L} = \tau_f\frac{L_{per}}{\lambda_n} \quad (L_{per}\ in\ mm;\ \lambda_n\ in\ mm^{-1};\ \tau_f\ in\ MPa;\ \ P_{(\max)L}\ in\ kN) \qquad 69$$

$$P_{(\max)L} = \tau_f\frac{L_{per}}{\lambda_n}\sin(\lambda_n L_{beam}) \qquad (L_{beam} in\ mm;\ P_{(\max)L} in\ kN) \qquad 70$$

The maximum bond strength ($P_{(\max)L}$) should be higher than the largest tensile force at the position of the last crack ($F_{fa}$ or $F_{fb}$) calculated with Eq. 57 and 58.

If the maximum force in the CFRP is equalized to the debonding strength ($P_{(\max)L}$) the corresponding maximum tensile strain ($\varepsilon_{max}$) which is allowed in the fibres can be calculated (see Eq. 71). This strain has to be smaller than the ultimate fibre strain ($\varepsilon_f$).

$$\varepsilon_{max} = \tau_f \frac{L_{per}}{E_f A_f \lambda_n} \qquad 71$$

$(\tau_f \ in \ MPa; \ L_{per} in \ mm; \ E_f in \ MPa; \ A_f in \ mm^2; \ \lambda_n \ in-; \ \ \varepsilon_{max} \ in \ \%)$

## 3.5 Shear strengthening

The most common way to strengthen a beam with CFRP in shear is with sheets, as they can be wrapped around the beam as seen in Figure 21. At first some choices have to be made; the number of sheets ($n$), the modulus of elasticity ($E_f$), the thickness ($t_f$), the spacing of the sheets ($s$) and the inclination of the fibres ($\beta$). The equations in this section are from Täljsten et al. (2011), if nothing else is indicated.

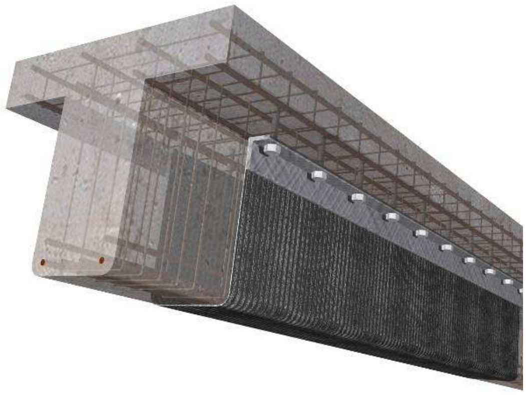


**Figure 21. Beam strengthened in shear with a U-wrap and mechanical anchorage (Täljsten et al., 2011).**

### Characteristic bond length

The characteristic bond length ($l_{ef}$) is the length needed to anchor the sheets, see Figure 22. This length depends on the properties of the chosen sheet and is calculated as in Eq. 72.

$$l_{ef} = \sqrt{\frac{E_f t_f}{2 f_{ctm}}} \qquad (l_{ef} \ in \ mm) \qquad 72$$

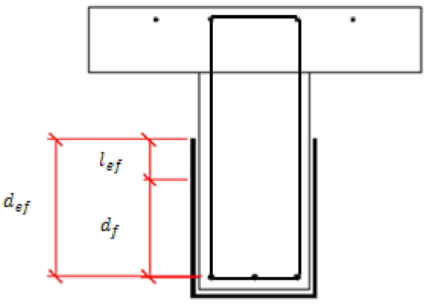



**Figure 22. Distances for anchorage of shear strengthening.**

### Effective height

The effective height of the CFRP ($d_{ef}$) is the available bond length of the CFRP and is given as the lesser one of the available anchorage length and the inner lever arm of the tensile steel reinforcement ($z$) (Eq. 73 and Figure 22).

$$d_{ef} = \min \begin{cases} z \\ d_f - l_{ef} \end{cases} \qquad 73$$

### Anchorage length

The method of anchorage needs to be determined. If the available bond length ($d_{ef}$) is lesser than the needed length ($l_{ef}$), the full bond capacity cannot be achieved, and mechanical anchorage is required.

The effective anchorage length of the CFRP ($L_{ef}$) is calculated with Eq. 74. If the sheets are applied perpendicular to the beam, the effective anchorage length will be the same as the effective height ($d_{ef}$).

$$L_{ef} = d_{ef}(1.0 + cot\beta) \qquad 74$$

### Spacing of sheets

The calculation of the spacing of the sheets measured in a horizontal direction ($s_f$) is based on the width of the sheets ($b_f$) added with the spacing of perpendicular sheets ($r$), and the angle of the fibres ($\beta$), see Figure 23.

$$s_f = \frac{b_f + r}{\sin(\beta)} \qquad 75$$

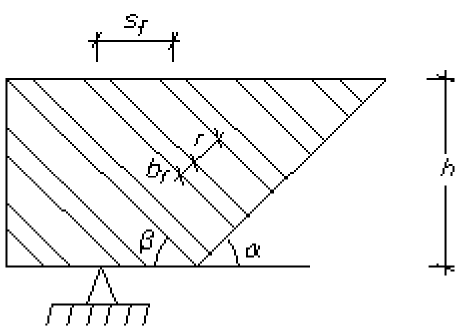



**Figure 23. Spacing of sheets (Täljsten et al., 2011).**

**Cross-sectional area of CFRP**

The cross-sectional area for sheets with a spacing is calculated according to Eq.76.

$$A_f = \frac{2t_f b_f}{s_f} \qquad (A_f \; in \; mm^2/mm) \qquad 76$$

**Effective strain in the fibres**

In case mechanical anchorage is not required, the effective strain in the CFRP is the lesser one of the governing strain due to debonding ($\varepsilon_{fb,d}$) and the ultimate strain of the fibre ($\varepsilon_{fu}$), Eq. 77.

$$\varepsilon_f = \min \begin{cases} \varepsilon_{fb,d} \\ \varepsilon_{fu} \end{cases} \qquad 77$$

If debonding occurs, the effective strain in the fibres is lower than the ultimate strain. The strain due to debonding ($\varepsilon_{fb,d}$) is according to Carolin et al. (2008) dependent on the properties of the concrete and its bonding to the CFRP. $\varepsilon_{fb,d}$ is therefore based on the fracture energy of concrete in the bonded zone ($G_f$) as seen in Eq. 56. $\varepsilon_{fb,d}$ is calculated according to Eq. 78.

$$\varepsilon_{fb,d} = \sqrt{\frac{2G_f}{E_f t_f}} \qquad 78$$

The geometrical factor ($k_b$) in Eq. 56 depends on the width and the spacing of the sheets where $b_f/s_f$ for a not fully covered system should be greater than, or equal to 0.33 (see Eq. 79).

$$k_b = \sqrt{\frac{2 - b_f/s_f}{1 + b_f/s_f}} \geq 1 \qquad 79$$

If mechanical anchorage is required, a higher strain in the fibers can be allowed. The governing strain in the fibres ($\varepsilon_f$) does, however, need to be limited due to non uniform distribution of the shear stresses. A value of the governing strain in the fibre is therefore assumed to 60% of the design ultimate strain, $0.6\varepsilon_{fu}$, based on recommendations from Carolin and Täljsten (2005).

## Shear capacity

The contribution of the CFRP to the shear capacity ($V_{Rd,f}$) is based on the properties of the material used, but also the effective anchorage length ($L_{ef}$) and the effective strain in the CFRP ($\varepsilon_f$). $V_{Rd,f}$ in Eq. 80 can be added to the shear capacity described in Section 3.3.2.

$$V_{Rd,f} = A_f \varepsilon_f E_f L_{ef} sin\beta_f \tag{80}$$

# 4 Case study – Road bridge in Odensberg

## 4.1 General Description

The investigated bridge is situated in Odensberg, south of Falköping, and was built in 1933. The Odensberg Bridge is important to preserve from a cultural historical perspective, and is included in a national sustainable plan for bridges (Ahlberg et al., 2001).

The Odensberg Bridge has a total length of 27.4 meters and is a road bridge that crosses the Swedish west trunk line. It has a middle span of 10.5 meters, and two side spans of 8.45 meters each (see Figure 24). Because the bridge is crossing a railway, and the construction work in 1933 was not allowed to disturb the railway traffic, the two side spans were first constructed as cast-in-place reinforced concrete frames. When they were finished, the steel beams in the middle span could be placed on top of short cantilevers from the concrete columns. Finally, the beams were covered with a cast-in-place bridge deck. In that way, the railway traffic was only disturbed for a short while, when placing the beam across the middle span. The structure of the bridge was thereby influenced by the construction conditions, and it is the specific structure of the bridge that has made it valuable to preserve today (Ahlberg et al., 2001).

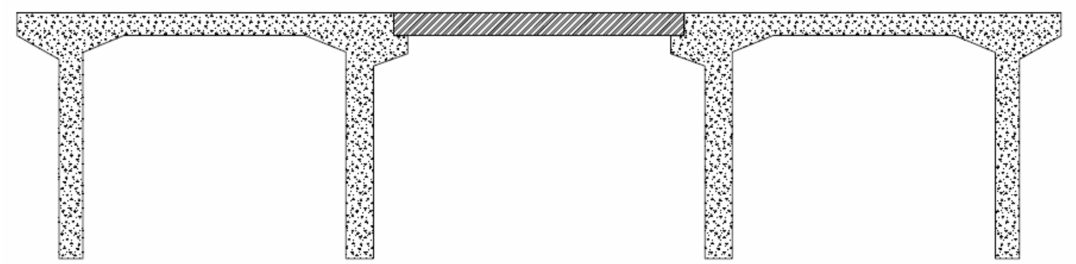


**Figure 24. The Odensberg Bridge**

During the 2000's, the assignment with the Odensberg Bridge was to increase the capacity from Capacity Class 3 (BK 3) to Capacity Class 1 (BK 1). For bridges with a BK 1 capacity, the heaviest vehicle weights are allowed, while for bridges with BK 3 capacity, restrictions are made for the total weight and axle configuration of the vehicles (Vectura, 2010).

Vectura Consulting AB was in 2008 consulted to perform a capacity assessment of the bridge as well as strengthening suggestions. Since the bridge was important to preserve, the strengthening was not allowed to change the appearance of the bridge too much. To be able to strengthen the beams in the side spans without changing their appearance too much, the strengthening was done with CFRP (Vectura, 2010). This case study is limited to an analysis of the four identical reinforced concrete beams in the side spans.

For the real case the flexural strengthening was done along the bottom of the part of the beam with a constant height, with three layers of StoFRP Sheet S300 C300, with a modulus of elasticity of 228 MPa and a characteristic ultimate tensile strain of 1.8%. The shear strengthening was done with two layers of the same product, vertically applied as six U-wraps with a spacing of 100 mm from node 2 (see Figure 25) towards the middle of the beam. Mechanical anchorage with plates was used, as seen in Figure 7.

## 4.2 Material properties

### Concrete and steel

The characteristic values of the strength of the concrete are based on laboratory tests that have been done on the existing concrete. For the classification assessments as well as the capacity analyses done in accordance with Bro2004, the concrete is defined as K35. For the capacity analyses done in agreement with TK Bro, the concrete is defined as C25/30.

$f_{ck} = 25MPa$ (Eurocode/BBK)
$f_{ctk} = 1.8MPa$ (Eurocode/BBK)
$E_{ck} = 31GPa$ (BBK)
$E_{cm} = 31GPa$ (Eurocode)
$\varepsilon_{cu} = 3.5\%$ (Eurocode/ BBK)
$\gamma_c = 1.5$ (Eurocode/ BBK)
$\gamma_{cE} = 1.2$ (Eurocode/BBK)
$f_{yk} = 300MPa$ (Drawings, Appendix 2)
$f_{ywk} = 250MPa$ (Drawings, Appendix 2)
$\gamma_s = 1.15$ (Eurocode/BBK)
$E_s = 200GPa$ (Eurocode)

### CFRP

All material properties of the CFRP strengthening alternatives from the manufacturer Sto Scandinavia AB are presented in Table 1-Table 3 below.

### Laminates

**Table 1. Alternatives with laminates.**

| Name | $E_{fk}[GPa]$ | $\varepsilon_{fu}[\%]$ | $f_{fd}[MPa]$ | $t_f[mm]$ | $b_f[mm]$ |
|---|---|---|---|---|---|
| StoFRP Plate E 50 C | 150 | 1.2 | 1800 | 1.4 | 50 |
| StoFRP Plate E 80 C | 150 | 1.2 | 1800 | 1.4 | 80 |
| StoFRP Plate E 100 C | 150 | 1.2 | 1800 | 1.4 | 100 |
| StoFRP Plate E 120 C | 150 | 1.2 | 1800 | 1.4 | 120 |
| StoFRP Plate E 150 C | 150 | 1.2 | 1800 | 1.4 | 150 |
| StoFRP Plate S 50 C | 163 | 1.6 | 2800 | 1.4 | 50 |
| StoFRP Plate S 80 C | 163 | 1.6 | 2800 | 1.4 | 80 |
| StoFRP Plate S 100 C | 163 | 1.6 | 2800 | 1.4 | 100 |
| StoFRP Plate S 120 C | 163 | 1.6 | 2800 | 1.4 | 120 |
| StoFRP Plate S 150 C | 163 | 1.6 | 2800 | 1.4 | 150 |
| StoFRP Plate IM 50 C | 200 | 0.8 | 2000 | 1.4 | 50 |
| StoFRP Plate IM 80 C | 200 | 0.8 | 2000 | 1.4 | 80 |
| StoFRP Plate IM 100 C | 200 | 0.8 | 2000 | 1.4 | 100 |
| StoFRP Plate IM 120 C | 200 | 0.8 | 2000 | 1.4 | 120 |
| StoFRP Plate IM 150 C | 200 | 0.8 | 2000 | 1.4 | 150 |
| StoFRP Plate M 50 C | 245 | 0.8 | 2000 | 1.4 | 50 |
| StoFRP Plate M 80 C | 245 | 0.8 | 2000 | 1.4 | 80 |
| StoFRP Plate M 100 C | 245 | 0.8 | 2000 | 1.4 | 100 |
| StoFRP Plate M 120 C | 245 | 0.8 | 2000 | 1.4 | 120 |
| StoFRP Plate M 150 C | 245 | 0.8 | 2000 | 1.4 | 150 |

Other important material properties of the alternatives with laminates and the adhesives are:

$\gamma_{frp} = 1.2$ for flexural strengthening according to Féderation International du béton (fib).
$E_a = 7GPa$ Modulus of elasticity of the adhesive.
$s_a = 2mm$ Thickness of the adhesive.
$\nu = 0.2$ Poisson's ratio of the adhesive.

### Sheets

**Table 2. Alternatives with sheets.**

| Name | $\boldsymbol{E_{fk}}[\boldsymbol{GPa}]$ | $\boldsymbol{\varepsilon_{fu}}[\%]$ | $\boldsymbol{f_{fd}}[\boldsymbol{MPa}]$ | $\boldsymbol{t_f}[\boldsymbol{mm}]$ | $\boldsymbol{b_f}[\boldsymbol{mm}]$ |
|---|---|---|---|---|---|
| **StoFRPSheet S 300 C 200** | 240 | 1.7 | 4000 | 0.11 | 300 |
| **StoFRPSheet S 300 C 300** | 240 | 1.7 | 4000 | 0.17 | 300 |
| **StoFRP Sheet IMS S 300 C 300** | 290 | 1.9 | 5500 | 0.17 | 300 |
| **StoFRPSheet M 300 C 200** | 395 | 1.2 | 4600 | 0.11 | 300 |
| **StoFRPSheet M 300 C 300** | 395 | 1.2 | 4600 | 0.17 | 300 |

Other important material properties of the alternatives with sheets and the adhesives are:

$\gamma_{frp} = 1.3$ for flexural strengthening according to Féderation International du béton (fib).
$\gamma_{frp} = 1.35$ for shear strengthening according to Féderation International du béton (fib).
$E_a = 7GPa$ Modulus of elasticity of the adhesive.
$s_a = 1mm$ Thickness of the adhesive.
$\nu = 0.2$ Poisson's ratio of the adhesive.

### Bars

**Table 3. Alternatives with NSMR.**

| Name | $\boldsymbol{E_{fk}}[\boldsymbol{GPa}]$ | $\boldsymbol{\varepsilon_{fu}}[\%]$ | $\boldsymbol{f_{fd}}[\boldsymbol{MPa}]$ | $\boldsymbol{t_f}[\boldsymbol{mm}]$ | $\boldsymbol{b_f}[\boldsymbol{mm}]$ |
|---|---|---|---|---|---|
| **StoFRP Bar E 10 C** | 150 | 1.2 | 1800 | 10 | 10 |
| **StoFRP Bar M 10 C** | 245 | 0.8 | 2000 | 10 | 10 |

Other important material properties of the alternatives with bars are:

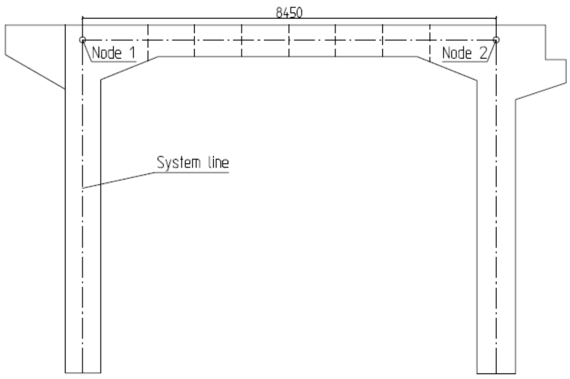
$\gamma_{frp} = 1.2$ for flexural strengthening according to Féderation International du béton (fib).
$b_g = 12\ mm$ Depth of the groove.
$t_g = 14\ mm$ Thickness of the groove.

## 4.3 Geometrical properties

The length of the beam is measured between node 1 and node 2. These nodes are positioned at the system lines that pass straight through the middle of the columns (see Figure 26). The free span of the beam is therefore not the same as the total length of the beam. The column by node 1 has a total width of $700\ mm$ while the column at node 2 has a total width of $800\ mm$.

Further on, the beam does not have a constant height ($h$) along its free span. The beam has a greater height close to the columns which decreases to a constant height closer to the middle of the span.

To manage the analysis of some properties of the beam, the calculations are based on data from nine different coordinates along the beam between node 1 and node 2, as presented in Figure 25.




$$x_{beam} = \begin{pmatrix} 0 \\ 1056 \\ 2113 \\ 3169 \\ 4225 \\ 5281 \\ 6338 \\ 7394 \\ 8450 \end{pmatrix} mm$$

**Figure 25. x-coordinates along the beam.**

The height of the beam ($h$) are presented for the cross-sections of $x_{beam}$. The width of the beam ($b$) is constant and the geometry of the beam is shown in Figure 26.

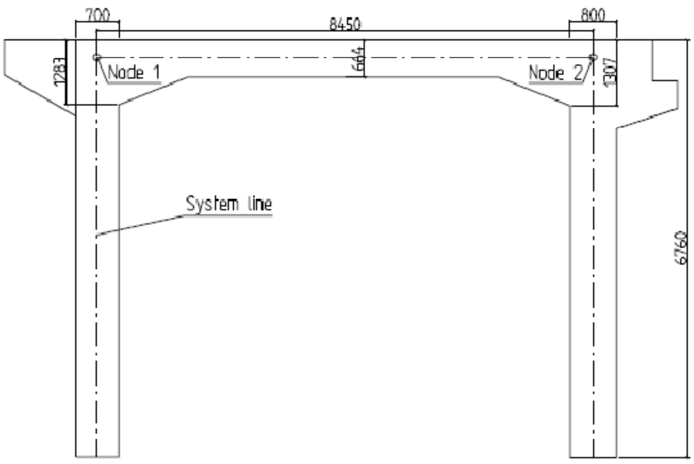



**Figure 26. Geometry of the frame.**

The amount of tensile steel reinforcement is varying along the beam as presented in Figure 27 and Appendix 2.

$$A_s = \begin{pmatrix} 347.64 \\ 1021.61 \\ 2427.47 \\ 2945.24 \\ 2945.24 \\ 2945.24 \\ 2626.18 \\ 855.52 \\ 0 \end{pmatrix} mm^2$$

**Figure 27. The cross-sectional area of the tensile steel reinforcement along the beam.**

| | |
|---|---|
| Cross-sectional area of shear reinforcement: | $A_{sw} = 100.53\ mm^2$ |
| Length of the beam: | $L_{beam} = 8450mm$ |
| Width of cross-section: | $b = 400mm$ |
| Depth of concrete cover: | $c = 30mm$ |
| Diameter of tensile steel reinforcement: | $\phi = 25mm$ |
| Diameter of shear reinforcement: | $\phi_s = 8mm$ |
| Spacing of the shear reinforcement units: | $s = 200mm$ |
| Angle of stirrup with respect to the member axis: | $\alpha = 90°$ |
| Height of flange: | $h_f = 170mm$ |

Based on BBK 04 the angle of concrete compression struts ($\theta$) is set to $45°$ for the case study.

Height of the cross-section:

$$h = \begin{pmatrix} 1282.7 \\ 873.9 \\ 664.0 \\ 664.0 \\ 664.0 \\ 664.0 \\ 664.0 \\ 942.9 \\ 1307.3 \end{pmatrix} mm$$

The inner lever arm for the tensile steel reinforcement ($z$) based on SS-EN 1992-1-1:2005 and BBK 04.

$$z = 0.9d$$

The inner lever arm of the CFRP is ($z_f$) as assumed according to Täljsten et al. (2011):

$$z_f = 0.9h$$

Safety classification 2 will be considered, since the beam has a span shorter than 15 m for Classification 2009 and 10 m for Classification 1998. The partial coefficient for safety classification as well as for the strengthening design done with Bro2004 will therefore be:

$$\gamma_n = 1.1$$

The partial coefficient for safety classification 2 is also defined in agreement with Eurocode and VVFS 2004:43 as:

$$\gamma_d = 0.91$$

The calculations were done in ULS, since fatigue did not need to be considered for road bridges built before 1988. The same holds for Classification 2009.

### 4.4 Loads for classification assessment

The governing loads for the classification assessments are calculated as described in Section 3.1.1. The permanent loads are based on the values below. As each of the side spans of the bridge consist of two identical and parallel beams, each of the beams are assumed to take half of the total permanent load each (Banverket, 2009a).

| | |
|---|---|
| Self-weight of concrete (including steel reinforcement): | $24kN/m^3$ |
| Pavement: | $22kN/m^3$ |
| Fill material: | $20kN/m^3$ |

The bridge is classified for the axle load $A = 130kN$ and the bogie load $B = 200kN$. The bridge is not designed for military vehicles.

The load combination for ULS (Eq. 1) is considered for two cases; the case with the design truck based on the axle load $A$ and the case of the governing design truck based on $B$ (Appendix 1).

The bridge has a total length of 27.4 m, and the braking force is therefore interpolated. Only half of the braking force is considered for each beam. It is assumed that the braking force will not be the most unfavorable live load, and it will therefore result in an interacting live load.

### 4.5 Loads for strengthening design

The loads for the strengthening design done with Bro2004 are equal to the one done with Classification 1998 in Section 4.4. The loads in TK Bro are however defined slightly differently as earlier described in Section 3.1.2. Since the bridge is rather short, it is assumed that the permanent load is lower than the varying traffic load; hence, Eq. 4 is used.

As described earlier, only half of the permanent load is considered for one beam, and the self-weights are defined in Section 4.4. For this bridge, it is assumed that the main live load consists of the traffic load and the braking force. There is therefore no interacting live load. The braking force is assumed to be the same as in the classification assessments.

The capacity of the unstrengthened beam should be calculated for the load combination in ULS, with the A/B values, presented earlier in Section 4.4.

## 4.6 Flexural capacity for classification assessment

The classification assessments in flexure follow the procedures described in Section 3.2.1, with specific conditions, loads, and load combinations according to Section 4.2 - 4.4.

As described earlier, there are two main load combinations that have to be considered for flexural capacity; the case where the design truck is based on the axle load ($A$), and the case where the design truck is based on the bogie load ($B$). Consequently, the capacity analyses are done for both these cases, with the governing bending moment ($M_d$) and the governing normal force ($N_d$) for the considered case.

Finally, if $\Delta F > 0$, strengthening is considered in agreement with TK Bro for Classification 2009 and in agreement with Bro2004 for Classification 1998.

## 4.7 Flexural capacity for strengthening design

The procedure for calculating the flexural capacity is almost identical to the one for the classification assessments, with the differences described in Section 3.2.2. The conditions specific for this case are however based on Eurocode for TK Bro and BBK 04 for Bro2004.

## 4.8 Shear capacity for classification assessment

For the Odensberg bridge the shear capacity is controlled for nine different cases, where the position of the section and the loads are varied. The loads that are included in each of the nine cases are the wheel load, the self-weight, and the superstructure. The loads are derived in Stripstep.

Cases on the left side of the beam: For case 1 and 2, the section is at the column, and the wheel load is $3d$ and $0.9d$ from node 1, in that order. For case 3 and 4 the section is where the height of the beam becomes constant, and where the tensile reinforcement is bent, respectively. The wheel load is placed at the section for the two last cases.

Cases on the right side of the beam: Case 1, 2, 3 and 4 are the same as for the left side. The section for case 5 is at the middle of the beam, with the wheel load placed at the section.

The shear capacity ($V_{Rd}$) is calculated according to Section 3.3.1, and it is assumed that the impact on the capacity due to varying effective depth ($V_i$) does not make any contribution to the capacity of the beam to simplify the calculations in a conservative way.

The allowed load for the member is calculated with the capacity of the beam and the design loads. The loads for the self-weight and the superstructure were factored when calculated in StripStep, but the live load was not. Therefore, the traffic load is factored in Eq. 81 and Eq. 82 to give the allowed load for the specific case.

$$A_{allowed} = \frac{V_{Rd} - V_{sw} - V_{ss}}{1.3\frac{V_{B\_BBK}}{A}} \quad 81$$

$$B_{allowed} = \frac{V_{Rd} - V_{sw} - V_{ss}}{1.3\frac{V_{B\_BBK}}{B}} \quad 82$$

The strengthening that is required for the beam is calculated by choosing the greater difference occurring for $A$ or $B$ in Eq. 81 and 82, see Eq. 83.

$$V_r = max\begin{cases} A - A_{allowed} \\ B - B_{allowed} \end{cases} \quad 83$$

## 4.9 Shear capacity for strengthening design

The shear capacity in agreement with Bro2004 is done in the same way as the capacity assessment in agreement with Classification 1998, as described in Section 4.8. For the shear analysis according to TK Bro, there are also nine different cases to check.

The loads that are included in the analysis are the wheel load, the self-weight of the structure and the weight of the superstructure, all calculated according to TK Bro (Section 3.3.2).

Cases on the left side of the beam: For case 1 and 2, the section is at the column, and the wheel load is $2d$ and $0.5d$ from node 1, in that order. For case 3 and 4 the section is where the height of the beam becomes constant, and where the tensile reinforcement is bent, respectively. The wheel load is for the two last cases placed at the section.

Cases on the right side of the beam: Case 1, 2, 3 and 4 are the same as for the left side. The section for case 5 is at the middle of the beam, with the wheel load placed at the section.

The allowed load for the member is calculated in the same way as in the classification assessments, with the wheel load factored, but with the factors according to Eurocode, Eq. 84, 84 and 83.

$$A_{allowed} = \frac{V_{Rd} - V_{s-w} - V_{ss}}{1.5 \cdot 0.91\frac{V_{traffic\_A}}{A}} \quad 84$$

$$B_{allowed} = \frac{V_{Rd} - V_{s-w} - V_{ss}}{1.5 \cdot 0.91\frac{V_{traffic\_B}}{B}} \quad 85$$

The strengthening that is required for the beam is calculated as earlier with Eq. 83, but by using Eq. 84 and Eq. 85 instead.

## 4.10 Flexural strengthening

The flexural strengthening follows the calculation procedure in Section 3.4. The calculations dealing with the anchorage of the CFRP are calculated for both sides of the beam, as the beam is not symmetrical and not exposed to a symmetrical load. This regards the calculation of the position of the last crack, the anchorage length, and the peeling stress.

The possible anchorage length is calculated from the columns and not from the nodes. To optimize the use of the CFRP, the final positions of the ends of the CFRP are optimized for the peeling stresses.

## 4.11 Shear strengthening

To simplify the analysis, the whole length of the beam will be strengthened in shear for the maximum required strengthening as a result of the strengthening design described in Section 4.9. The beam can only be wrapped with CFRP sheets on three sides because of the slab and will therefore be U-wrapped, as seen in for example Figure 21.

The spacing of the sheets ($r$) is chosen to 100, 200, 300 and 400 mm, and the inclination of the fibres is chosen to 45° or 90°. In Figure 28 an example of strengthening in shear is presented, where the inclination of the fibres is 90°. All the calculated alternatives are presented in Appendix 6.

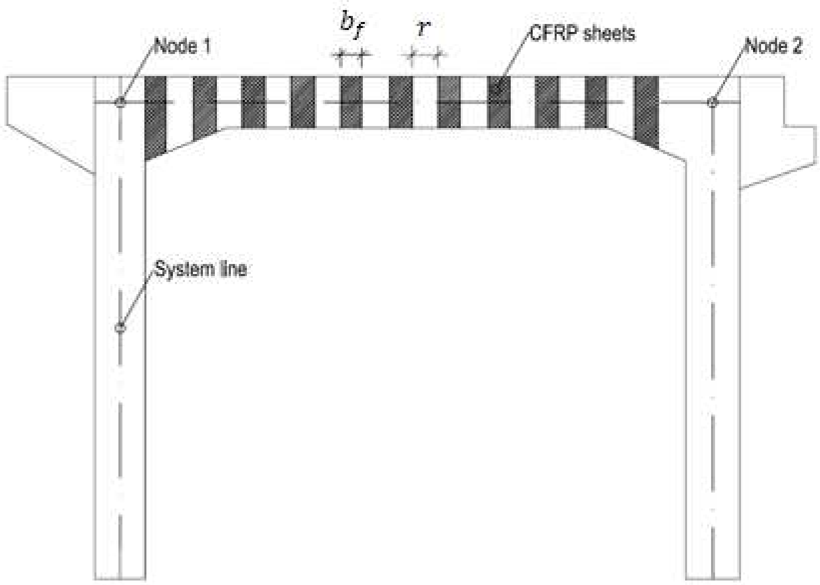



**Figure 28. Strengthening with sheets in shear.**

## 4.12 Economic analysis

The economic analysis is based on an estimation of the total cost per meter of the CFRP. The cost of the labor is approximated to be 112% of the material cost, based on actual costs for other strengthening cases. The cost for scaffolds is not included in the analysis, but it would be approximately the same for any chosen system.

For flexural strengthening, the cost for labor and material is multiplied with the number of layers ($n$) for laminates and the number of bars ($n$) for NSMR. To consider the extra labor when dealing with the wet lay-up system with sheets, additionally 30% of the cost for labor and material is added for each layer (Blanksvärd, 2011).

For shear strengthening, the total length of the CFRP is the length around the beam as a U-wrap multiplied by the number of layers of sheets. To consider the spacing between the sheets, the cost is thereafter calculated for the number of sheets per meter along the beam.

The costs are presented for each of Sto's products in Appendix 7 and Appendix 8.

# 5 Results

## 5.1 Flexural capacity for classification assessment

For the material properties used in the two different classification assessments, see Table 4.

**Table 4. Design values of concrete and reinforcement steel for the classification assessments and Bro2004.**

| Component: | Compressive strength | Tensile strength |
|---|---|---|
| **Concrete** | $f_{cc}$=15.2MPa | $f_{ct}$=1.09 MPa |
| **Steel** | - | $f_{yd}$=244.2 MPa |

The load combination with $A = 130kN$ is not presented in the results as the load combinations with $B = 200kN$ are governing for both the classification assessments (see Appendix 3).

The governing bending moment has increased with 6% for Classification 2009 compared to Classification 1998 because of the additional design trucks. The maximum normal force has increased with 2% for Classification 2009 compared to Classification 1998 (see Appendix 3).

**Table 5. Shortage of flexural capacity according to the classification assessments.**

| | Shortage of flexural capacity $[kN]$ | Length of the beam with shortage of flexural capacity $[mm]$ |
|---|---|---|
| **Classification 1998** | $\Delta F = 37.65kN$ | $L_{\Delta F} = 1587mm$ |
| **Classification 2009** | $\Delta F = 83.66kN$ | $L_{\Delta F} = 2069mm$ |

Flexural strengthening is required according to both the classification assessments as seen in Table 5. The shortage of the flexural capacity for Classification 2009 has increased with 120% compared to Classification 1998, and the shortage is, moreover, present for a longer part of the beam. The shortage in flexural capacity in agreement with Classification 2009 is shown in Figure 30 and in Figure 29 for Classification 1998.

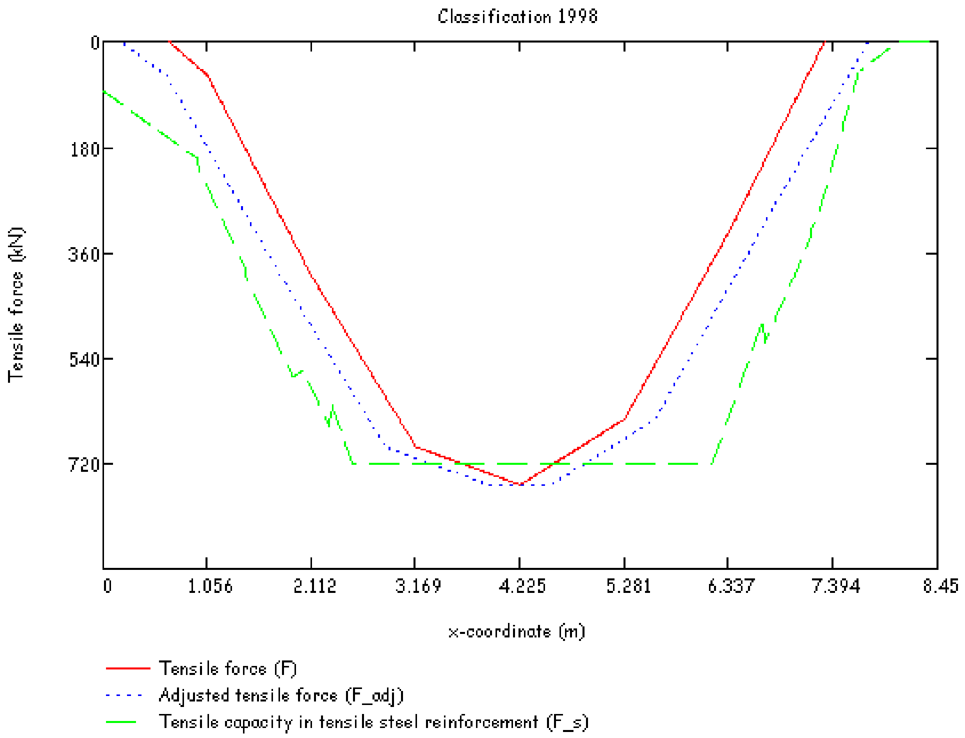



**Figure 29. Tensile force and capacity of the tensile steel reinforcement in accordance with Bro2004 and Classification 1998.**

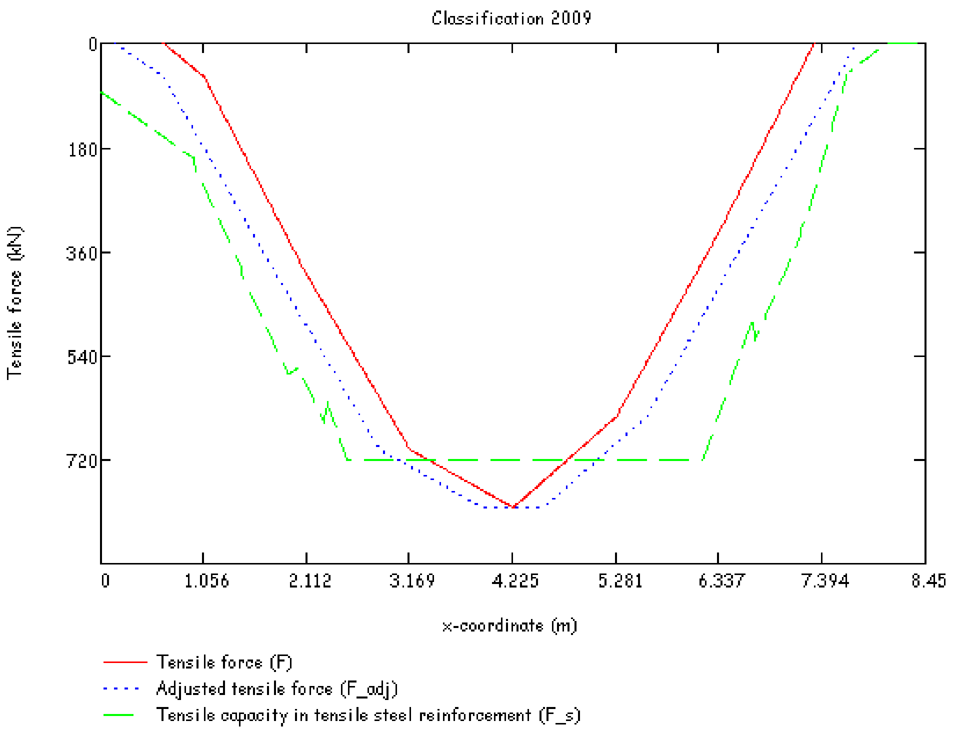



**Figure 30. Tensile force and capacity of the tensile steel reinforcement in accordance with Classification 2009.**

## 5.2 Flexural capacity for strengthening design

The material properties differ between Bro2004 and TK Bro, and the divergence can be seen in Table 4 and Table 6, respectively.

**Table 6. Design values of concrete and reinforcement steel for capacity analysis according to TK Bro.**

| Component: | Compressive strength | Tensile strength |
|---|---|---|
| **Concrete (C25/30)** | $f_{cc}$=16.67MPa | $f_{ct}$=1.20MPa |
| **Steel** | - | $f_{yd}$= 268.6MPa |

The governing bending moment and normal force are the same for the capacity analysis done according to Bro2004 as the capacity analysis done in the classification assessment according to Classification 1998 (see Appendix 3). The governing bending moment and normal force from calculations done with TK Bro are presented in Appendix 3.

The final results from the strengthening design are presented in Figure 29 for Bro2004 and Figure 31 for TK Bro, with the required strengthening presented in Table 7. The required strengthening in accordance with Bro2004 is the same as the required strengthening presented for the classification assessment Classification 1998. The required strengthening for TK Bro is, on the other hand almost insignificant.

**Table 7. Required tensile strengthening according to capacity analyses.**

| | Required tensile strengthening $[\boldsymbol{kN}]$ | Length of the beam that requires tensile strengthening $[\boldsymbol{mm}]$ |
|---|---|---|
| **Bro2004** | $\Delta F = 37.65kN$ | $L_{\Delta F} = 1587mm$ |
| **TK Bro** | $\Delta F = 5.96kN$ | $L_{\Delta F} = 705mm$ |
| **TK Bro (reduced)** | $\Delta F = 86.28kN$ | $L_{\Delta F} = 2453mm$ |

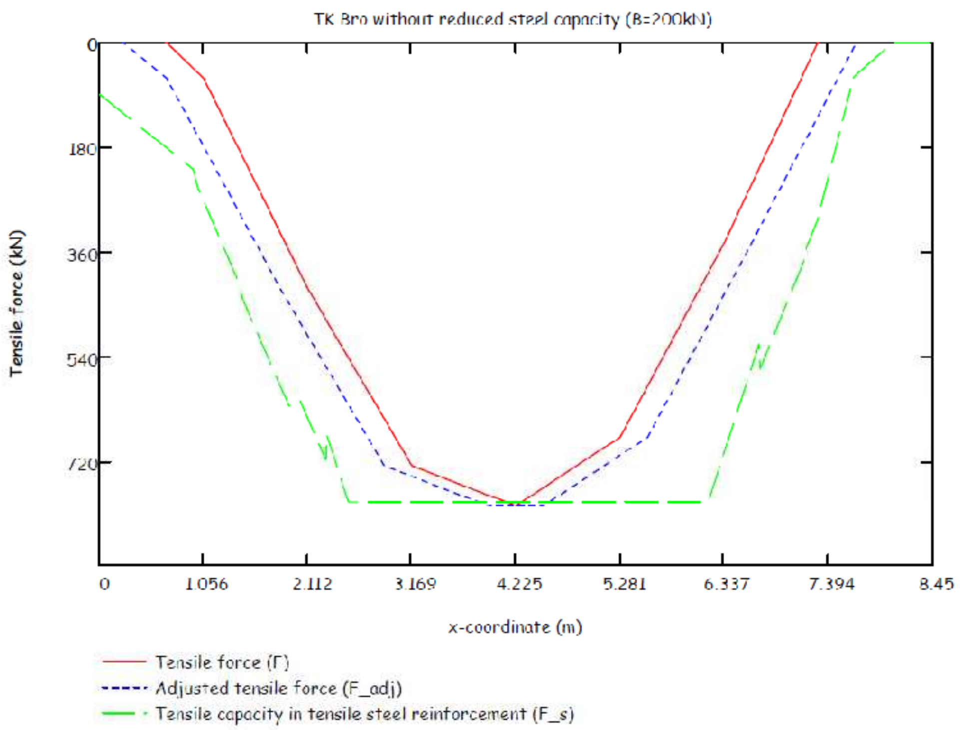



**Figure 31. Tensile force and capacity of the tensile steel reinforcement in accordance with TK Bro.**

The results in agreement with TK Bro show that there is no significant flexural strengthening required (see Table 7 and Figure 30). To be able to continue the analysis with the strengthening, a third case is presented where the tensile strength of the steel reinforcement is reduced with 10%. This reduction results in a required strengthening ($\Delta F$) that corresponds to $\Delta F$ for Classification 2009. The strength of the shear reinforcement is also reduced with 10%.

**Table 8. Design values of concrete and tensile reinforcing steel for capacity analyses according to TK Bro with reduced capacity of the tensile steel reinforcement.**

| **Component:** | **Compressive strength** | **Tensile strength** |
|---|---|---|
| **Concrete (C25/30)** | $f_{cc}$=16.67MPa | $f_{ct}$=1.20MPa |
| **Steel** | - | $f_{yd}$=241.3MPa |

As a result, the required tensile strengthening is increased and required for a longer part of the beam (see Figure 32 and Table 7).

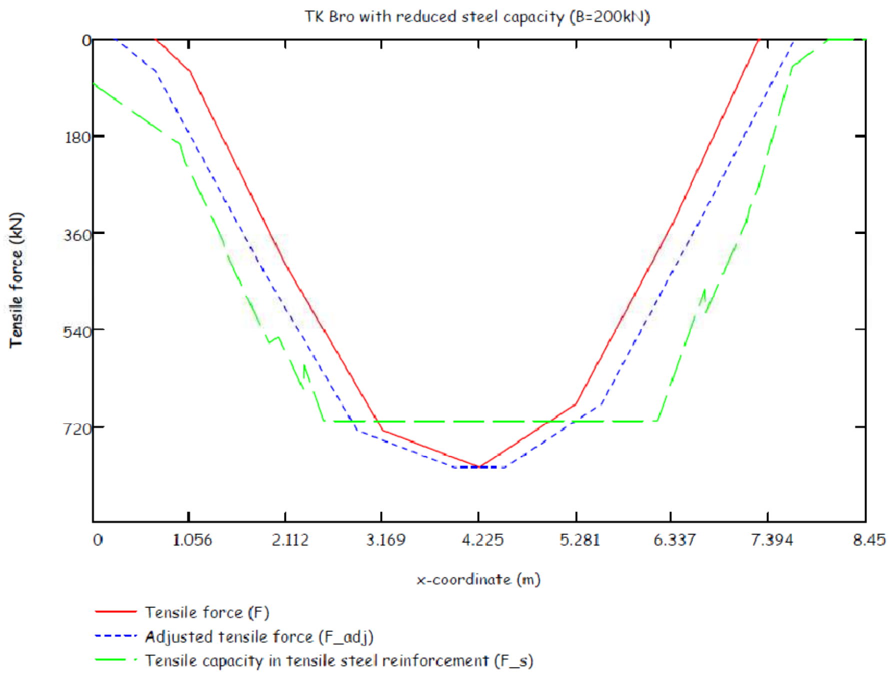



**Figure 32. Tensile force and capacity of the tensile steel reinforcement in accordance with TK Bro for reduced capacity.**

## 5.3 Shear capacity for classification assessment

The design loads for the self-weight, superstructure and the wheel loads, derived in Strip Step, are presented in Appendix 4. The self-weight, superstructure, and wheel load for the axle load *A* are the same for Classification 2009 as for Classification 1998. The only difference from Classification 1998 is that the design load for *B* is higher for Classification 2009 (see Appendix 4).

The allowed load that the structure can carry, as well as the shortage of the shear capacity for the nine different cases, is presented in Appendix 4. As presented in Figure 33, strengthening is required according to both the classification assessments and the maximum required strengthening are identical.

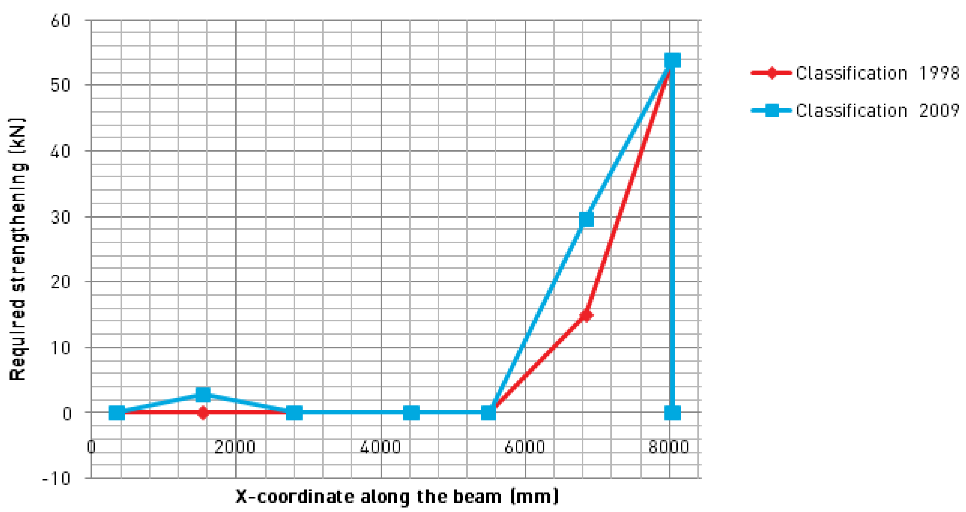



**Figure 33. Shortage of shear capacity according to the classification assessments.**

## 5.4 Shear capacity for strengthening design

The design loads for the different cases, calculated with TK Bro are presented in Appendix 4. The results for the capacity analyses done with Bro2004 are identical to the ones from the classification assessment with Classification 1998 (see Figure 34).

The allowed loads for the governing case with B and the required strengthening are presented in Appendix 4. The required strengthening in agreement with TK Bro is higher than the required strengthening in agreement with Bro2004, which can be seen in Figure 34.

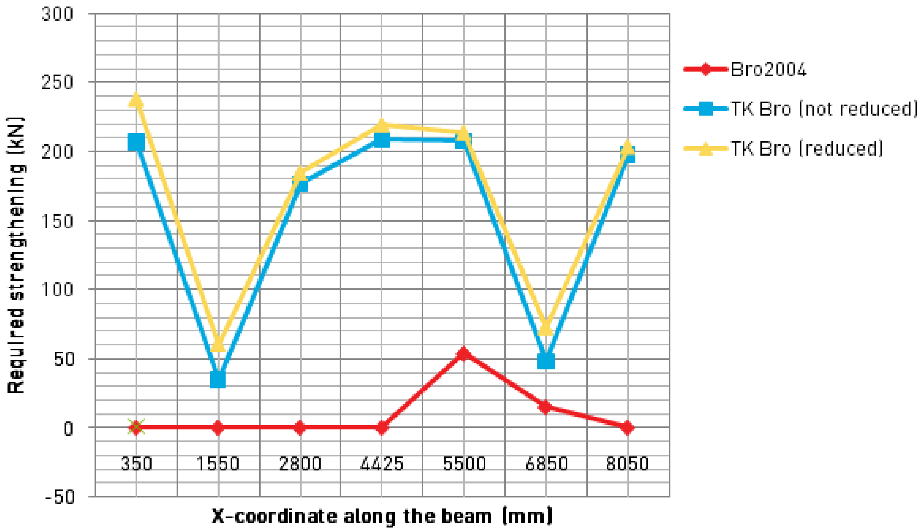



**Figure 34. Required strengthening according to Bro2004 and TK Bro.**

In consistency with the flexural capacity calculations, the tensile strength of the shear reinforcement is also reduced with 10%, as described in Section 5.2 (see Table 9). The results are presented in Appendix 4 and the required strengthening, on which the strengthening analysis is based, is presented in Figure 34.

**Table 9. Design values of shear reinforcement.**

| **Component:** | **Tensile strength** |
|---|---|
| **Steel** | $f_{ywd}$=223.9MPa |
| **Steel (reduced)** | $f_{ywd}$=201.1MPa |

## 5.5 Flexural strengthening

The design values for material properties of the CFRP are presented in Appendix 5. The design value of the tensile strain in the CFRP ($\varepsilon_f$) is governed by the intermediate crack debonding ($\varepsilon_{fd,ic}$) for all alternatives with laminates or sheets. For the alternative with NSMR, on the other hand, $\varepsilon_f$ is the same as the design value of ultimate strain in the CFRP ($\varepsilon_{fu}$).

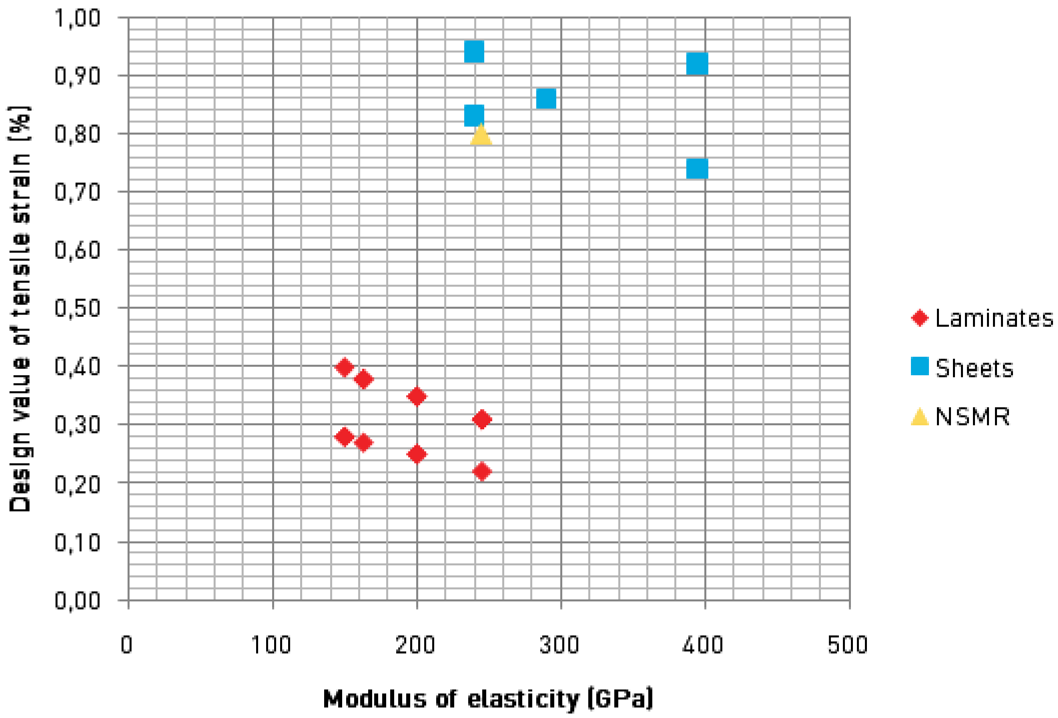



**Figure 35. Design values of tensile strain dependent on the modulus of elasticity.**

All alternatives resulted in a normally reinforced cross-section without concrete crushing except from the NSMR alternative with the product name StoFRP Bar E 10 C with a low modulus of elasticity. The tensile steel reinforcement is yielding for all the strengthening alternatives.

For the system with laminates, there are some alternatives that require more than one layer. According to Blanksvärd (2011) it is possible to apply two layers of laminates, but it is more common to only use one layer. The alternatives that require more than two layers are not included in the result.

### Failure in anchorage because of cracked concrete

The part of the beam between the intersections of the cracking moment ($M_{xcr}$) and the design moment ($M_d$) is cracked (see Figure 36).

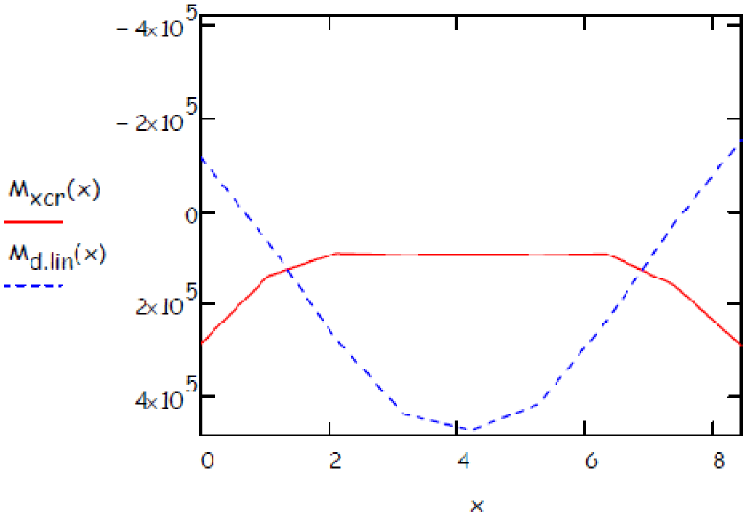



**Figure 36. Intersection of the cracking moment and the design moment.**

The distances from node 1 to the last cracks ($x_{cr}$) (see Figure 37):

Left side of the beam (close to node 1): $x_{cr} = 1350\ mm$
Right side of the beam (close to node 2): $x_{cr} = 6880\ mm$

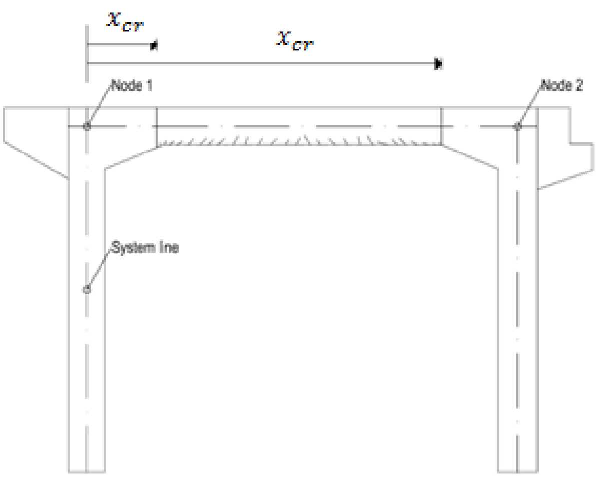



**Figure 37. The distance to the cracks on the left, and on the right side ($x_{cr}$).**

The beam is cracked all the way out to the part of the beam with a varying height, as seen in Figure 36 and Figure 37. Mechanical anchorage is therefore the only solution as the anchorage needs to be done on uncracked concrete, and in this case there is no room for anchorage beyond the last cracks.

To be able to continue the analysis for strengthening without mechanical anchorage, the beam is instead assumed to have a constant height all the way to the columns, as presented in Figure 38. In that way the anchorage length can be analyzed. The results presented above for the failure modes are still relevant since those failure modes are analyzed at the middle of the beam, where the height is constant. The results for the assumed beam, seen in Figure 39, are not relevant for the actual case but are interesting when analyzing the anchorage lengths and costs for the different strengthening alternatives.

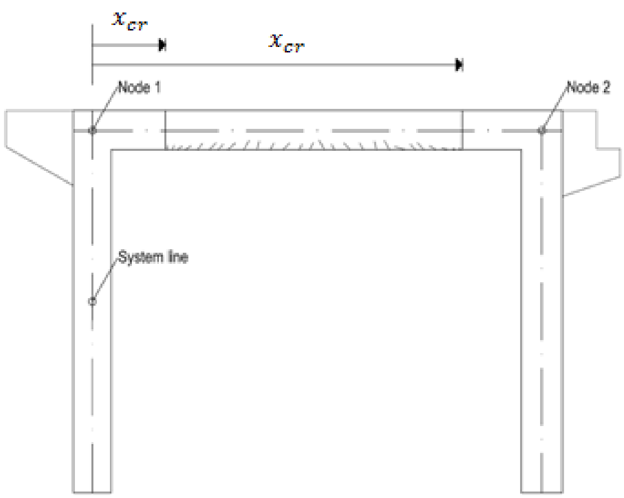



**Figure 38. The distance to the cracks on the left, and on the right side ($x_{cr}$), for the assumed beam with a constant height.**

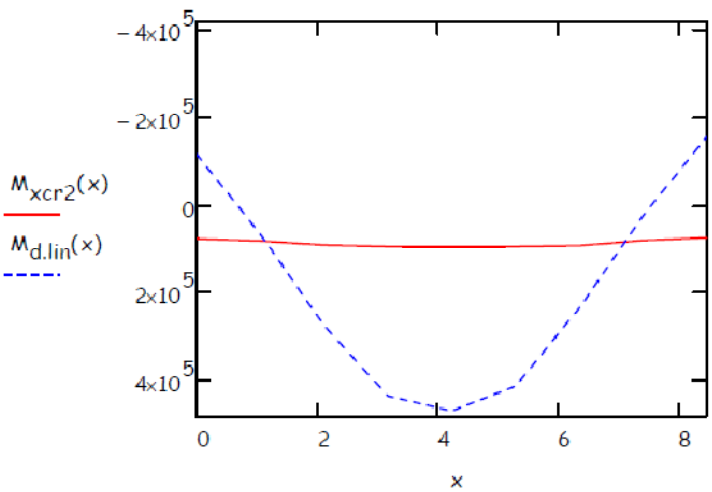



**Figure 39. Intersection of the cracking moment and the design moment, for the assumed beam with a constant height.**

The distances from node 1 to the last cracks ($x_{cr}$) (see Figure 39):

Left side of the beam (close to node 1): $x_{cr} = 1130mm$
Right side of the beam (close to node 2): $x_{cr} = 7100mm$

For the assumed beam with a constant height, the beam is not cracked all the way to the columns, and mechanical anchorage is therefore not necessary.

### Anchorage length for sheets and laminates

The minimum bond length ($l_e$) is greater for the alternatives with laminates than for the alternatives with sheets. It does, however, never exceed the critical anchorage length ($l_{cr}$), meaning that the minimum anchorage length is equal to $l_{cr}$ for all alternatives.

For all alternatives, the peeling stress is too high in case the anchorage is done at the positions of the last crack, with the minimum anchorage length of 250 mm. The anchorage length increases with increasing number of layers due to increased peeling stresses.

As seen in Figure 40, the total required length of the CFRP ($l_{\mathrm{frp}}$) is higher for all alternatives with laminates than for all alternatives with sheets. The variation in total length for the alternatives within each system is however small.

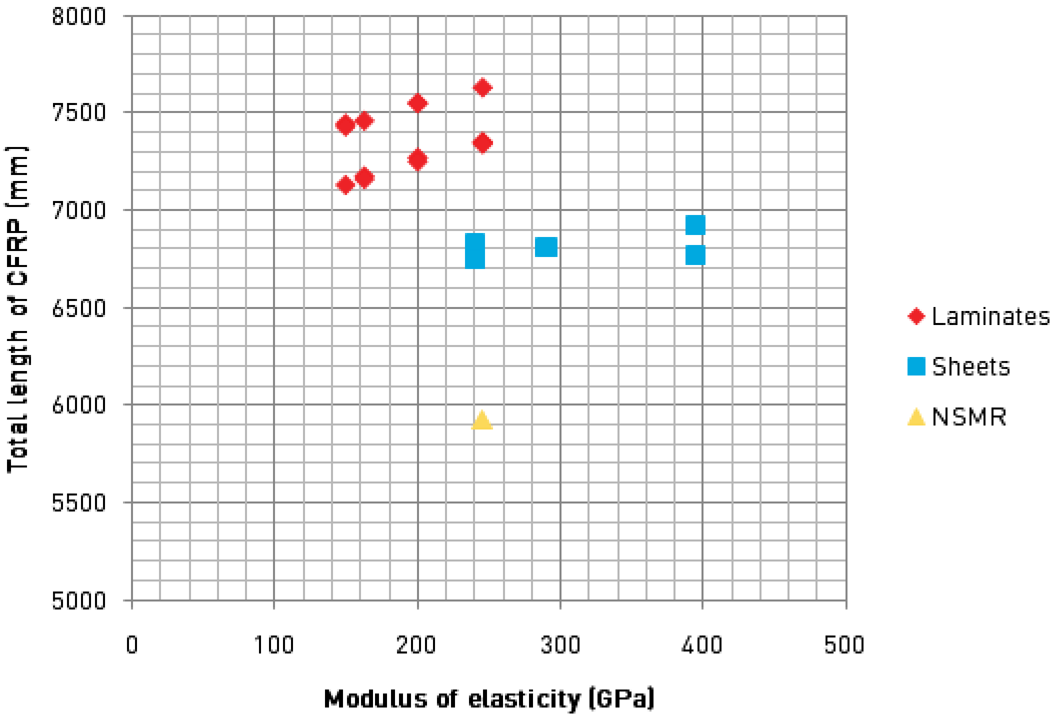



**Figure 40. The total length of CFRP dependent on the modulus of elasticity.**

### Anchorage length for NSMR bars

The results for the alternative with NSMR are presented in Appendix 5. The minimum bond length for the alternative with NSMR ($L_e$) is higher than for the alternatives with laminates or sheets. On the other hand, the fact that the peeling stresses do not have to be considered for the NSMR bar, results in a shorter total length of the NSMR than for the other systems (see Figure 40).

## 5.6 Shear strengthening

**Table 10. Required number of layers for the different alternatives.**

| Name | $E_{fd}$ $[GPa]$ | $\varepsilon_{fuk}$ [%] | $t_f$ $[mm]$ | $b_f$ $[mm]$ | $r$ $[mm]$ | $\varepsilon_f$ [%] | $n$ $[-]$ |
|---|---|---|---|---|---|---|---|
| **StoFRPSheet S 300 C 200** | 240 | 1.7 | 0.11 | 300 | 100 | 0.76 | 4 |
| | | | | | 200 | 0.76 | 5 |
| | | | | | 300 | 0.76 | 6 |
| | | | | | 400 | 0.76 | 7 |
| **StoFRPSheet S 300 C 300** | 240 | 1.7 | 0.17 | 300 | 100 | 0.76 | 3 |
| | | | | | 200 | 0.76 | 3 |
| | | | | | 300 | 0.76 | 4 |
| | | | | | 400 | 0.76 | 5 |
| **StoFRP Sheet IMS S 300 C 300** | 290 | 1.9 | 0.17 | 300 | 100 | 0.84 | 2 |
| | | | | | 200 | 0.84 | 2 |
| | | | | | 300 | 0.84 | 3 |
| | | | | | 400 | 0.84 | 3 |
| **StoFRPSheet M 300 C 200** | 395 | 1.2 | 0.11 | 300 | 100 | 0.53 | 3 |
| | | | | | 200 | 0.53 | 4 |
| | | | | | 300 | 0.53 | 6 |
| | | | | | 400 | 0.53 | 7 |
| **StoFRPSheet M 300 C 300** | 395 | 1.2 | 0.17 | 300 | 100 | 0.53 | 2 |
| | | | | | 200 | 0.53 | 3 |
| | | | | | 300 | 0.53 | 4 |
| | | | | | 400 | 0.53 | 5 |

The shear strengthening is done for fibres with an inclination of 45° and 90°, and with spacing and moduli of elasticity according to Table 10 (see also Appendix 6). For a specific modulus of elasticity, the number of sheets increases with a greater spacing. For the two highest moduli of elasticity, the lowest numbers of layers are needed. The required number of layers is the same for 90º as for 45º when applying the sheets as strips with spacing.

It is not possible to reach the required strengthening with a reasonable number of layers without mechanical anchorage. With mechanical anchorage, more of the ultimate strain in the CFRP can be taken into account, and in that way it is possible to reach the required strengthening. All alternatives are possible to use with mechanical anchorage. The design value of the strain in the CFRP then varies from 0.53 % to 0.84 %, as seen in Table 10. For a low modulus of elasticity the strain is high, while it is low for a high modulus of elasticity. The reason why StoFRP Sheet IMS S 300 C 300 has a higher allowed strain than the other alternatives is because it consists of a different kind of fibre that can take a higher strain.

## 5.7 Economic analysis

In Figure 41 it can be seen that the cost increases with increasing modulus of elasticity, and that the difference between the alternatives is high. The differences in costs for the alternatives with laminates and the same modulus of elasticity depend on the different widths. For sheets, the differences in cost for alternatives with the same modulus of elasticity depend on different thicknesses.

The cheapest alternative is one layer of the product StoFRP Sheet S 300 C 300, with a modulus of elasticity of 240 GPa. The alternatives with sheets are generally cheap, as well as the alternative with NSMR. The alternatives with laminates are generally more expensive.

Appendix 7 presents the total costs for the different flexural strengthening alternatives.

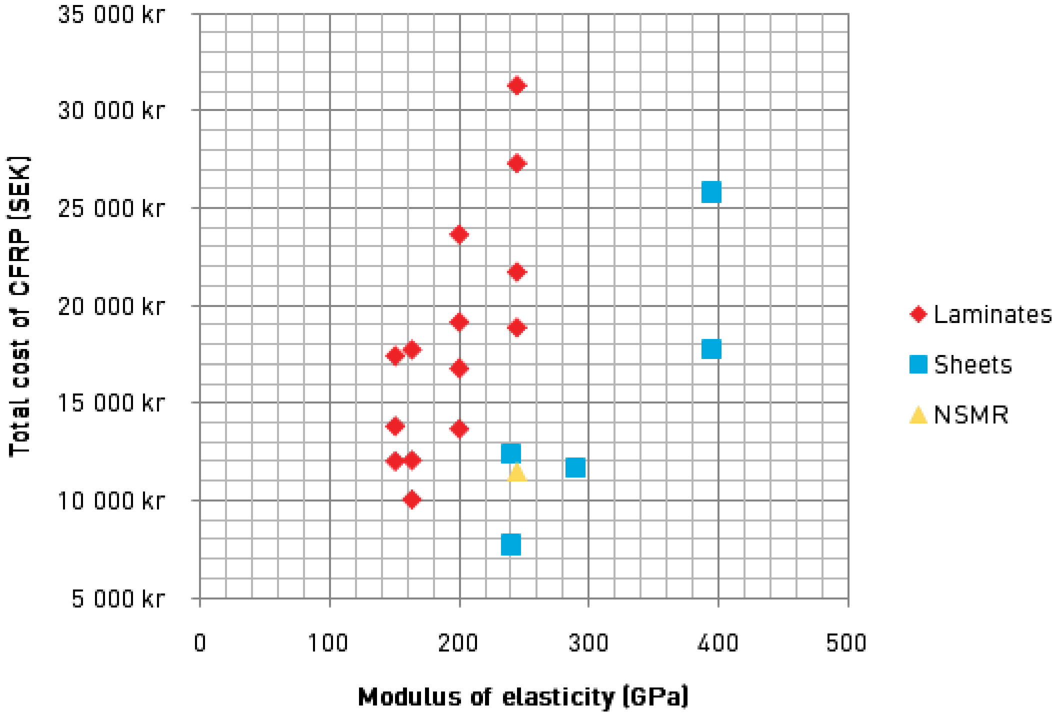



**Figure 41. Total cost for the flexural strengthening alternatives with CFRP.**

In Appendix 8 and Figure 42 the total costs for the different alternatives for strengthening in shear are presented. The alternatives with the two lower moduli of elasticity are cheaper than the alternatives with the highest modulus of elasticity. For the alternatives with the two lower moduli of elasticity, the spacing is not an important variable for the cost. On the other hand, the cost increases with the spacing for the alternatives with the higher modulus of elasticity. For the alternatives with identical moduli of elasticity and spacing, there are two optional thicknesses. The difference in thickness does not seem to have a significant impact on the cost for the alternatives with the same modulus of elasticity and spacing.

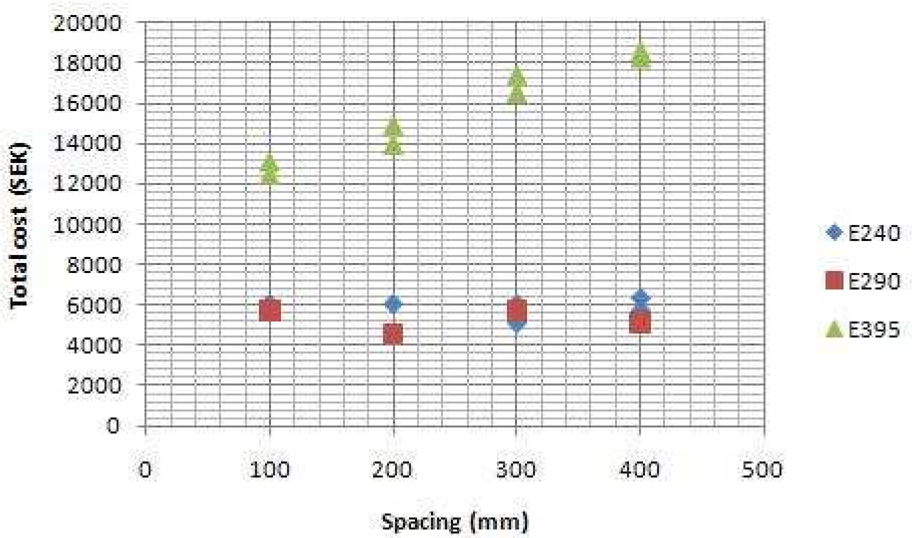



**Figure 42. Cost for the shear strengthening alternatives for fibres in 90º or 45º per meter along the beam .**

# 6 Discussion

## Classification assessment

There is almost no difference between Classification 2009 and Classification 1998 for the flexural capacity. The only difference is the additional design trucks in Classification 2009 that for this case study has an impact on the design load. The bridge has to be strengthened according to both the classification assessments. The shortage of the capacity of the existing structure has increased for Classification 2009, but considering the total capacity of the structure it is not a significant increase.

Both the classification assessments resulted in a required strengthening in shear. The governing shortage is the same for both cases, which means that the additional design trucks have no significant impact on the design load for shear for this case study. Strengthening is however required for a longer part of the beam based on Classification 2009 compared to Classification 1998.

It might be questioned if strengthening in flexure is really necessary, but as strengthening in shear is required, it might be reasonable to strengthen in flexure at the same time. In general, the additional design trucks will lead to a greater number of bridges in need of strengthening, in agreement with the present codes.

## Strengthening design

For the strengthening design, the required strengthening for Bro2004 is identical to the shortage in Classification 1998. This is, however, not the case for TK Bro compared to Classification 2009. More time is consequently required when designing the strengthening with the new codes, as the classification assessment and the strengthening design require different calculations.

The required strengthening in flexure according to TK Bro is significantly lower than for Bro2004. The calculation procedures for the flexural capacity are almost the same for TK Bro and Bro2004. The differences in the results are mostly due to the different ways the codes deal with the safety classification and the loading.

The strengthening design in shear done with TK Bro resulted in a significantly higher required strengthening than the capacity calculation done with Bro2004. The reason for this difference is mainly because of different design models used to calculate the shear capacity. When the bridge was designed in the 1930’s, the capacity of the concrete was included in the shear capacity, which is not allowed for the more conservative model used in TK Bro. Therefore the required strengthening is greater according to TK Bro and might in general result in a lower shear capacity for existing structures.

Considering how the additional design trucks have been included in Classification 2009 but not in TK Bro, it could be appropriate to add them to TK Bro as well. In that way, the resulting required strengthening in TK Bro is more in consistence with the classification assessment.

## Strengthening with CFRP and economic analysis

For flexural strengthening, the tensile strength in the CFRP is for alternatives with laminates or sheets dependent on the risk of intermediate crack debonding ($\varepsilon_{fd,ic}$). For all alternatives with laminates or sheets, the strain ($\varepsilon_f$) is governed by this failure mode. The equation for $\varepsilon_{fd,ic}$ is empirically derived. Even though the definition mostly is based on test results of laminates, it is according to Maalej and Leong (2005) representative for sheets as well. An increase of the thickness of the CFRP increases the interfacial shear stress concentration and reduces the ultimate strain of the CFRP. $\varepsilon_{fd,ic}$ is consequently higher for the alternatives with sheets than for the alternatives with laminates, since the sheets have a lower thickness. From this perspective, it is therefore more advantageous to use systems with low thicknesses. Considering the impact $\varepsilon_{fd,ic}$ has on the results, it might be important to evaluate the definition of $\varepsilon_{fd,ic}$ further.

The alternative with NSMR that failed in concrete crushing had a high allowed strain ($\varepsilon_f$), as the strain is not limited due to intermediate crack debonding. It might be questioned if the strain should be limited in some other way for strengthening systems with NSMR due to debonding. On the other hand, the system with NSMR does probably have a better anchorage than the other systems. This is because the bars are placed in a groove, and are therefore anchored to the concrete on three sides instead of one.

The average value of concrete axial strength is used for calculating the position of the last crack along the beam. It might however be safer to use the characteristic value of concrete axial strength to make sure that anchorage of flexural strengthening is not done on cracked concrete.

The total length of the CFRP is the shortest for the alternative with NSMR, while the alternatives with laminates have the largest lengths. Overall the differences in lengths are not significant for the different alternatives. The total length of the CFRP is governed by peeling stresses for sheets and laminates, while it is governed by debonding for NSMR. The alternatives with high moduli of elasticity are more difficult to anchor and have longer anchorage lengths. The anchorage length for these alternatives does however not have a significant effect on the cost, since the variation of the anchorage length is small.

The choice of CFRP systems does not only depend on the cost. It is for example more reasonable to use NMSR bars when there is a risk that externally applied CFRP strengthening systems might be damaged. A disadvantage with the sheets is that they are partly manufactured at the construction site. There is therefore a greater risk that a mistake will be done and that the system does not get the assigned properties.

Overall for the flexural strengthening, the cost of the alternatives with sheets and laminates increases with the modulus of elasticity. This does mostly depend on the fact that the allowed strain in the fibres is reduced with increased modulus of elasticity. The results show that generally the alternatives with sheets are cheaper.

The conclusions are based on results from one case study, where the economically optimal alternative for flexural strengthening is only one layer of sheets. For other studied cases, the alternatives with sheets might not have been the most economical ones. It might also have been appropriate to base the analysis on more strengthening alternatives, from for example other producers, to get more reliable relationships. Also, it is not always a good choice to strengthen a structure with CFRP, depending on different conditions.

When strengthening in shear, for the alternative with the highest modulus of elasticity, the tensile strain is limited to a greater extent. Almost as many layers are therefore required, compared to the alternatives with lower moduli of elasticity. Since the alternatives with higher moduli of elasticity are more expensive, and almost as many layers are needed, it is more economical to use the cheaper alternatives with lower moduli of elasticity.

For this case study, mechanical anchorage is required to achieve an adequate strengthening in shear. Even if the shear strengthening could have been done without mechanical anchorage, it is probably safer to use mechanical anchorage. Without mechanical anchorage, a sufficient anchorage length ($l_{ef}$) is required to ensure that the anchorage is done beyond shear cracks. It is difficult to estimate the length of the shear cracks and they might continue all the way to the top of the beam. Mechanical anchorage can therefore be a safer choice.

Even though it is possible to strengthen in shear with seven layers, it is recommended to use as most three layers to achieve a good bonding between the layers. Therefore, if it is possible to choose a strengthening alternative that requires fewer layers, it would be a safer choice. In general, the alternatives with three layers or less are cheaper.

The spacing of the sheets does not seem to be an important variable for the cost. The cost does however increase with the spacing for alternatives with the highest modulus of elasticity. Further on, it is not more efficient to apply the sheets with the fibres in 45° inclination than in 90°. It will probably be more expensive to apply the fibres in 45° as it is more complicated to apply the sheets with an inclination. The estimated cost for the alternatives when strengthening in shear is the same as in flexure, even though the labor cost might be higher for the shear strengthening.

The alternative with the product name StoFRP Sheet IMS S300 C300 consists of a certain kind of fibre that can handle a higher tensile strain than the other alternatives. It is therefore in some cases more optimal to use this product even if it has a higher cost.

The estimation that the labor cost is 112% of the material cost is based on actual labor costs for several strengthening cases. The labor cost can however easily change, depending on the experience of the installer and the conditions at the construction site. In some cases, the labor cost might even be 300-400% of the material cost. For this analysis, the labor cost is dependent on the material cost, and will therefore increase for more expensive materials. A better analysis could have been done with either a fixed labor cost, or only the material cost and a factor for the alternatives with sheets due to the extra labor. According to Falldén (2011) the preparatory work is more expensive for the system with NSMR, where grooves have to be cut in the concrete. For laminates and sheets the concrete surface only has to be evened out, and therefore requires less preparation.

The total cost of the strengthening usually depends a lot on the cost for scaffolds, dependent on the situation on the construction site. The cost for scaffolds does not depend on the chosen strengthening system. The material- and labor cost for the mechanical anchorage is not included in the total cost, but would also have been almost the same for any alternative.

Finally, it can be concluded that it is difficult to decide what alternative that is the most economical optimal in general. The results show however that there are big differences in costs for the different alternatives, and an analysis can therefore be appropriate when choosing a system.

### The optimal alternative for the case study

For flexural strengthening, the economically optimal system for the case study would be strengthening with one layer of StoFRP Sheet S300 C300. Mechanical anchorage is required and due to the cracks it might be a good idea to strengthen the whole free span of the beam. In reality, the flexural strengthening was done with three layers of a sheet with a modulus of elasticity of 228 MPa.

For strengthening in shear, the most cost effective alternative is two layers of StoFRP Sheet IMS S300 C300, with a spacing of 200 mm, with the inclination of 90° and mechanical anchorage. In reality, the strengthening in shear was done with two layers of the same sheet as used for flexural strengthening and the spacing of 100 mm.

It is difficult to analyze if the system that was used in reality is more optimal than the systems that are chosen in this thesis. The material used in reality does not exist anymore, and the cost of the material is therefore unknown. Based on the fact that the real strengthening was done with sheets with a low modulus of elasticity, it was probably a cost effective choice, considering the results of this thesis.

### Further studies

Intermediate crack debonding is an important variable when strengthening with sheets and laminates as it limits the strain in the fibres. This limitation is empirically derived and is mostly based on test results of laminates. More studies need to be done to verify that the limitation is utilizable for sheets as well.

So far the focus has been on flexural strengthening with CFRP, but as shear is a complex mechanism, it would be appropriate if more studies were carried out on shear strengthening.

For the economical analysis, the labor costs have only been approximated, but it could be interesting to investigate the costs further, for example by gathering experiences from both different installers and producers of CFRP.

Further on, it could be interesting to evaluate different systems from other perspectives than the economical, for example its long time effects, environmental effects, and abilities in the service limit state.

# 7 Conclusions

- For the case study, the additional design trucks in Classification 2009 result in a higher required flexural strengthening. The required strengthening in shear is on the other hand almost identical to the one for Classification 1998, but is needed for a longer part of the beam.

- The strengthening design in flexure done according to TK Bro results in a significantly lower required strengthening than the analysis done according to Bro2004. The difference does mostly depend on the different design philosophies in the codes. The strengthening design for shear done with TK Bro resulted in a significantly higher required shear strengthening than the strengthening design done with Bro2004. This is mainly because TK Bro uses a more conservative model.

- For this case study almost as many layers are required independently of the modulus of elasticity. Since the alternatives with a higher modulus of elasticity are more expensive it is more economical to use the cheaper alternatives with a lower modulus of elasticity.

- It is more advantageous to use systems with low thicknesses, since a lower thickness of the CFRP results in a higher utilization of the tensile strain of the CFRP, due to intermediate crack debonding.

- The results show that generally the alternatives with sheets are cheaper when strengthening in flexure.

- The alternative with NSMR is cheap compared to the laminates, because of a high allowed tensile strain. The high tensile strain can however result in concrete crushing, as for one alternative in this case study.

- The total length for the alternatives in flexure does not have a significant effect on the cost, since the variation is small.

- For this case study, when strengthening in shear, mechanical anchorage is required for all alternatives, since the required strengthening is too high for a sufficient strengthening without mechanical anchorage.

- The spacing of the sheets does not seem to be an important variable for the cost. The cost does however increase with the spacing for alternatives with the highest modulus of elasticity.

- For the case study, the most economically optimal flexural strengthening would be one layer of StoFRP Sheet S300 C300, with mechanical anchorage. In shear the alternative would be two layers of StoFRP Sheet S300 C300 with mechanical anchorage, an inclination of 90°, and a spacing of 200 mm.

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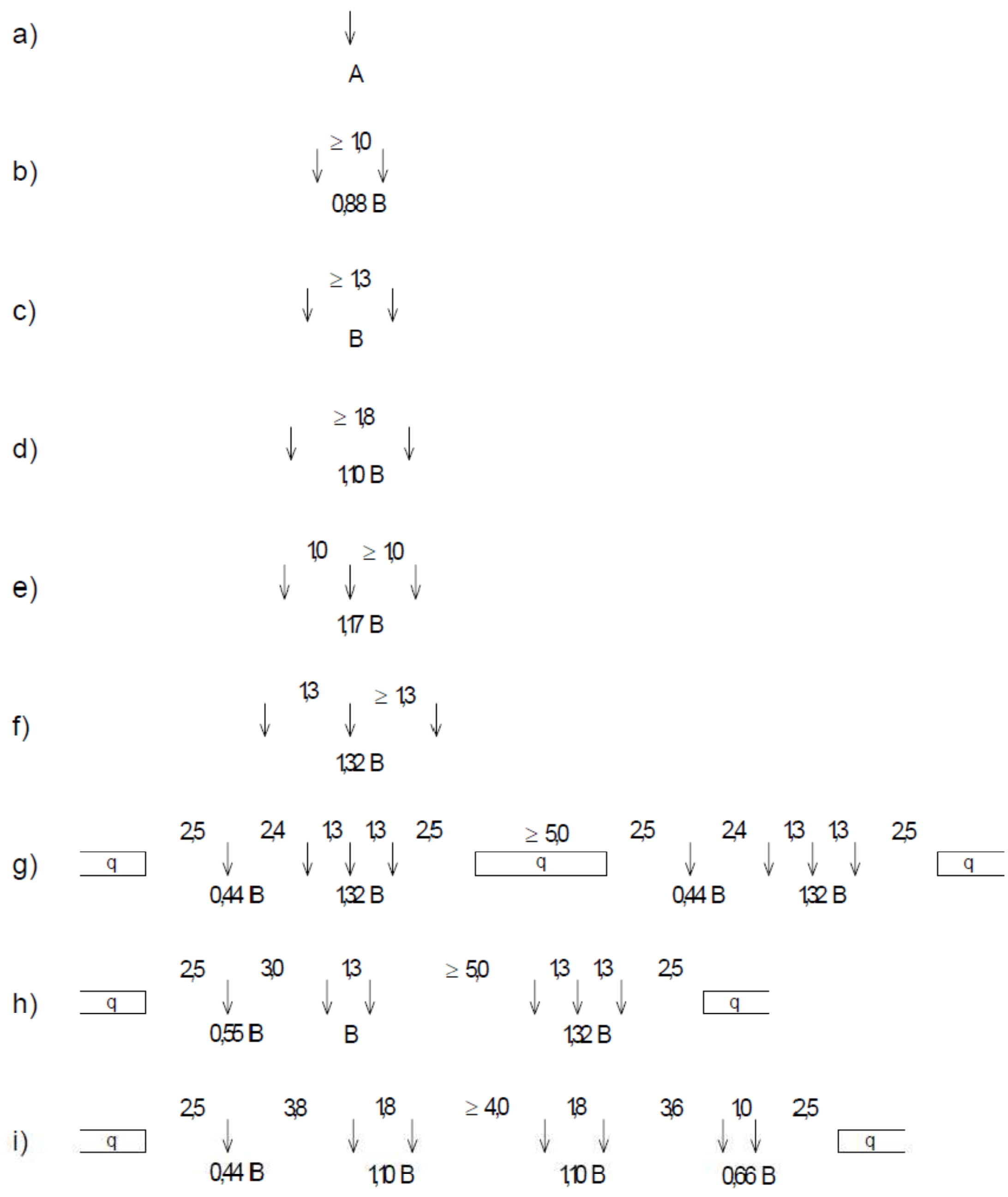
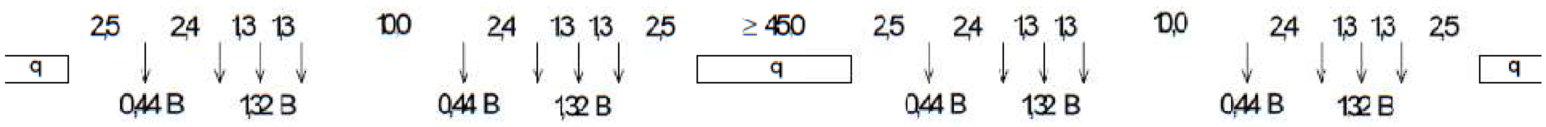
# Appendix

## Appendix 1: Design trucks

Design trucks a-l, according to Classification 1998, Bro2004 and TK Bro.
Design trucks a-n, according to Classification 2009.
(Measurements in meters)

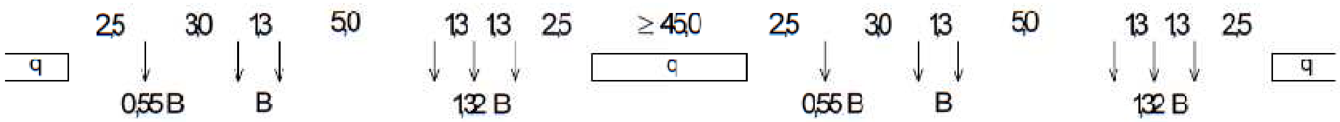


j)



q
2,5 2,4 1,3 1,3 10,0 2,4 1,3 1,3 2,5
0,44 B 1,32 B 0,44 B 1,32 B
≥ 45,0
q
2,5 2,4 1,3 1,3 10,0 2,4 1,3 1,3 2,5
0,44 B 1,32 B 0,44 B 1,32 B
q


k)



q
2,5 3,0 1,3 5,0 1,3 1,3 2,5
0,55 B B 1,32 B
≥ 45,0
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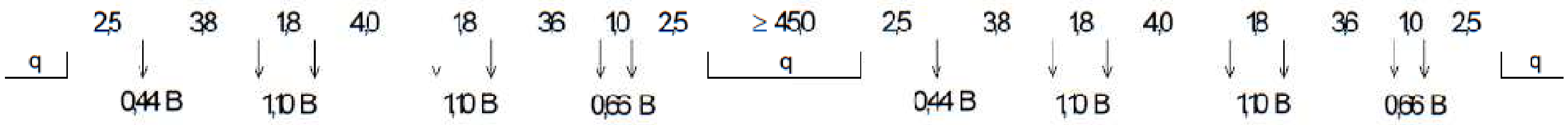
2,5 3,0 1,3 5,0 1,3 1,3 2,5
0,55 B B 1,32 B
q


l)



q
2,5 3,8 1,8 4,0 1,8 3,6 1,0 2,5
0,44 B 1,10 B 1,10 B 0,66 B
≥ 45,0
q
2,5 3,8 1,8 4,0 1,8 3,6 1,0 2,5
0,44 B 1,10 B 1,10 B 0,66 B
q

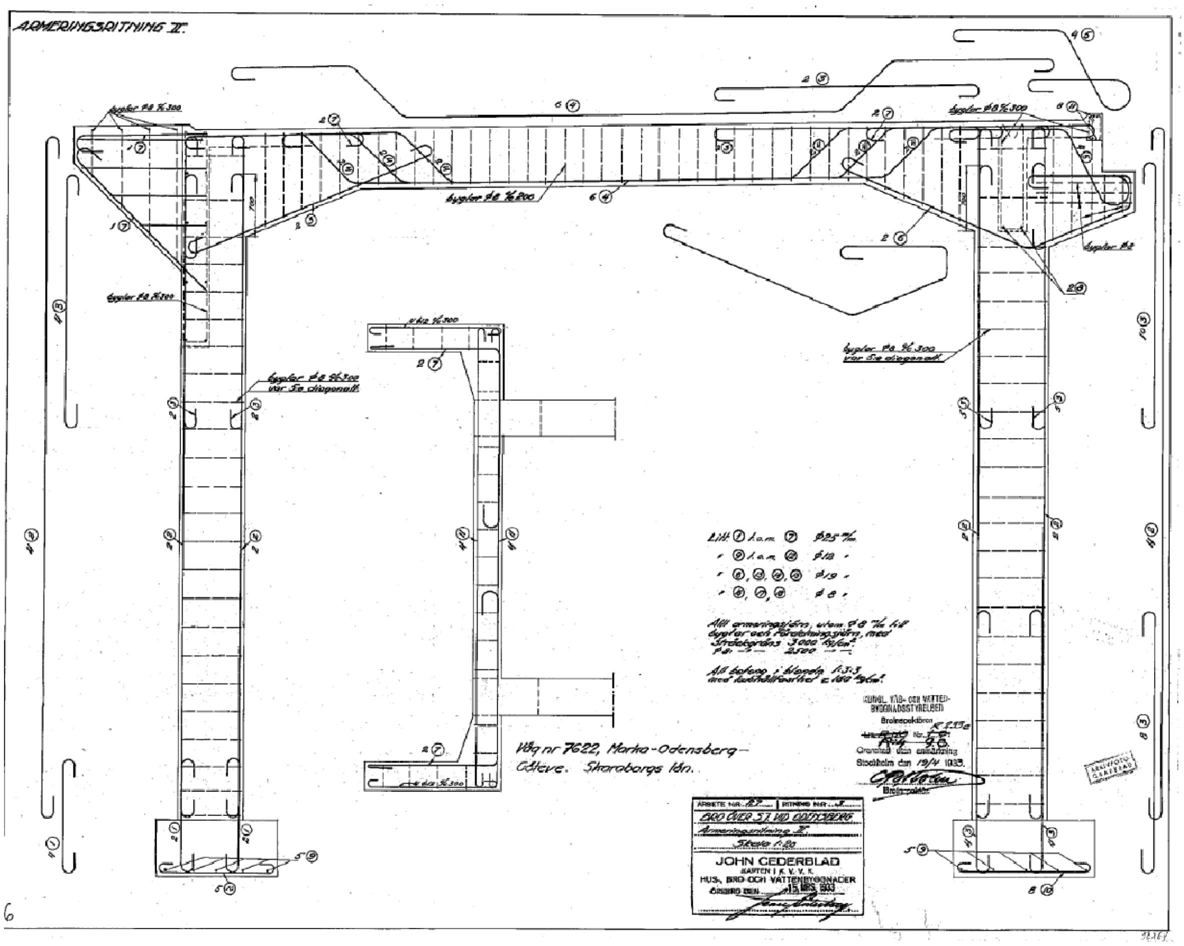

m)

1,3 1,8 3,4 1,8 1,3
0,33 B B 1,32 B


n)

2,0 1,0 1,0 1,0 1,6
0,67 B 1,32 B 0,17 B

# Appendix 2 Drawing of steel reinforcement



# Appendix 3 Results from flexural capacity analyses

$$M_d = \begin{pmatrix} -103.75 \\ 67.16 \\ 263.93 \\ 415.46 \\ \mathbf{447.45} \\ 394.33 \\ 225.55 \\ 23.25 \\ -152.60 \end{pmatrix} kNm \qquad N_d = \begin{pmatrix} -30.69 \\ -44.64 \\ -107.61 \\ -117.57 \\ -120.88 \\ \mathbf{-128.26} \\ -117.85 \\ -104.46 \\ -100.58 \end{pmatrix} kN$$

**Figure 43.The governing bending moment and normal force according to Classification 1998.**

$$M_d = \begin{pmatrix} -97.61 \\ 67.10 \\ 263.87 \\ 423.47 \\ \mathbf{471.08} \\ 396.05 \\ 225.48 \\ 23.16 \\ -152.69 \end{pmatrix} kNm \qquad N_d = \begin{pmatrix} -22.09 \\ -44.67 \\ -107.63 \\ -133.65 \\ -134.09 \\ \mathbf{-134.76} \\ -117.88 \\ -104.49 \\ -100.61 \end{pmatrix} kN$$

**Figure 44. The governing bending moment and normal force according to Classification 2009.**

$$F_s = \begin{pmatrix} 84.43 \\ 249.35 \\ 592.28 \\ 719.22 \\ \mathbf{719.22} \\ 719.22 \\ 641.30 \\ 209.06 \\ 0.0 \end{pmatrix} kN \qquad F_s = \begin{pmatrix} 88.97 \\ 273.352 \\ 647.26 \\ 791.14 \\ \mathbf{791.14} \\ 791.14 \\ 705.44 \\ 231.23 \\ 0.0 \end{pmatrix} kN \qquad F_s = \begin{pmatrix} 83.90 \\ 246.56 \\ 585.86 \\ 710.82 \\ \mathbf{710.82} \\ 710.82 \\ 633.81 \\ 206.48 \\ 0.0 \end{pmatrix} kN$$

**Figure 45. Tensile yield force in tensile steel reinforcement ($\mathbf{F_s}$) in accordance with Bro2004, TK Bro (not reduced capacity) and TK Bro (reduced capacity) , in that order.**

$$M_d = \begin{pmatrix} -114.37 \\ 68.54 \\ 277.89 \\ 438.56 \\ \mathbf{472.73} \\ 416.56 \\ 237.97 \\ 26.94 \\ -153.77 \end{pmatrix} kNm \qquad N_d = \begin{pmatrix} -34.54 \\ -42.88 \\ -109.00 \\ -119.46 \\ -122.94 \\ \mathbf{-130.69} \\ -119.76 \\ -132.76 \\ -128.68 \end{pmatrix} kN$$

**Figure 46. The governing bending moment and normal force in accordance with TK Bro.**

# Appendix 4 Results from shear capacity analyses

**Table 11. Design loads according to Publikation1998:78 for the left side of the beam.**

| **Classification 1998 – Design load, left side** | | | | |
|---|---|---|---|---|
| **Case** | Self-weight ($V_{s-w}$) (kN) | Superstructure ($V_{ss}$) (kN) | Wheel load ($V_{traffic\_A}$) ($A = 130kN$) | Wheel load ($V_{traffic\_B}$) ($B = 200kN$) |
| **1** | 89.35 | 9.12 | 70.51 | 95.83 |
| **2** | 89.35 | 9.12 | 112.11 | 197.52 |
| **3** | 86.51 | 8.37 | 109.43 | 189.53 |
| **4** | 28.46 | 2.04 | 81.22 | 116.88 |

**Table 12. Design loads according to Publikation1998:78 for the right side of the beam.**

| **Classification 1998 – Design load, right side** | | | | |
|---|---|---|---|---|
| **Case** | Self-weight ($V_{s-w}$) (kN) | Superstructure ($V_{ss}$) (kN) | Wheel load ($V_{traffic\_A}$) ($A = 130kN$) | Wheel load ($V_{traffic\_B}$) ($B = 200kN$) |
| **1** | 112.53 | 12.93 | 74.58 | 122.73 |
| **2** | 112.53 | 12.93 | 114.42 | 207.47 |
| **3** | 109.40 | 12.19 | 112.00 | 199.49 |
| **4** | 58.88 | 6.84 | 89.65 | 150.85 |
| **5** | 13.67 | 0.78 | 72.62 | 99.45 |

**Table 13. Design loads for B = $200$kN, according to Classification 2009.**

| **Classification 2009 - design load, wheel load ($\boldsymbol{B = 200kN}$)** | | |
|---|---|---|
| **Case** | Wheel load ($B = 200kN$) Left side | Wheel load ($B = 200kN$) Right side |
| **1** | 96.01 | 122.73 |
| **2** | 217.44 | 226.09 |
| **3** | 208.01 | 216.67 |
| **4** | 123.96 | 150.85 |
| **5** | - | 101.48 |

**Table 14. Shear strength according to Classification 1998 and Classification 2009 for the left side of the beam.**

| **Classification 1998 & Classification 2009 – Shear strength of the beam (left side)** | | | |
|---|---|---|---|
| **Case** | Strength of concrete ($V_c$)(kN) | Strength of steel reinforcement ($V_s$)(kN) | Total strength (kN) |
| **1** | 227.40 | 103.43 | 330.83 |
| **2** | 291.39 | 103.43 | 394.82 |
| **3** | 135.52 | 112.96 | 248.48 |
| **4** | 135.52 | 56.48 | 192.00 |

**Table 15. Shear strength according to Classification 1998 and Classification 2009 for the right side of the beam.**

| Classification 1998 & Classification 2009 – Shear strength of the beam (right side) | | | |
|---|---|---|---|
| **Case** | Strength of concrete ($V_c$)(kN) | Strength of steel reinforcement ($V_s$)(kN) | Total strength (kN) |
| **1** | 211.34 | 103.43 | 314.77 |
| **2** | 401.79 | 103.43 | 505.22 |
| **3** | 135.52 | 226.00 | 361.52 |
| **4** | 152.47 | 56.48 | 208.95 |
| **5** | 135.52 | 56.48 | 192.00 |

**Table 16. Allowed load according to Classification 1998 for the left side of the beam.**

| Classification 1998 - Allowed load for B=200 kN | | |
|---|---|---|
| **Case** | Left side (kN) | Right side (kN) |
| **1** | 373.03 | 237.30 |
| **2** | 236.30 | 281.60 |
| **3** | 216.44 | 185.04 |
| **4** | 216.40 | 146.07 |
| **5** | - | 274.67 |

**Table 17. Required strengthening for Classification 1998.**

| Classification 1998 - Strengthening required, $V_r$ (kN) | | |
|---|---|---|
| **Case** | Leftside (kN) | Right side (kN) |
| **1** | 0 | 0 |
| **2** | 0 | 0 |
| **3** | 0 | 14.96 |
| **4** | 0 | 53.93 |
| **5** | - | 0 |

**Table 18. Allowed load for B=200kN according to Classification 2009.**

| Classification 2009 - Allowed load for B=200 kN, left side | | |
|---|---|---|
| **Case** | Leftside (kN) | Right side (kN) |
| **1** | 372.33 | 237.30 |
| **2** | 214.66 | 258.41 |
| **3** | 197.21 | 170.37 |
| **4** | 200.44 | 146.07 |
| **5** | - | 269.17 |

**Table 19. Required strengthening for Classification 2009.**

| Classification 2009 - Strengthening required, $V_r$ (kN) | | |
|---|---|---|
| **Case** | Left side (kN) | Right side (kN) |
| **1** | 0 | 0 |
| **2** | 0 | 0 |
| **3** | 2.79 | 29.63 |
| **4** | 0 | 53.93 |
| **5** | - | 0 |

**Table 20. Design loads according to TK Bro, for the left side of the beam.**

| **TK Bro – Design loads, left side** | | | | |
|---|---|---|---|---|
| **Case** | Self-weight ($V_{s-w}$) (kN) | Superstructure ($V_{ss}$) (kN) | Wheel load ($V_{traffic\_A}$) ($A = 130kN$) | Wheel load ($V_{traffic\_B}$) ($B = 200kN$) |
| **1** | 107.23 | 9.12 | 91.42 | 141.65 |
| **2** | 107.23 | 9.12 | 118.66 | 217.21 |
| **3** | 104.17 | 8.37 | 109.43 | 189.53 |
| **4** | 41.53 | 2.04 | 81.22 | 116.88 |

**Table 21. Design loads according to TK Bro, for the right side of the beam.**

| **TK Bro – Design loads, right side** | | | | |
|---|---|---|---|---|
| **Case** | Self-weight ($V_{s-w}$) (kN) | Superstructure ($V_{ss}$) (kN) | Wheel load ($V_{traffic\_A}$) ($A = 130kN$) | Wheel load ($V_{traffic\_B}$) ($B = 200kN$) |
| **1** | 110.61 | 12.93 | 94.96 | 160.85 |
| **2** | 110.61 | 12.93 | 120.34 | 228.40 |
| **3** | 107.23 | 12.19 | 112.00 | 199.49 |
| **4** | 52.72 | 6.84 | 89.65 | 150.85 |
| **5** | 11.74 | 4.37 | 77.85 | 128.97 |

**Table 22. Shear capacity of the beam without and with shear reinforcement, left side.**

| **TK Bro - Shear strength of the beam, left side ($V_{Rd}$)** | | |
|---|---|---|
| **Case** | $V_{Rd,c}$ (kN) | $V_{Rd}$(kN) |
| **1** | 137.00 | 113.77 |
| **2** | 137.00 | 113.77 |
| **3** | 106.86 | 325.86 |
| **4** | 101.62 | 62.13 |

**Table 23. Shear capacity of the beam without and with shear reinforcement, right side.**

| **TK Bro - Shear strength of the beam, right side ($V_{Rd}$)** | | |
|---|---|---|
| **Case** | $V_{Rd,c}$ (kN) | $V_{Rd}$(kN) |
| **1** | 152.48 | 113.77 |
| **2** | 152.48 | 113.77 |
| **3** | 116.61 | 325.84 |
| **4** | 103.39 | 62.13 |
| **5** | 102.62 | 62.13 |

**Table 24. Allowed load for B=200 kN, with and without shear reinforcement, according to TK Bro, for the left side of the beam.**

| TK Bro - Allowed load for B=200 kN, left side | | |
|---|---|---|
| **Case** | Allowed load for structure without shear reinforcement (kN) | Allowed load for structure with shear reinforcement included (kN) |
| 1 | 21.36 | -2.67 |
| 2 | 13.93 | -6.95 |
| 3 | -4.39 | 164.90 |
| 4 | 72.77 | 23.26 |

**Table 25. Allowed load according to TK Bro for the right side of the beam.**

| TK Bro- Allowed load for B=200 kN, right side | | |
|---|---|---|
| **Case** | Allowed load for structure without shear reinforcement (kN) | Allowed load for structure with shear reinforcement included (kN) |
| **1** | 26.36 | -8.90 |
| **2** | 17.14 | -7.69 |
| **3** | -2.07 | 151.61 |
| **4** | 42.57 | 2.49 |
| **5** | 98.28 | 44.70 |

**Table 26. Required strengthening for TK Bro needed for the different cases along the beam.**

| TK Bro- Strengthening required, $V_r$ (kN) | | |
|---|---|---|
| **Case** | Left side (kN) | Right side (kN) |
| **1** | 202.67 | 208.90 |
| **2** | 206.95 | 207.69 |
| **3** | 35.10 | 48.39 |
| **4** | 176.74 | 197.51 |
| **5** | - | 155.30 |

## TK Bro - reduced load

**Table 27. Shear capacity of the beam without and with shear reinforcement, left side.**

| TK Bro (reduced capacity)- Shear strength of the beam, left side ($V_{Rd}$) | | |
|---|---|---|
| **Case** | $V_{Rd,c}$ (kN) | $V_{Rd}$(kN) |
| **1** | 137.00 | 102.22 |
| **2** | 137.00 | 102.22 |
| **3** | 106.86 | 292.76 |
| **4** | 101.62 | 55.82 |

**Table 28. Shear capacity of the beam without and with shear reinforcement, right side.**

| TK Bro (reduced capacity)- Shear strength of the beam, right side($V_{Rd}$) | | |
|---|---|---|
| **Case** | $V_{Rd,c}$ (kN) | $V_{Rd}$(kN) |
| **1** | 152.48 | 102.22 |
| **2** | 152.48 | 102.22 |
| **3** | 116.61 | 292.76 |
| **4** | 103.39 | 55.82 |
| **5** | 102.62 | 55.82 |

**Table 29. Allowed load for B=200 kN, with and without shear reinforcement, according to TK Bro, for the left side of the beam.**

| TK Bro (reduced capacity) - Allowed load for B=200 kN, left side | | |
|---|---|---|
| **Case** | Allowed load for structure without shear reinforcement (kN) | Allowed load for structure with shear reinforcement included (kN) |
| 1 | 21.36 | -14.61 |
| 2 | 13.93 | -38.12 |
| 3 | -4.39 | 139.32 |
| 4 | 72.77 | 15.36 |

**Table 30. Allowed load according to TK Bro for the right side of the beam.**

| TK Bro (reduced capacity)- Allowed load for B=200 kN, right side | | |
|---|---|---|
| **Case** | Allowed load for structure without shear reinforcement (kN) | Allowed load for structure with shear reinforcement included (kN) |
| **1** | -21.38 | -19.42 |
| **2** | -16.87 | -13.68 |
| **3** | 147.40 | 127.31 |
| **4** | -3.97 | -3.63 |
| **5** | 48.58 | 38.57 |

**Table 31. Required strengthening for TK Bro needed for the different cases along the beam.**

| TK Bro (reduced capacity) – Required strengthening$V_r$ (kN) | | |
|---|---|---|
| **Case** | Left side (kN) | Right side (kN) |
| **1** | 214.61 | 219.42 |
| **2** | 238.12 | 213.68 |
| **3** | 60.68 | 72.69 |
| **4** | 184.64 | 203.63 |
| **5** | - | 161.43 |

# Appendix 5 Strengthening in flexure

**Table 32. Flexural strengthening for alternatives with laminates.**

| Name | $E_{fd}[GP$ | $\varepsilon_{fuk}[\%$ | $f_{fd}[MPa]$ | $n\,[-]$ | $A_f[mm^2]$ | $\varepsilon_f[\%]$ | $F_f[kN]$ |
|---|---|---|---|---|---|---|---|
| StoFRP Plate E 100 C | 150 | 1.2 | 1800 | 2 | 280 | 0.28 | 99.02 |
| StoFRP Plate E 120 C | 150 | 1.2 | 1800 | 2 | 336 | 0.28 | 118.83 |
| StoFRP Plate E 150 C | 150 | 1.2 | 1800 | 1 | 210 | 0.40 | 105.03 |
| StoFRP Plate S 100 C | 163 | 1.6 | 2800 | 2 | 280 | 0.27 | 103.23 |
| StoFRP Plate S 120 C | 163 | 1.6 | 2800 | 1 | 168 | 0.38 | 87.59 |
| StoFRP Plate S 150 C | 163 | 1.6 | 2800 | 1 | 210 | 0.38 | 109.49 |
| StoFRP Plate IM 80 C | 200 | 0.8 | 2000 | 2 | 224 | 0.25 | 91.47 |
| StoFRP Plate IM 100 C | 200 | 0.8 | 2000 | 2 | 280 | 0.25 | 114.34 |
| StoFRP Plate IM 120 C | 200 | 0.8 | 2000 | 1 | 168 | 0.35 | 97.02 |
| StoFRP Plate IM 150 C | 200 | 0.8 | 2000 | 1 | 210 | 0.35 | 121.28 |
| StoFRP Plate M 80 C | 245 | 0.8 | 2000 | 2 | 224 | 0.22 | 101.24 |
| StoFRP Plate M 100 C | 245 | 0.8 | 2000 | 1 | 140 | 0.31 | 89.49 |
| StoFRP Plate M 120 C | 245 | 0.8 | 2000 | 1 | 168 | 0.31 | 107.39 |
| StoFRP Plate M 150 C | 245 | 0.8 | 2000 | 1 | 210 | 0.31 | 134.23 |

**Table 33. Flexural strengthening for alternatives with sheets.**

| Name | $E_{fd}[GPa]$ | $\varepsilon_{fuk}[\%]$ | $f_{fd}[MPa]$ | $n\,[-]$ | $A_f[mm^2]$ | $\varepsilon_f[\%]$ | $F_f[kN]$ |
|---|---|---|---|---|---|---|---|
| StoFRPSheet S 300 C 200 | 240 | 1.7 | 4000 | 2 | 66 | 0.38 | 101.20 |
| StoFRPSheet S 300 C 300 | 240 | 1.7 | 4000 | 1 | 51 | 0.94 | 88.96 |
| StoFRP Sheet IMS S 300 C 300 | 290 | 1.9 | 5500 | 1 | 51 | 0.86 | 97.79 |
| StoFRPSheet M 300 C 200 | 395 | 1.2 | 4600 | 1 | 33 | 0.92 | 91.80 |
| StoFRPSheet M 300 C 300 | 395 | 1.2 | 4600 | 1 | 51 | 0.74 | 114.13 |

**Table 34.Flexural strengthening for alternatives with NSMR.**

| Name | $E_{fd}[GPa]$ | $\varepsilon_{fuk}[\%]$ | $f_{fd}[MPa]$ | $n\,[-]$ | $A_f[mm^2]$ | $\varepsilon_f[\%]$ | $F_f[kN]$ |
|---|---|---|---|---|---|---|---|
| StoFRP Bar M 10 C | 245 | 0.8 | 2000 | 1 | 100 | 0.67 | 196 |

**Table 35. Strains in strengthened cross-section for alternatives with laminates.**

| Name | $\varepsilon_f[\%]$ | $\omega_{bal}[-]$ | $\omega[-]$ | $\varepsilon_c\,[\%]$ | $\varepsilon_s[\%]$ |
|---|---|---|---|---|---|
| StoFRP Plate E 100 C | 0.28 | 0.43 | 0.18 | 0.10 | 0.27 |
| StoFRP Plate E 120 C | 0.28 | 0.43 | 0.18 | 0.10 | 0.27 |
| StoFRP Plate E 150 C | 0.40 | 0.37 | 0.18 | 0.14 | 0.38 |
| StoFRP Plate S 100 C | 0.27 | 0.44 | 0.18 | 0.10 | 0.26 |
| StoFRP Plate S 120 C | 0.38 | 0.37 | 0.18 | 0.14 | 0.37 |
| StoFRP Plate S 150 C | 0.38 | 0.37 | 0.19 | 0.14 | 0.37 |
| StoFRP Plate IM 80 C | 0.25 | 0.46 | 0.18 | 0.09 | 0.24 |
| StoFRP Plate IM 100 C | 0.25 | 0.46 | 0.19 | 0.09 | 0.24 |
| StoFRP Plate IM 120 C | 0.35 | 0.39 | 0.18 | 0.13 | 0.33 |
| StoFRP Plate IM 150 C | 0.35 | 0.39 | 0.19 | 0.13 | 0.33 |
| StoFRP Plate M 80 C | 0.22 | 0.48 | 0.18 | 0.08 | 0.22 |
| StoFRP Plate M 100 C | 0.31 | 0.41 | 0.18 | 0.11 | 0.30 |
| StoFRP Plate M 120 C | 0.31 | 0.41 | 0.18 | 0.11 | 0.30 |
| StoFRP Plate M 150 C | 0.31 | 0.41 | 0.19 | 0.11 | 0.30 |

**Table 36. Strains in strengthened cross-section for alternatives with sheets.**

| Name | $\varepsilon_f[\%]$ | $\omega_{bal}[-]$ | $\omega[-]$ | $\varepsilon_c\,[\%]$ | $\varepsilon_s[\%]$ |
|---|---|---|---|---|---|
| StoFRPSheet S 300 C 200 | 0.83 | 0.23 | 0.18 | 0.29 | 0.77 |
| StoFRPSheet S 300 C 300 | 0.94 | 0.21 | 0.18 | 0.33 | 0.88 |
| StoFRP Sheet IMS S 300 C 300 | 0.86 | 0.23 | 0.18 | 0.30 | 0.80 |
| StoFRPSheet M 300 C 200 | 0.92 | 0.22 | 0.20 | 0.35 | 0.85 |
| StoFRPSheet M 300 C 300 | 0.74 | 0.25 | 0.19 | 0.26 | 0.69 |

**Table 37. Strains in strengthened cross-section for alternatives with NSMR.**

| Name | $\varepsilon_f[\%]$ | $\omega_{bal}[-]$ | $\omega[-]$ | $\varepsilon_c\,[\%]$ | $\varepsilon_s[\%]$ |
|---|---|---|---|---|---|
| StoFRP Bar M 10 C | 0.67 | 0.27 | 0.20 | 0.24 | 0.63 |

**Table 38. Maximum allowed tensile force at ends of CFRP and the minimum bond length for alternatives with laminates.**

| Name | $\varepsilon_{fx}\,[\%]$ | $F_{f,e}\,[kN]$ | Right side | | Left side | | $l_e[mm]$ |
|---|---|---|---|---|---|---|---|
| | | | $F_{fa}$ | $F_{fb}$ | $F_{fa}$ | $F_{fb}$ | |
| StoFRP Plate E 100 C | 0.18 | 63.1 | -52.4 | 30.6 | -153.8 | 27.1 | 190 |
| StoFRP Plate E 120 C | 0.18 | 74.4 | -52.5 | 35.7 | -153.8 | 31.6 | 192 |
| StoFRP Plate E 150 C | 0.17 | 45.4 | -52.4 | 23.8 | -153.7 | 21.1 | 194 |
| StoFRP Plate S 100 C | 0.17 | 65.8 | -52.5 | 32.8 | -153.8 | 29.1 | 198 |
| StoFRP Plate S 120 C | 0.17 | 38.8 | -52.4 | 21.0 | -153.7 | 18.6 | 200 |
| StoFRP Plate S 150 C | 0.17 | 47.3 | -52.4 | 25.6 | -153.7 | 22.7 | 202 |
| StoFRP Plate IM 80 C | 0.16 | 59.3 | -52.5 | 32.3 | -153.8 | 28.6 | 218 |
| StoFRP Plate IM 100 C | 0.16 | 72.8 | -52.6 | 38.9 | -153.8 | 34.5 | 220 |
| StoFRP Plate IM 120 C | 0.15 | 43.0 | -52.4 | 25.2 | -153.7 | 22.3 | 222 |
| StoFRP Plate IM 150 C | 0.15 | 52.4 | -52.5 | 30.6 | -153.8 | 27.1 | 225 |
| StoFRP Plate M 80 C | 0.14 | 65.6 | -52.6 | 38.3 | -153.8 | 33.9 | 241 |
| StoFRP Plate M 100 C | 0.14 | 40.3 | -52.5 | 26.7 | -153.7 | 22.7 | 243 |
| StoFRP Plate M 120 C | 0.14 | 47.6 | -52.5 | 30.1 | -153.8 | 26.6 | 245 |
| StoFRP Plate M 150 C | 0.14 | 58.0 | -52.5 | 36.3 | -153.8 | 32.2 | 248 |

**Table 39. Maximum allowed tensile force at ends of CFRP and the minimum bond length for alternatives with sheets.**

| Name | $\varepsilon_{fx}$ [%] | $F_{f,e}$ $[kN]$ | Right side | | Left side | | $l_e$ $[mm]$ |
|---|---|---|---|---|---|---|---|
| | | | $F_{fa}$ $[kN]$ | $F_{fb}$ $[kN]$ | $F_{fa}$ $[kN]$ | $F_{fb}$ $[kN]$ | |
| StoFRPSheet S 300 C 200 | 0.49 | 59.3 | -52.3 | 11.8 | -153.7 | 10.4 | 66 |
| StoFRPSheet S 300 C 300 | 0.39 | 36.8 | -52.3 | 9.2 | -153.7 | 8.2 | 84 |
| StoFRP Sheet IMS S 300 C 300 | 0.36 | 40.5 | -52.3 | 11.0 | -153.7 | 9.8 | 92 |
| StoFRPSheet M 300 C 200 | 0.38 | 38.0 | -52.3 | 9.8 | -153.7 | 8.7 | 87 |
| StoFRPSheet M 300 C 300 | 0.30 | 47.3 | -52.4 | 14.8 | -153.7 | 13.0 | 108 |

**Table 40. Total length of CFRP and the peeling stress at the ends of the CFRP for alternatives with laminates.**

| Name | $a$ $(mm)$ | | $l_{frp}$ (mm) | $\sigma_1$ (MPa) | |
|---|---|---|---|---|---|
| | Left side | Right side | | Left side | Right side |
| StoFRP Plate E 100 C | 475 | 530 | 7450 | 2.50 | 2.50 |
| StoFRP Plate E 120 C | 485 | 535 | 7430 | 2.55 | 2.51 |
| StoFRP Plate E 150 C | 610 | 710 | 7130 | 2.53 | 2.56 |
| StoFRP Plate S 100 C | 470 | 520 | 7460 | 2.56 | 2.55 |
| StoFRP Plate S 120 C | 600 | 680 | 7180 | 2.54 | 2.52 |
| StoFRP Plate S 150 C | 600 | 690 | 7160 | 2.55 | 2.56 |
| StoFRP Plate IM 80 C | 430 | 470 | 7550 | 2.56 | 2.53 |
| StoFRP Plate IM 100 C | 430 | 470 | 7550 | 2.53 | 2.51 |
| StoFRP Plate IM 120 C | 550 | 630 | 7270 | 2.49 | 2.52 |
| StoFRP Plate IM 150 C | 560 | 640 | 7250 | 2.53 | 2.56 |
| StoFRP Plate M 80 C | 390 | 430 | 7630 | 2.51 | 2.54 |
| StoFRP Plate M 100 C | 520 | 580 | 7350 | 2.55 | 2.53 |
| StoFRP Plate M 120 C | 520 | 580 | 7350 | 2.54 | 2.51 |
| StoFRP Plate M 150 C | 475 | 590 | 7340 | 2.51 | 2.54 |

**Table 41. Total length of CFRP and the peeling stress at the ends of the CFRP for alternatives with sheets.**

| Name | $a$ $(mm)$ | | $l_{frp}$ (mm) | $\sigma_1$ (MPa) | |
|---|---|---|---|---|---|
| | Left side | Right side | | Left side | Right side |
| StoFRPSheet S 300 C 200 | 730 | 890 | 6830 | 2.51 | 2.56 |
| StoFRPSheet S 300 C 300 | 760 | 940 | 6750 | 2.51 | 2.56 |
| StoFRP Sheet IMS S 300 C 300 | 740 | 900 | 6810 | 2.52 | 2.54 |
| StoFRPSheet M 300 C 200 | 760 | 920 | 6770 | 2.55 | 2.56 |
| StoFRPSheet M 300 C 300 | 700 | 830 | 6920 | 2.53 | 2.51 |

**Table 42. Total length of alternative with NSMR.**

| Name | $\tau_f$ | $\delta_f$ | $L_{per}$ | $\lambda_n$ | $L_e$ | $P_{(\max)L}$ | $\varepsilon_{max}$ | $l_{\mathrm{frp}}$ |
|---|---|---|---|---|---|---|---|---|
| StoFRP Bar M 10 C | 11.05 | 0.755 | $38mm$ | 0.0048 | $330mn$ | 88.15 | 0.36% | 5930 |

**Table 43. Tensile force in the anchorage point of NSMR alternative.**

| Name | Right side | | Left side | |
|---|---|---|---|---|
| | $F_{fa}$ | $F_{fa}$ | $F_{fb}$ | $F_{fb}$ |
| StoFRP Bar M 10 C | -154.04 | -53.02 | 22.55 | 15.87 |

# Appendix 6 Strengthening in shear

**Table 44. Strengthening alternative in shear with fibres in 90°.**

| Name | $E_{fd}$ $[GPa]$ | $\varepsilon_{fuk}$ [%] | $t_f$ $[mm]$ | $r$ $[mm]$ | $n$ $[-]$ | $A_f$ $[mm^2]$ | $\varepsilon_f$ [%] | $V_{fs}$ $[kN]$ | $l_{ef}$ $(mm)$ | $d_{ef}$ $(mm)$ | $L_{ef}$ $(mm)$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **StoFRPSheet S 300 C 200** | 240 | 1.7 | 0.11 | 100 | 4 | 0.66 | 0.76 | 290.8 | 123.5 | 328.0 | 328.0 |
| | | | | 200 | 5 | 0.66 | | 277.9 | 138.1 | 313.4 | 313.4 |
| | | | | 300 | 6 | 0.66 | | 266.2 | 151.2 | 300.3 | 300.3 |
| | | | | 400 | 7 | 0.66 | | 255.5 | 163.4 | 288.2 | 288.2 |
| **StoFRPSheet S 300 C 300** | 240 | 1.7 | 0.17 | 100 | 3 | 0.77 | 0.76 | 327.3 | 132.9 | 318.6 | 318.6 |
| | | | | 200 | 3 | 0.61 | | 261.9 | 132.9 | 318.6 | 318.6 |
| | | | | 300 | 4 | 0.68 | | 271.2 | 153.3 | 298.0 | 298.0 |
| | | | | 400 | 5 | 0.73 | | 273.9 | 171.6 | 279.9 | 279.9 |
| **StoFRP Sheet IMS S 300 C 300** | 290 | 1.9 | 0.17 | 100 | 2 | 0.51 | 0.84 | 307.3 | 119.3 | 332.2 | 332.2 |
| | | | | 200 | 2 | 0.41 | | 245.9 | 119.3 | 332.2 | 332.2 |
| | | | | 300 | 3 | 0.51 | | 282.5 | 146.1 | 305.4 | 305.4 |
| | | | | 400 | 3 | 0.44 | | 242.1 | 146.1 | 305.4 | 305.4 |
| **StoFRPSheet M 300 C 200** | 395 | 1.2 | 0.11 | 100 | 3 | 0.50 | 0.53 | 242.8 | 137.2 | 314.3 | 314.3 |
| | | | | 200 | 4 | 0.53 | | 241.5 | 158.4 | 293.1 | 293.1 |
| | | | | 300 | 6 | 0.66 | | 265.2 | 194.0 | 451.5 | 451.5 |
| | | | | 400 | 7 | 0.66 | | 249.2 | 209.6 | 241.9 | 241.9 |
| **StoFRPSheet M 300 C 300** | 395 | 1.2 | 0.17 | 100 | 2 | 0.51 | 0.53 | 248.5 | 139.3 | 312.2 | 312.2 |
| | | | | 200 | 3 | 0.61 | | 268.3 | 170.6 | 281.0 | 281.0 |
| | | | | 300 | 4 | 0.68 | | 270.1 | 196.9 | 254.6 | 254.6 |
| | | | | 400 | 5 | 0.73 | | 263.0 | 220.2 | 231.3 | 231.3 |

# Appendix 7 Costs for strengthening alternatives in flexure

**Table 45. Cost for alternatives with laminates.**

| Name | $E_{fd}$ $[GPa]$ | $\varepsilon_{fuk}$ [%] | $t_f$ $[mm]$ | $n$ $[-]$ | $l_{frp}$ $[mm]$ | $Cost/m$ $[SEK]$ | $Total\ cost$ $[SEK]$ |
|---|---|---|---|---|---|---|---|
| StoFRP Plate E 100 C | 150 | 1.2 | 1.4 | 2 | 7450 | 927 | 13800 |
| StoFRP Plate E 120 C | 150 | 1.2 | 1.4 | 2 | 7430 | 1171 | 17400 |
| StoFRP Plate E 150 C | 150 | 1.2 | 1.4 | 1 | 7130 | 1683 | 12000 |
| StoFRP Plate S 100 C | 163 | 1.6 | 1.4 | 2 | 7460 | 1187 | 17700 |
| StoFRP Plate S 120 C | 163 | 1.6 | 1.4 | 1 | 7180 | 1398 | 10000 |
| StoFRP Plate S 150 C | 163 | 1.6 | 1.4 | 1 | 7160 | 1683 | 12000 |
| StoFRP Plate IM 80 C | 200 | 0.8 | 1.4 | 2 | 7550 | 1267 | 19100 |
| StoFRP Plate IM 100 C | 200 | 0.8 | 1.4 | 2 | 7550 | 1566 | 23700 |
| StoFRP Plate IM 120 C | 200 | 0.8 | 1.4 | 1 | 7270 | 1880 | 13700 |
| StoFRP Plate IM 150 C | 200 | 0.8 | 1.4 | 1 | 7250 | 2313 | 16800 |
| StoFRP Plate M 80 C | 245 | 0.8 | 1.4 | 2 | 7630 | 2052 | 31300 |
| StoFRP Plate M 100 C | 245 | 0.8 | 1.4 | 1 | 7350 | 2565 | 18900 |
| StoFRP Plate M 120 C | 245 | 0.8 | 1.4 | 1 | 7350 | 2954 | 21700 |
| StoFRP Plate M 150 C | 245 | 0.8 | 1.4 | 1 | 7340 | 3721 | 27300 |

**Table 46. Costs for alternatives with sheets.**

| Name | $E_{fd}$ $[GPa]$ | $\varepsilon_{fuk}$ [%] | $t_f$ $[mm]$ | $n$ $[-]$ | $l_{frp}$ $[mm]$ | $Cost/m$ $[SEK]$ | $Total\ cost$ $[SEK]$ |
|---|---|---|---|---|---|---|---|
| StoFRPSheet S 300 C 200 | 240 | 1.7 | 0.11 | 2 | 6830 | 697 | 12 400 |
| StoFRPSheet S 300 C 300 | 240 | 1.7 | 0.17 | 1 | 6750 | 882 | 7 800 |
| StoFRP Sheet IMS S 300 C 300 | 290 | 1.9 | 0.17 | 1 | 6810 | 1320 | 11 700 |
| StoFRPSheet M 300 C 200 | 395 | 1.2 | 0.11 | 1 | 6770 | 2016 | 17 800 |
| StoFRPSheet M 300 C 300 | 395 | 1.2 | 0.17 | 1 | 6920 | 2871 | 25 800 |

**Table 47. Costs for alternatives with NSMR.**

| Name | $E_{fd}$ $[GPa]$ | $\varepsilon_{fuk}$ [%] | $t_f$ $[mm]$ | $n$ $[-]$ | $l_{frp}$ $[mm]$ | $Cost/m$ $[SEK]$ | $Total\ cost$ $[SEK]$ |
|---|---|---|---|---|---|---|---|
| StoFRP Bar M 10 C | 245 | 0.8 | 10 | 1 | 5930 | 1937 | 11 500 |

# Appendix 8 Costs for strengthening alternatives in shear

**Table 48. Strengthening alternative in shear with fibres in 90°.**

| Name | $E_{fd}$ $[GPa]$ | $\varepsilon_{fuk}$ [%] | $t_f$ $[mm]$ | $r$ $[mm]$ | $n$ $[-]$ | $Sheets$ $/m$ | $Cost$ $/m$ $[SEK]$ | $Total\ cost$ $[SEK]$ |
|---|---|---|---|---|---|---|---|---|
| **StoFRPSheet S 300 C 200** | 240 | 1.7 | 0.11 | 100 | 4 | 2.50 | 334 | 6035 |
| | | | | 200 | 5 | 2.00 | 334 | 6035 |
| | | | | 300 | 6 | 1.67 | 334 | 6035 |
| | | | | 400 | 7 | 1.50 | 334 | 6337 |
| **StoFRPSheet S 300 C 300** | 240 | 1.7 | 0.17 | 100 | 3 | 2.50 | 422 | 5719 |
| | | | | 200 | 3 | 2.00 | 422 | 4575 |
| | | | | 300 | 4 | 1.67 | 422 | 5084 |
| | | | | 400 | 5 | 1.50 | 422 | 5719 |
| **StoFRP Sheet IMS S 300 C 300** | 290 | 1.9 | 0.17 | 100 | 2 | 2.50 | 632 | 5710 |
| | | | | 200 | 2 | 2.00 | 632 | 4568 |
| | | | | 300 | 3 | 1.67 | 632 | 5710 |
| | | | | 400 | 3 | 1.50 | 632 | 5139 |
| **StoFRPSheet M 300 C 200** | 395 | 1.2 | 0.11 | 100 | 3 | 2.50 | 964 | 13065 |
| | | | | 200 | 4 | 2.00 | 964 | 13936 |
| | | | | 300 | 6 | 1.67 | 964 | 17419 |
| | | | | 400 | 7 | 1.50 | 964 | 18290 |
| **StoFRPSheet M 300 C 300** | 395 | 1.2 | 0.17 | 100 | 2 | 2.50 | 1373 | 12405 |
| | | | | 200 | 3 | 2.00 | 1373 | 14886 |
| | | | | 300 | 4 | 1.67 | 1373 | 16540 |
| | | | | 400 | 5 | 1.50 | 1373 | 18608 |