Strengtheningofdamaged Bridgesusing Frp Reinforcing Elements

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Abstract: Strengthening and retrofitting existing structures to ensure a long service life has always been a matter of concern that has to be studied every period of time to innovate and produce new beneficial materials that could yield the optimum capacity for the strengthened structural element. FRP has emerged and succeeded in fulfilling that role whether by attaching the FRP outside the structure as externally bonded FRP or by placing them inside the structure itself as near surface mounted. The paper deals with the application of Fiber Reinforced Polymer (FRP) laminates as flexural and shear strengthening materials for bridges, the overall objective of this project is to strengthen two bridges using CFRP (Carbon Fiber Reinforced Polymers); the first bridge is strengthened using Externally Bonded Technique EBR and the second bridge using Near Surface Mounted NSM. The design of the strengthening elements is done based on the Euro code; the paper offers a manual for using the code to strengthen structural elements whether by grooving rods or externally applying them at the needed area. After designing the two bridges, the results showed that they only needed to be flexure strengthened but they did not need to be shear strengthened as the bridges are capable to hold the coming shear loads without being strengthened. The two bridges are strengthened according to the Euro code and the results showed that the strengthening factor of the first bridge is 1,4 and for the second bridge is approximately 2.

Index Terms: Bridges, CFRP, Euro code, Externally bonded rods,FRP, Near surface mounted,Strengthening

1 INTRODUCTION

Bridges will definitely need to be strengthened after a period of time because of the elevated increase in traffic and vehicles that exceeds the one used in the design. Also the environment and the atmosphere could affect the bridge negatively together with the ignorance of the signs that appear to show the distressed parts. FRP is attached to the tension face to the member to increase its strength, it is as if adding steel plates but the FRP succeeded in showing some advantages over steel in strengthening as it has better resistance to corrosion and its density is lower than the steel so it is easier in installation which leads to less installation time. It has higher tensile strength, higher stiffness and has good fatigue resistance. Strengthening using FRP can be found in different shapes, the available materials are rods which are similar to the steel reinforcement, strips which are unidirectional and sheets that are unidirectional or bidirectional (fibers are in the longitudinal and transverse directions).

1.1 Limitations of the FRP

With the growing use of the FRP due to their attractive properties, several problems shown below arised by time, that needed to be studied and taken into considerations.

- FRP behaves in a linear elastic manner; it does not have a yielding point which means brittle failure could easily occur so special design considerations are needed to ensure that no brittle sudden failure occurs [1].
- The coefficient of thermal expansion in the transverse direction differs from the longitudinal one so longitudinal splitting in the concrete at high temperatures could occur [1].
- FRP bars cannot be bended in the field so it has to be accurately produced with the exact shape as one cannot change it, not like the steel bars [1].
- FRP are very bad when it comes to fire resistance as they lose their strength and stiffness with elevated temperatures, all the materials have the same problem but the FRP loses its stiffness much faster than the others which makes the performance of strengthened structures using FRP much worse than without FRP [2].
- Concrete is an alkaline material so when it reacts with FRP, a problem regarding the durability of the structure arises especially if it is Glass FRP as it will be damaged due to the chemical attack of the alkaline concrete, the concentration and growth of hydration products between individual filaments [3].
- FRP is affected by the ultraviolet rays; it leads to surface oxidation because of the different chemical properties of the resin so this may lower the FRP's strength [4].
- FRP elongates by time which is defined as creep, creep occurs due to the elongation of the resin after a period of time due to the applied loads not because of the fibers itself [4].

FRP can be reinforced with a variety of materials to improve its properties; the most used ones are the glass, carbon and aramid. Every type has its own characteristics that distinguish it from the other and based on the environment and the structure; the suitable type is chosen to satisfy the conditions.

The following text is taken from [5] if not indicated otherwise.

GFRP is Glass FRP, it has a white color and when they are well protected by the resin, they resist salt water, chemicals and acid rains. They can insulate heat and electricity and they are easily processed. Their individual filaments are produced with a coating around its surface called sizing, the sizing is used to improve the bond between the resin and the glass fibers as it contains coupling agents. CFRP is carbon FRP, it has a charcoal black color; they are very durable and can sustain hot environments and perform very well in moisture environments. They have negative or very lowcoefficient of thermal expansion in its longitudinaldirection which offers perfect dimensional stability. They are also thermal and electrical conductive but one should pay attention when using carbon with another metal as a galvaniccell may develop due to the electro potential difference between the carbon and the other metal. AFRP is aramid FRP, it has a yellow color and like carbon fibers, they have negative coefficient of thermal expansion in their longitudinal direction so dimensional stability is gained. They are expensive, difficult in processing, have low melting temperature, poor compressive properties and high moisture absorption but they are very tough. When comparing their stiffness, it was found out that carbon and glass are less stiff in compression than in tension but the aramid's compressive strength is 1/10 of its tensile strength, which means that it is very weak in compression when compared to its tensile strength. Both aramid and carbon resist very high temperatures as they do not have a melting point while glass have a melting point but it can also resist elevated temperatures until it melts. Aramid is the lightest with a density of 1.4 g/m^3 and it can resist fatigue more than glass and carbon. Glass is the cheapest in high performance fibers. Aramid and glass do not conduct electricity while the carbon does but galvanic corrosion is a concern when carbon conducts electricity. The stiffness of the three types are different, carbon is the stiffer with twice the stiffness of the aramid and about five times stiffer than the glass. This

1.2 Polymer Resin

Polymer resin is the primary polymer ingredient in the non fibrous part of the FRP material that connects and binds the fibers together to form the FRP composite. It is divided into two types; thermosetting and thermoplastic. The thermosetting is more used due to its strong bonds between its components and here is a comparison between them.

TABLE 1: *COMPARISON BETWEEN THERMOSETTING AND THERMOPLASTIC POLYMER RESINS*

	Thermosetting	Thermoplastic
Cost	higher	Less expensive
Recyclable	no	yes
Strength and stiffness	higher	lower
Connection	Cross linked; molecular chains are joined to form a continuous 3 dimensional network by strongly covalent bonded atoms	Not cross linked; their molecular chains are held together by weak van der waals forces or by hydrogen bonds
Properties	Set when it solidifies or cures, cannot be reformed into different shapes	Does not set, remains plastic and when heated it softens and can be reset into a different shape upon cooling
Process	easier	difficult
Elongation	lower	Higher so more ductile

2 ATTACHING THE FRP

2.1 Near surface mounted (NSM)

A groove is cut in the structure using the saw and then the FRP strips are inserted inside it and filled with the resin. The groove should be 1.5*diameter if it is a FRP rod, if it is a rectangular strip then the groove should be 1.5* bar

stiffness has a drawback; it tends to be more brittle, it fails without deformations or warnings.
As it is shown in the stress strain curve in Fig.1 that the carbon can sustain the highest stress with the least strain deformation which makes it the stiffer as explained before. The three types showed that they do not have a yield point, their strain increases more and more until suddenly it fails.

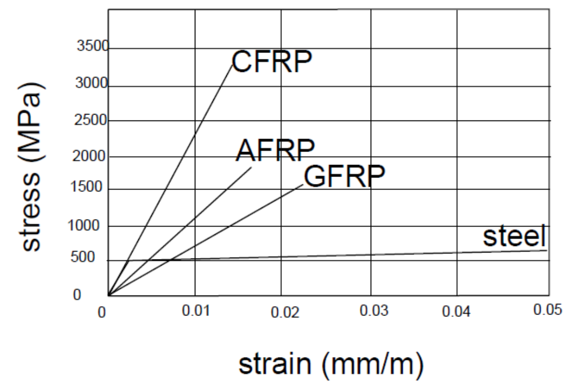


***Fig.1**. Stress-strain curve of different types of the FRP and the steel*

height and 3*bar width. It is better used to resist the moment than any other attachment process.

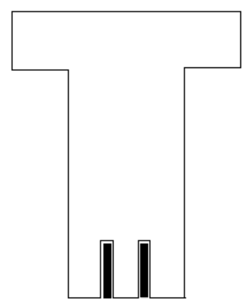


***Fig.2**. Near surface mounted FRP for a T-section beam*

2.2 Externally bonded technique (EBR)

As its name indicates, you simply attach the FRP externally on the surface of the structure not inside it. This is mainly used for flexural strengthening of the structure.

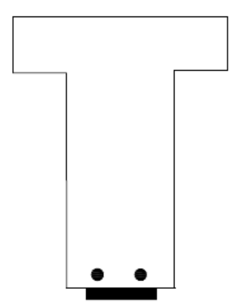


***Fig.3**. Externally bonded FRP for a T-section beam*

You can wrap it around the structure to strengthen it; used for shear strengthening and there are several types for wrapping:

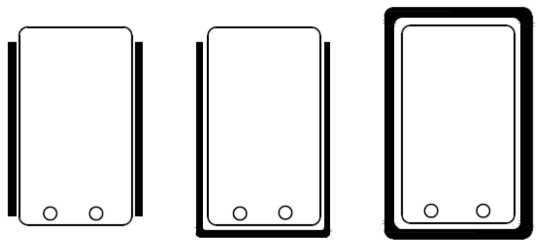


***Fig.4**. Types of wrapping*

Fig.4 shows different types of wrapping, first it shows the two sided wrapped cross section then the three sided U-wrap then the four sided wrap; they are presented respectively from the left to the right. Not all the structures can be wrapped around their four sides because some sides are not accessible for the FRP to be applied. The corners of the beams should be rounded to avoid stress concentration which might cause premature failure of the FRP wrap.

The following text is taken from [8] if not indicated otherwise.
When NSM and EBR were compared together, NSM showed more advantages over the EBR. Using the NSM, the structure is virtually unchanged as the groove is inside the structure and not recognized as the EBR. NSM does not need a lot of work preparation in the beginning; all it needs is cutting a groove using the saw which is a very easy process. NSM bars can be more easily attached to adjacent members to prevent debonding so this will be very useful for example in rigidly jointed frames where the maximum moment occurs at the end of the member. NSM bars are put inside the structure so that the bars will be protected by the concrete cover and it will be less exposed to environmental and weather impacts.NSM has larger bond surface which will definitely lead to better anchorage.

3 DESIGN USING THE EURO CODE

The following text is taken from [14] if not indicated otherwise. The load based on the ultimate limit state (permanent and temporary dimensioning of situation) results in:

$p_d = \gamma_G \times D.L. + \gamma_O \times L.L. = 1.35 \times D.L. + 1.5 \times L.L$

Theloadbased on the internationalstateofserviceabilitycalculatedunderrarecombination:

$p_{rare} = D.L. + L.L.$

The load used for the determination of the pre-strain state throughout the amplifier:

$p_{perm} = D.L.$

3.1 Externally bonded FRP

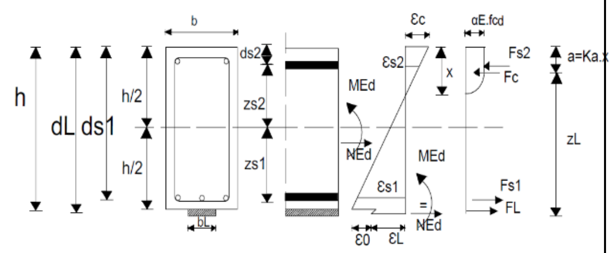


***Fig.5**. Strain and stress distribution of a strengthened cross section with EBR*

3.1.1 Prestrain calculations

Get maximum moment, maximum and the minimum shear according to the given structure. The following is the calculation of the properties of the reinforcing steel carried out with an approximately determined inner lever arm.

$Z_{s1} \approx \chi \times d_{s1}$

where χ is a variable that is based on iterations, it is first assumed by any value then by equating this equation with the coming equation$Z_{s1} = d_{s1} - a$, χcould be accurately determined.
The steel tensile force at the time of strengthening can be calculated by the moment and the internal lever arm:

$F_{s1} = \frac{\text{maximum moment}}{Zs1}$

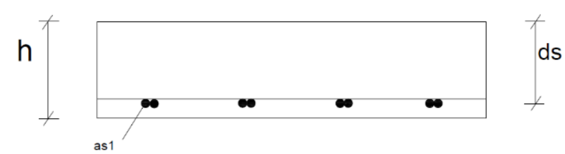


***Fig.6**. Typical cross section of a reinforced beam*

Having a known value of the young's modulus of the used steel, the strain can then be easily determined

$$\varepsilon s1 = \frac{Fs1}{as1 \,.\, Es}$$

Depending on the value of the strain of the concrete, K_a and α_R will be known

$K_a = \frac{8+\varepsilon c}{24+4.\varepsilon c}$ and $\alpha_R = \frac{-\varepsilon c}{2} - \frac{\varepsilon c^2}{12}$
for $\varepsilon_c \geq -2$mm/m

$K_a = \frac{3.\mathcal{E}c^2+4.\mathcal{E}c+2}{6.\mathcal{E}c^2+4.\mathcal{E}c}$ and $\alpha_R = 1+\frac{2}{3.\mathcal{E}c}$
for -2 mm/m $> \mathcal{E}_c \geq$ -3.5mm/m

First it is assumed that the strain of the concrete isgreater than -2 mm/m and the force in the concrete will be equal to:

F_c= b. x. f_{cm}. α_R where $x = \xi.d_{s1} = (\frac{-\mathcal{E}c}{-\mathcal{E}c+\mathcal{E}s1}).d_{s1}$

Over the balance of the internal forces obtained an equation with which the concrete without deformation can be determined, the force in the steel is equal to the force in the concrete, therefore

$F_{s1} = F_c$

By equating both equations, the strain of the concrete will be obtained and must be checked if the assumption made was justified or not and by getting εc the values of K_a and α_R will be accurately determined and the value of a will be also obtained as:

$a = K_a.\xi.d_{s1}$

By knowing the value of a, the inner lever arm could be determined with its exact value by subtracting the value if a from the ds1

$Z_{s1} = d_{s1}$-a

then the exact maximum moment will be easily obtained as the multiplication of Z_{s1} by F_{s1} ; it should be exactly equal to the moment obtained from solving the given section.

3.1.2 Flexural strengthening

A simplified proof of the composite is used but the following boundary conditions must be satisfied:

1. No prestressed concrete components.
2. The component is reinforced with ribbed rebar.
3. It is a field fortification; the longitudinal reinforcement is not graded.
4. In addition, the following condition must be met:

$f_{ctm,surface} \geq 0.26 .fcm^{\frac{2}{3}}$
Then get the maximum strain of the lamella

$\mathcal{E}_{Ld,max}$ = 0.5 mm/m + 0.1 mm/m $.\frac{l0}{h}$ - 0.04 mm/m $.\Phi_s$ +0.06 mm/m $.f_{cm}$
max.of→3mm/m. $\frac{\mathbf{l0}}{\mathbf{9700\ mm}}.(\mathbf{2}-\frac{\mathbf{l0}}{\mathbf{9700mm}})$ for $l_0 \leq 9700$ mm
→ 3mm/m for $l_0 > 9700$ mm

Where l_0 is the effective length at the area needed to be strengthened.It is checked in compliance with the international expansion that the laminate strength is not exceeded. As can be seen from the following equation:

$\mathcal{E}_{Lud} = \frac{fLuk}{EL\,.\gamma LL}$where$\gamma_{LL}$= 1.2andcompare it with$\mathcal{E}_{Ld,max}$

The typical values for the bonded CFRP aref$_{Luk}$=2200 N/mm^{2}and E_L=170 N/mm^2.

$\mathcal{E}_{Lud}$ should be greater than $\mathcal{E}_{Ld,max}$ as the $\mathcal{E}_{Ld,max}$ is the strain occurring at the section that is why this is used later in the design while the $\mathcal{E}_{Lud}$ is the ultimate strain that could occur at the section.

t_L and b_L are assumed with reasonable values then the required CFRP laminate cross section is estimated but there are some limitations:

≤ 0.2 × effective unsupported length
max $a_L \leq 5$ ×panel thickness
≤ 0.4 × cantilever length t_L

$a_L = t_L \times b_L \times$ 1 m
$F_{Ld} = a_L \times f_{Ld} = a_L \times \mathcal{E}_{Ld,max} \times E_L$
$\mathcal{E}_{L,0} = \mathcal{E}_{s1,0} + \frac{dL-ds1}{ds1}.(\mathcal{E}_{s1,0} - \mathcal{E}_{c,0})$
Therefore the total strain= $\mathcal{E}_{L,0} + \mathcal{E}_{Ld,max}$

The force in the steel at the yielding point will be equal:

$F_{s1d} = \frac{as1\,.\,fyk}{\gamma s}$ where γ_s = 1.15

The force in the lamella will simply be = $a_L.\ \mathcal{E}_L.E_L$

The force in the concrete will be calculated one more time using nearly the same equation but this time with the presence of the FRP and also by assuming that the strain of the concrete is greater than -2 mm/m.

$F_c = b.x.f_{cd}.\alpha_R = b.\xi.d_L.\alpha_{cc}\ .\frac{fck}{\gamma c}.(\frac{-\mathcal{E}c^2}{12} - \frac{\mathcal{E}c}{2})$ where $\xi = \frac{-\mathcal{E}c}{-\mathcal{E}c+\mathcal{E}L,0+\mathcal{E}Ld,max}$

By balancing the internal forces,

$F_{s1d} + F_{Ld} = F_{cd}$

So ε_c will be accurately determined and by repeating the same steps, the value of a will also be determined and the inner lever arm.

$Z_{s1} = d_{s1}$-a
Z_L= h-a

The torque-bearing capacity of the reinforcing steel concrete cross section is thus obtained as

$m_{Rd} = z_{s1}.F_{sld} + z_L.F_{Ld}$

m_{Rd} should be compared with the actual maximum ultimate moment acting on the structure and the strengthening factor can be determined by dividing the m_{Rd} with the actual moment.

3.1.3 Anchorage

The cracking moment of the cross section must be determined by using the surface tensile strength.

$M_{cr} = \kappa_{fl} .f_{ctm,surf}.W_{c,0}$ where $\kappa_{fl}=1.6 \frac{h}{1000} \geq 1$ and $W_{c,0} = \frac{b.h^2}{6}$

The first bending crack must be determined.

The end for the anchorage proof associate composite length and the associated expansion of the cross lamella is determined.

$l_{bL,lim}= 0.86 . l_{bL,max}$

$l_{bL,max}= \frac{2}{KLb} . \sqrt{\frac{EL.tL.sLok}{\tau L1k}}$

where $\tau_{L1k} = 0.366\sqrt{\alpha cc.fcm.\alpha ct.fctm,surf}$ and $s_{L0k} = 0.201$ mm

α_{cc} is a coefficient taking into its account the long term effects on the compressive strength and of the unfavorable effects resulting from the way load is applied, it is taken as 0.85 for flexure and axial loads and 1.0 for other phenomena.

α_{ct} is a coefficient to consider long term effects for the tensile concrete.

$\varepsilon^a_{LRk,lim}= 0.985 . \frac{fbLk,max}{ELm}$

$f_{bLk,max}= \sqrt{\frac{EL.\ sL0k.\tau L1k}{tL}}$

Compare $l_{bL,lim}$ with l_{bL}, then the extent of the lamella and the corresponding slippage can be calculated by the following equations:

$\varepsilon^a_{LRk}(l_{bL}) = \varepsilon^a_{LRk,lim}$ for $l_{bL,lim} \leq l_{bL}$

$\varepsilon^a_{LRk}(l_{bL})=\sin(\frac{\pi}{2}.\frac{lbL}{lbL,lim}).\ \varepsilon^a_{LRk,lim}$ for $0<l_{bL}<l_{bL,lim}$

$s^a_{Lr}(L_{bL}) = 0.213$ mm+ $(l_{bL} - l_{bL,lim}) . \varepsilon^a_{LRk,lim}$ for $l_{bL,lim} \leq l_{bL}$

$s^a_{Lr}(L_{bL}) = 0.213$ mm.$(1- \cos(\frac{\pi}{2}.\frac{lbL}{lbL,lim}))$ for $0<l_{bL}<l_{bL,lim}$

The strain of the concrete steel reinforcement must be determined but there are many variables needed to be calculated first to get the strain.

TABLE 2: *VARIABLES DEFINED ACCORDING TO RIBBED OR SMOOTH STEEL REINFORCEMENT*

Variables	Ribbed steel reinforcement	smooth steel reinforcement
K_{b1K}	2.545	1.292
K_{b2}	1	1.3
K_{b3}	0.8	1
K_{b4}	0.2	0.3
α_N	0.25	0

$K_{bsk} = K_{b1K} .\sqrt{\frac{fcm^{Kb2}}{Es.\ \varphi^{Kb3}.(EL.tL)^{Kb4}}}$

$X=[-(\alpha_L.\rho_L+\alpha_s.\rho_{s1})+\sqrt{(\alpha L.\rho L + \alpha s.\rho s1)^2 + 2.(\alpha L.\rho L.\frac{dL}{h} + \alpha s.\rho s1.\frac{ds1}{h})}].h$

$\rho_{s1} =\frac{as1}{b.h}$

$\rho_L= \frac{aL}{b.h}$

$\alpha_s= \frac{Es}{Ec}$

$\alpha_L= \frac{EL}{Ec}$

With these variables, the extent of the concrete steel reinforcement can now be determined:

$\varepsilon^a_{sRk}(L_{bL}) = K_{VB} .K_{bsk} . (s^a_{Lr}(L_{bL}))^{(\alpha N+1)/2} .(\frac{d^a-x^a}{dL^a-x^a})^{(\alpha N+1)/2}$

Where K_{VB} =1 for good composite conditions and K_{VB} =0.7 for massive composite conditions.
To calculate the recordable moment, the lever arms are still required.

$Z_L^a= h-k_a.x$

$Z_s^a= d-k_a.x$

Thus, the recordable moment that gives to the support equipment the next bending crack:
$m_{Rd}(l_{bL}) = \varepsilon^a_{LRk}(l_{bL}).\ E_{Lm}.\ A_L.\ z^a_L.\frac{1}{\gamma BA}+ \varepsilon^a_{sRk}(l_{bL}).\ E_s.\ A_s.\ z^a_s.\frac{1}{\gamma s}$

The applied torque is obtained on the position of the support equipment next bending crack which may be adopted with full plates for this proof to h /2 :

$m_{Ed}(x) = \frac{p.l}{2} .(x_{cr} + \frac{h}{2}) - \frac{p.\ (xcr + \frac{h}{2})^2}{2}$

Compare $m_{Ed(x)}$ with $m_{Rd(lbL)}$,If the applied torque $m_{Ed(x)}$ is smaller than the recordable moment $m_{Rd(lbL)}$, then the end anchorage proof is met.

3.1.4 Shear strengthening

The value of the shear force used in design ($V_{Ed,red}$) is equal to:

$V_{Ed,red}= V_{Ed} - p_{Ed} . (\frac{t}{2} + d)$

where V_{Ed} is the maximum shear acting on the structure that is used in design and p_{Ed} is the distributed load on the cross section.
In the design equations, we use $V_{Ed,red}$ instead of using directly the maximum shear as the maximum shear V_{Ed} occurs at the support so it will carried by the support itself.

$V_{Ed,red} \leq V_{Rd,c}+ V_s+ V_f$

The shear force resistance of a component without shear force reinforcement is obtained from the following equation:

$V_{Rd,c} = [C_{Rd,c} .k .(100.\rho_1 .f_{ck})^{\frac{1}{3}}+ 0.12 .\sigma_{cp}].d$

$k = 1+(\frac{200}{d})^{0.5}$

$\sigma_{cp}=\frac{NEd}{Ac}$

$C_{Rd,c} = \frac{0.15}{\gamma c}$

$100.\rho_1 = \frac{as1}{d}$

The minimum lateral force resistance component without lateral force reinforcement results in:

$V_{Rd,c\ min} = [\frac{0.0525}{\gamma c} . \sqrt{k^3.fck} + 0.12 .\sigma_{cp}].d$

Compare $V_{Ed,red}$ with the $V_{Rd,c}$ or $V_{Rd,c\ min}$ whichever the bigger one and if $V_{Ed,red}$ is smaller then the structure is safe and does not need to be strengthened due to the acting shear loads as this has shown us that only the concrete can sustain the coming loads.

If the ratio $\frac{VEd,red}{maximum\ of\ VRd,cminor\ VRd,c}$ is less than 1, then also there is no need for the structure to be strengthened due to the applied shear loads.

If $V_{Ed,red}$ > maximum of $V_{Rd,c}$ and $V_{Rd,c\ min}$, then the structure should be strengthened.

3.1.5 Limitation of stresses

The strain in the steel should be less than or equal $\frac{fyk}{Es} = \frac{500}{200000}$ =2,5 mm/m and the strain in the lamella should be less than or equal to 2, so using the following four equations and by iterations, one can get the value of the stresses and check whether it is in the acceptable range or not.

$F_{s1d}+F_{Ld}=F_{cd}$
$F_{LEd}=a_L*E_L*\varepsilon_L$
$\varepsilon_{S,SLS}=\frac{-\varepsilon c+\varepsilon L,sLs+\varepsilon Lo}{dL*ds1} + \varepsilon_c$
$F_{s1d}=a_{s1}*E_s*\varepsilon_{s1}$

3.2 Near Surface Mounted

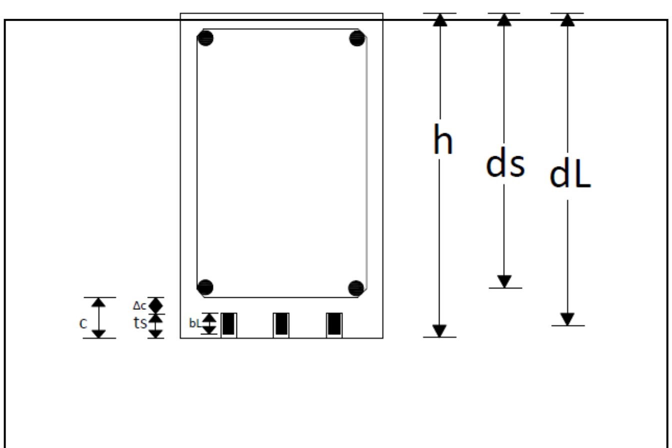


***Fig.7**. Typical cross section of NSM strengthening for a rectangular beam*

3.2.1 Prestrain calculations

It has the same rules of the EBR according to the load acting on the section, the strains and the forces.

3.2.2 Flexural strengthening

Mostly the same as the EBR but there are some different and new variables that needed to be calculated for the near surface mounted FRP.

$dL = h - (ts - \frac{bL}{2})$ where dL is the height of the CFRP laminate

It is considered on the safe side that the FRP always integrates fully into the maximum depth of the slot ts. The depth of the slot in the concrete shall be designed so that they lamella, taking into account the equalization of uneven ground, the maximum permissible slot depth ts .

$ts \leq c - \Delta cdev$

$\Delta cdev$ describes the existing concrete cover of the reinforcement, this is given by:

$\Delta cdev = \Delta cGerät + \Delta cSchnitt + \Delta cBauteil$

ΔcGerät is the device-specific error limit and should be at least 1 mm, the allowance for the depth of cut $\Delta c_{Schnitt}$ takes into account the execution tolerances in the cut of the slot and should be at least 2 mm. $\Delta c_{Bauteil}$ is for the different distribution of the concrete cover over the component into account. ΔcBauteil = 0 mm may be set for a slab otherwise ΔcBauteil is at least 2 mm are accepted. By suitable measures, such as the random checking of the concrete cover by spot expose the reinforcement, the accuracy of the concrete cover measurement is increased, ΔcBauteil can be omitted.

In the proof of the bending bearing capacity, the attachable fracture strain is mitigated by the factor K_ε = 0.8.

$\varepsilon LRd,max \leq K\varepsilon . \varepsilon Lud$

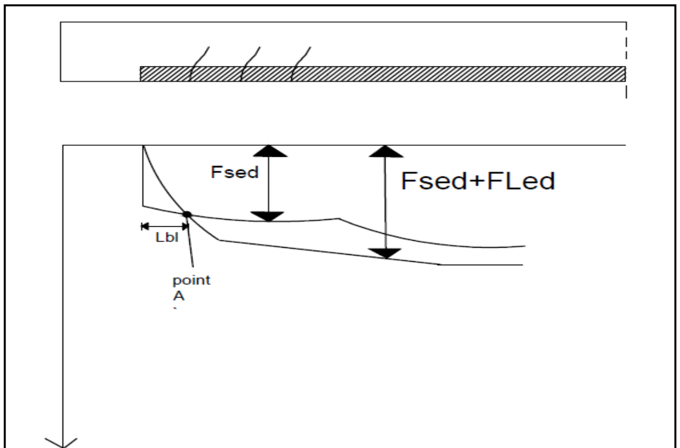


***Fig.8**. Graph showing the force of the steel, the FRP and the anchorage length*

3.2.3 Yielding point

The achieved yield strength of the existing concrete steel reinforcement must be determined along with the bending moment at which the rebars begin to flow.

$Fs1d=\frac{As1.fyk}{\gamma s}$

where Fs1d is the force in the steel at the yielding point

$\varepsilon s1=\frac{fyd}{Es}$

where ε_{s1} is the strain in the steel at the yielding point

Assume εc> -2 mm/m and by balancing the internal forces(Fc=Fs1d) and using the same steps used before to get the accurate value of εc and eventually the moment at the yielding point will be calculated.

$MRdy,0=zs1.Fs1d$

And according to the given structure x(MRdy, 0) can be determined which is the distance where the steel starts to yield.

3.2.4 Anchorage

The cracking moment is calculated as in the EBR and it should be compared with the working not the ultimate moment at x (MRdy, 0).

$M_{LFl,perm} \geq M_{cr}$

So the proof of the anchorage of the bonded CFRP in slots lamella on the spot (point A) in Fig.8 can be performed, from which the CFRP lamella is required for bearing capacity. The anchorage length of the lamella lbL is the distance between this point A and the blade end of this proof. For the proof of anchorage results in the design value of bond bearing capacity per CFRP lamella in dependence of the anchoring length l_{bL} and spacing of which may be recognized with more than 150 mm lamellar long axis on the free edge component ar.

$LbL = aL- al - x(MRdy, 0)$

$al=1*dL$

Where aL is the distance where the FRP should stop and it is assumed.

$FbLRd = bL.\text{Ţ}bLd .\sqrt[4]{ar} . lbL .(0.4 - 0.0015. lbL) . 0.95$ for $lbL \leq 115$ mm

$FbLRd = bL.\text{Ţ}bLd .\sqrt[4]{ar} . (26.2 + 0.065. \tanh(\frac{ar}{70}).(lbL -115)). 0.95$ for $lbL> 115$ mm

Where $\text{Ţ}bLd =\frac{1}{\gamma BE} . \min \{ \text{Ţ}bGk . \alpha bG \text{ or } \text{Ţ}bck . \alpha bc \}$

Depending on adhesive and environmental conditions of the application is the fatigue strength action coefficient for the composite fail in the concrete αbG is between 0.5 and 0.85.

The fatigue strength behavior of concrete has been studied extensively and for this reason, αbc found to be between 0.85 and 1.

$\text{Ţ}bGk=k_{sys}. \sqrt{(2.fGtk - 2\sqrt{(fGtk^2 + fGck . fGtk)} + fGck). fGtk}$

k_{sys} is the product-specific system action coefficient, it depends on system used. Its value is between 0.6 and 1. According to the general building system approvals, fGtk is between 21 and 28 N/mm^2 and fGck is between 75 and 85 N/mm^2.

$\text{Ţ}bck =kbck .\sqrt{fcm}$

The product-specific system action coefficient for the composite failure of the general type approval system concrete can be predicted after many trials at the technical university of Munich, a characteristic value of k_{bck} = 4.5 almost made.

The resistance of the total lamella can be calculated by multiplying the number of the lamella rebars by the anchored lamella tensile force/lamella.

$FbLRd, ges =nL.FbLRd$

FbLRd, ges should be compared with actual force of the lamella and it should be bigger as the lamella should resist more than what is acting on it.

3.2.5 Shear strengthening

Same calculations as the EBR but there are some small differences.

The value of the shear force used in design (VEd, red) acts at a distance a_i+d_{s1} :

$VEd, red = VEd - pEd . (ai+ ds)$

$ai=\min. \text{ of } (\frac{h}{2} \text{ or } \frac{t}{2})$

Where VEd is the maximum shear acting on the structure that is used in design and pEd is the distributed load on the cross section.

The lateral force resistance component without lateral force reinforcement is obtained from the following equation:

$VRd, c = [CRd, c .k .(100.\rho1 .fck)^{\frac{1}{3}} + 0.12 .\sigma_{cp}].d.b_w$

The variables are the same as the EBR except the $\rho_1= \frac{As1}{d.bw}$.

The minimum lateral force resistance component without lateral force reinforcement results in:

$V_{Rd,c} = [\frac{0.0525}{\gamma c} . \sqrt{k^3. fck} + 0.12 . \sigma_{cp}].d.b_w$

3.2.6 Limitation of stresses

Same as the externally bonded FRP.

By these rules and equations stated above, the properties of the FRP attached whether by NSM or EBR could be determined.

4 STRENGTHENING BRIDGES USING THE EURO CODE

4.1 Bridge 1

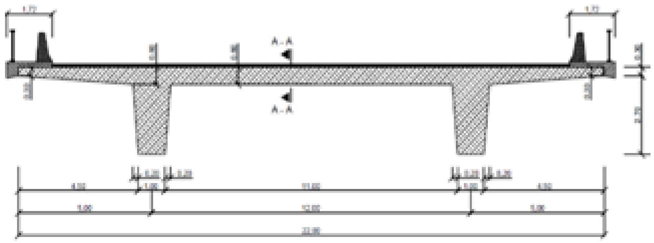


Fig.9*.Transverse section of bridge 1*

This bridge was designed using C50/60 and steel B500A, it was supposed to be constructed according to the reinforcement shown in Fig.10 but the labors done something wrong and switched the longitudinal bottom reinforcement with the transverse reinforcement in the middle section as shown in Fig. 11 so the bridge must be strengthened.

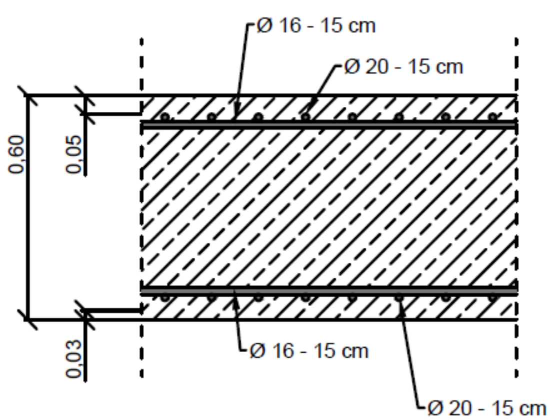


Fig.11.*Realized cross section A-A*

Strengthening the bridge using CFRP is to be designed including only the dead loads and the live loads without the traffic loads coming on the bridge as this mistake was found out before the bridge was available for the public, so before the influence of any traffic loads. Externally bonded CFRP strips was chosen over the Near Surface mounted as at section A-A as there is no enough concrete cover at the bottom.

4.1.1Pre-strain Calculations

d_{s1} is the distance from the top of the section till the reinforcement

d_{s1} =0,6-0,03-(0,16/2)=0,562 m = 562 mm
Z_{s1} ≈0,940* ds1=0,940*562=528,460 mm

Maximum moment will be calculated by adding the given moments due to the dead load superstructure, road surface, caps, the restraining systems and thelive loads (without the traffic loads as explained earlier).

Maximum moment =97,260 KN.m

$F_{s1} = \frac{\text{maximummoment}}{\text{Zs1}} = \frac{97,260}{528,460*10\text{^}-3}$ =184,044 KN
a_{s1}= 64π cm^2every 15 cm = $\frac{1280}{3}$ π mm^2/m
$\varepsilon_{s1} = \frac{\text{Fs1}}{\text{as1 .Es}} = \frac{184,044}{\frac{1280}{3}\pi*200}$ = 0,687 mm/m

Assume ε_c>-2 mm/m and by balancing of the internal forces, $F_{s1} = F_c$

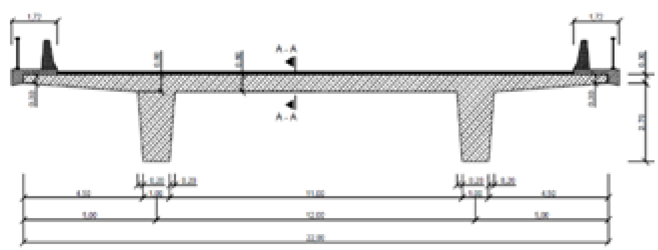
F_c= b. x. f_{ck}. α_R = b. f_{ck}. $(\frac{-\varepsilon c}{-\varepsilon c+\varepsilon s1})$.ds1 .$(\frac{-\varepsilon c}{2} - \frac{\varepsilon c^2}{12})$= 1000. 50. $(\frac{-\varepsilon c}{-\varepsilon c+0,687})$. 562. .$(\frac{-\varepsilon c}{2} - \frac{\varepsilon c^2}{12})$= 184,044 *103

Therefore ε_c= -0,103 mm/m > -2 mm/m , then the assumption was correct.

$K_a = \frac{8+\varepsilon c}{24+4.\varepsilon c} = \frac{8-0,103}{24-4*0,103}$ = 0,335

$\xi = (\frac{-\varepsilon c}{-\varepsilon c+\varepsilon s1}) = (\frac{0,103}{0,103\ +0,687})$= 0,130

a = K_a.ξ.d_{s1} = 0,335*0,130*562 =24,455 mm
Z_{s1}= d_{s1}-a=562 - 24,455 =537,545 mm
Exact moment = Zs1. Fs1= 537,545*184,044*10^-3 = 98,932 KN/m



4.1.2 Flexural Strengthening

$\varepsilon_{Ld,max}$ = 0.5 mm/m + 0.1 mm/m .$\frac{l0}{h}$ - 0.04 mm/m .Φs +0.06 mm/m .fcm
= 0.5 mm/m + 0.1 mm/m .$\frac{12000*0,7}{600}$ - 0.04 mm/m .16+0.06 mm/m .58
=4,740 mm/m > 3 mm/m

$\varepsilon_{Lud} = \frac{\text{fLuk}}{\text{EL .}\gamma\text{LL}} = \frac{2200}{170*1,2}$ = 10,784 mm/m >$\varepsilon_{Ld,max}$ → safe

Assume t_L =1,4 mm and b_L=160 mm

$a_L = t_L \cdot b_L$ = 1,4 . 160=224 mm2

$F_{Ld} = a_L \times f_{Ld} = a_L \times \varepsilon_{Ld,max} \times E_L$= 224 × 4,74 ×170/1000 = 180,499 KN/m

$\varepsilon_{L,0} = \varepsilon_{s1,0} + \frac{\text{dL}-\text{ds1}}{\text{ds1}}$. $(\varepsilon_{s1,0} - \varepsilon_{c,0})$ = 0,687 + $\frac{600-562}{562}$. (0,687 +0,103)= 0,740 mm/m

Total strain= $\varepsilon_{L,0} + \varepsilon_{Ld,max}$ = 0,740 +4,740 =5,480 mm/m

$F_{s1d} = \frac{\text{as1 . fyk}}{\gamma\text{s}} = \frac{\frac{1280}{3}\pi.500}{1.15}$ /1000 = 582,788 KN/m

Assume ε_c>-2 mm/m and by balancing the internal forces, $F_{s1d}+F_{Ld}=F_{cd}$

F_{cd}=582,788+180,499 =763,287KN/m

$$F_{cd}= b.x.f_{cd}.\alpha_R = b.\frac{-\varepsilon c}{-\varepsilon c+\varepsilon L,0+\varepsilon Ld,max}.dL.\alpha cc \ .\frac{fck}{\gamma c}.(\frac{-\varepsilon c^2}{12}-\frac{\varepsilon c}{2})$$
$$=1000.\frac{-\varepsilon c}{-\varepsilon c+5,480}.600 \ .\ 0,85 \ .\ \frac{50}{1,5}.(\frac{-\varepsilon c^2}{12}-\frac{\varepsilon c}{2}) =763,287*10\ 3$$

Therefore ε_c= -0,808 mm/m > -2 mm/m, then the assumption was correct.

$$\xi=\frac{-\varepsilon c}{-\varepsilon c+\varepsilon L,0+\varepsilon Ld,max}=\frac{0,808}{0,808\ +5,480}=0,128$$
$$k_a=\frac{8+\varepsilon c}{24+4.\varepsilon c}=\frac{8-0,808}{24-4*0,808}=0,346$$

a = Ka.ξ.d_{s1}= 0,346*0,128*562=26,693 mm

$Z_{s1}=d_{s1}$-a=562-26,693=535,307 mm

Z_L= h-a=600-26,693=573,307 mm

$m_{Rd}=z_{s1}.F_{sld}+z_L.F_{Ld}$=535,307*582,788*10-3+573,307*180,499*10-3=415,452KN/m < 496.301 KN/m (maximum value for the ultimate moment)→ not safe → the FRP should be increased.

So use FRP strips every 0,5 m not every 1 m by repeating exactly the same steps but using
$a_L=t_L*b_L$/ 0.5 m=1,4*160*$\frac{1}{0,5}$= 448 mm2/m

therefore m_{Rd} will be equal 515,892 which is greater than 496.301, so the design is acceptable and safe.

$$\omega=\frac{as1\ .\sigma sd-NEd}{b.d.fcd} \text{ where } \sigma_{sd}=\frac{fyk}{\gamma s}=\frac{500}{1,15}=434,782$$

$$\omega=\frac{\frac{1280}{3}\pi*10^{-6}.434,782-0}{1.0,562.28,3}=0,037 \rightarrow \mu_{Eds}=0,04$$
$M_{Rd,old}=\mu_{Eds}*b*d_2*f_{cd}$=0,04*1*0,5622*28,3*1000=357,535 KN.m

Therefore the strengthening factor that this bridge will be strengthened with $=\frac{mRd}{MRd,old}=\frac{515,892}{357,535}$ =1,443

4.1.3 Anchorage

$M_{cr}=\kappa_{fl}.f_{ctm,surf}.W_{c,0}$

κ_{fl} =1.6 $-\frac{h}{1000}$=1.6 $\frac{600}{1000}$ =1 and $W_{c,0}=\frac{b.h^2}{6}=\frac{1000.600^2}{6}$= 60000000
therefore M_{cr} =1*4,1*60000000*10-6=246 KN

x_{cr} can be obtained directly from the given ultimate moments using linear interpolation
with M_{cr}=246 KN → x_{cr}= 8130 mm

Distance from the beginning of the beam till the beginning of the FRP should be at least
$=\frac{t}{2}+a_{L,t}$+5 m (cantilever length) $=\frac{1400}{2}$ +50+5000=5750 mm
so I will assume this distance= 7000 mm
$l_{bL}=x_{cr}$ - 7000= 8130- 7000= 1130mm

$l_{bL,lim}$= 0.86 . $l_{bL,max}$

$$l_{bL,max}=\frac{2}{KLb}.\sqrt{\frac{EL.tL.sLok}{\tau L1k}}$$

$\tau_{L1k}=0.366\sqrt{\alpha cc.fcm.\alpha ct.fctm,surf}= 0.366\sqrt{0,85.58.0,85.4,1}$ = 4,797 N/mm2

Therefore $l_{bL,max}=\frac{2}{1,128}.\sqrt{\frac{170000.1,4\ .0,201}{4,797}}$ =177,053 → $l_{bL,lim}$= 0,86.177,053 =152,266 mm

$$\varepsilon_{LRk,lim}=0.985\ .\frac{fbLk,max}{ELm}$$

$$f_{bLk,max}=\sqrt{\frac{EL.\ sL0k\ .\tau L1k}{tL}}=\sqrt{\frac{170000\ .\ 0,201\ .4,797}{1,4}}=342,185 \text{ N/mm2}$$

therefore $\varepsilon_{aLRk,lim}=0,985\ .\frac{342,185}{170}$ = 1,983 mm/m

since $l_{bL,lim}<l_{bL}$

therefore $s^a_{Lr(LbL)}$ = 0.213 mm+ (l_{bL} - $l_{bL,lim}$) . $\varepsilon_{LRk,lim}$ = 0.213 mm+ (1130- 152,266) . $\frac{1,983}{1000}$=2,152 mm

$$\rho_{s1}=\frac{as1}{b.h}=\frac{\frac{1280}{3}\pi}{1000.600}=0,002$$

$$\rho_L=\frac{aL}{b.h}=\frac{448}{1000.600}=0,00037$$

$$\alpha_s=\frac{Es}{Ec}=\frac{200000}{37000}=5,405$$

$$\alpha_L=\frac{EL}{Ec}=\frac{170000}{37000}=4,595$$

$$K_{bsk}=K_{b1K}.\sqrt{\frac{fcm^{Kb2}}{Es\ .\ \varphi^{Kb3}.(EL\ .tL)^{Kb4}}}$$
$$=2,545.\sqrt{\frac{58^1}{20000\ .\ 16^{0,8}.(170000\ .1,4)^{0,2}}}=0,003$$
X=[-(αL.ρL+αs.ρs1)+
$\sqrt{(\alpha L\ .\rho L\ +\ \alpha s\ .\rho s1)^2+\ 2.(\alpha L\ .\rho L.\frac{dL}{h}+\alpha s\ .\rho s1.\frac{ds1}{h})}$].h
=[-(4,595.0,00037+5,405.0,002)+
$\sqrt{(4,595.0,00037\ +\ 5,405\ .0,002)^2+\ 2.(4,595.0,00037.\frac{600}{600}+5,405\ .}$
].600=94,140 mm

$\varepsilon^a sRk\ (LbL) = KVB\ .\ Kbsk\ .\ (saLr(LbL))(\alpha N+1)/2\ .\ (\frac{d^a-x^a}{dL^a-x^a})^{\frac{\alpha N+1}{2}}$
=1.0,003.(2,152)(0,25+1)/2$(\frac{562-94,140}{600-94,140})^{(0,25+1)/2}$=0,0051 mm/m> 0,0025 mm/m → use εasRk (LbL) =0,0025

Z^L_a = h-ka.x≈600-0,4 . 94,140=562,344 mm

Z^{s}_{a}= d-ka.x≈562-0,4 . 94,140=524,344 mm
$mRd(lbL) = \mathcal{E}^{a}LRk(lbL).\ ELm.\ AL.\ z^{a}L.\frac{1}{\gamma BA} + \mathcal{E}^{a}sRk(lbL).\ Es.\ As.\ z^{a}s.\frac{1}{\gamma s}$ = 0,0025.170. 448. 562,344.$\frac{1}{1,5}$ +0,0025. 200. $\frac{1280}{3}$ π. 524,344.$\frac{1}{1,15}$=362,191 KNm/m > mEd(x) =257,203 KNm/m → safe

4.1.4 Shear Strengthening

Maximum shear acting on the structure acts at a distance $\frac{t}{2}$ +d so from the given values of the shear acting on the structure, the shear at x=5+0,7+0,6=6,3 m → Max. shear =265,201 KN

$VRd,c = [\ CRd,c\ .k\ .(100.\rho_1\ .fck)^{\frac{1}{3}} + 0.12\ .\sigma_{cp}\ ].d$

$k = 1+(\frac{200}{d})^{0.5} = 1+(\frac{200}{562})^{0.5}= 1,597$

$\sigma cp = \frac{NEd}{Ac} = 0$

$CRd,c = \frac{0.15}{\gamma c} = \frac{0.15}{1,5} = 0,1$

$100.\rho 1 = \frac{as1}{d} = \frac{\frac{1280}{3}\pi*10^2}{56,2}=0,239$

Therefore VRd,c = [0,1. 1,597 .(0,239.50)$^{\frac{1}{3}}$].562=204,995 KN/m

$VRd,c\ min = [\ \frac{0.0525}{1,5}\ .\ \sqrt{1,597^3.50}\ ].562$ =280,584 KN/m> VRd,c→ $VRd,c\ min$ is used

$\frac{Max.shear}{VRd,c\ min} = \frac{265,201}{280,584}$ = 0,945 < 1 → acceptable, no need to be strengthened due to the shear loads.

4.1.5 Limitation of stresses

$\varepsilon Lo = \varepsilon s1 + \frac{dL-ds1}{ds1}\ .\ (\mathcal{E}s1,0 - \mathcal{E}c,0) = 0,687 + \frac{600-562}{562}\ .\ (0,687 +0,103)$= 0,740 mm/m

First assume $\varepsilon L,SLS$ and ε_c

Fs1d+FLd=Fcd→ 1

FLEd=aL *EL*εL→ 2

$\varepsilon S,SLS = \frac{-\varepsilon c+\varepsilon L,sLs+\varepsilon Lo}{dL*ds1} + \varepsilon c$ → 3

Fs1d=as1*Es*εs1→4

Using equation 1,2,3 and 4 and by iterations, εc and $\varepsilon L,SLS$ could be determined
εc= -0,352 mm/m

$\varepsilon L,SLS$= 0,94 mm/m < 2 mm/m → safe

therefore $\varepsilon S,SLS = \frac{0,352+0,94+0,740}{600*562}$ -0,352= 1,551 mm/m < 2,5 mm/m → safe

The bridge needed only flexural strengthening as it was strong enough in shear; the concrete alone can sustain the shear loads acting on the section so it did not need to be shear-strengthened.

After doing the calculations to get the needed dimensions of the CFRP, the results showed that the CFRP required to strengthen that bridge will be (1,4*160) mm every half a meter and will be placed at the bottom of the section as shown in the following figures.

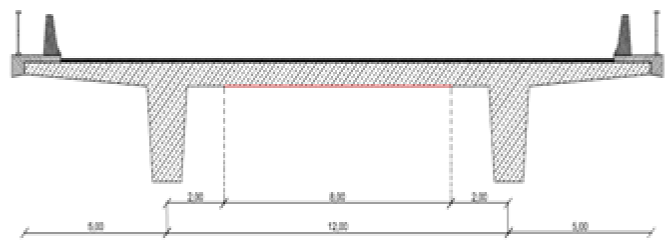


***Fig.12**. Transverse section of bridge 1 after the FRP has been attached*

The following figures will show how the CFRP will look like when it is attached to the bridge

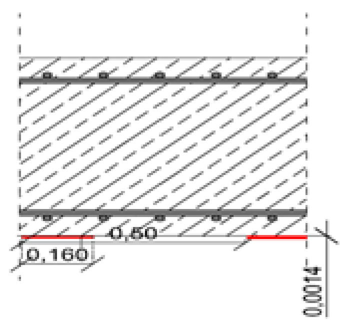


***Fig.13**. Retrofitted cross section A-A*

Fig.12 shows the EBR for bridge one in the longitudinal section, the length of the shown FRP is 8 m.Fig.13 shows it but from the cross sectional view, the spacing between the EBR is 0,5 m and the width of the shown FRP is 0,16 m and the thickness is 0,0014 m.

4.2 Bridge 2

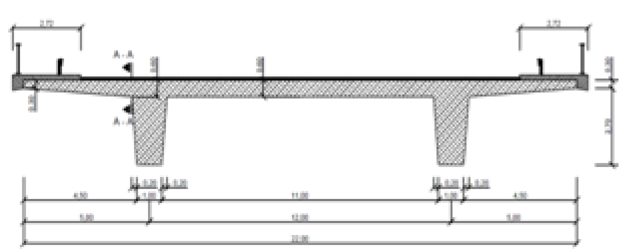


***Fig.14.** Transverse section of bridge 2*

After constructing the bridge shown in Fig.14, one lane needed to be added on both sides at the cantilevers so the restraining systems have to be changed from steel to concrete and the caps' lengths decreased as shown in Fig.15 which means more load is added to the outer cantilevers.

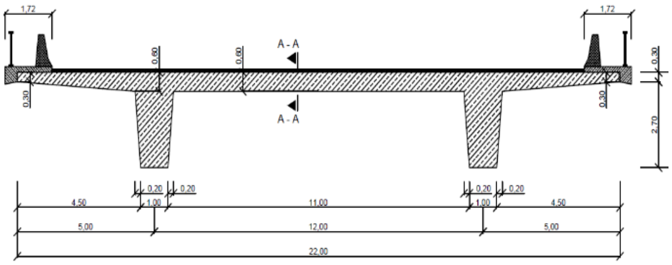


Fig.15. ***Transverse section of bridge 2 after the required modification***

This bridge was also designed using C50/60 and steel B500A, traffic loads will be considered in the strengthening process as changes occur in the bridge after it has been used. This bridge is strengthened using Near Surface mounted as there is enough concrete cover to place in it the FRP, the upper concrete cover is 0.05 m which is appropriate for the FRP.

4.2.1 Pre-strain calculations

d_{s1} =0,6-0,05-0,01=0,540 m = 540 mm
Z_{s1} ≈0,952* ds1≈0,952*0,540≈0,514 m≈ 513,970 mm

Maximum moment = 178,770 KN.m

$F_{s1} = \frac{maximummoment}{Zs1} = \frac{178,770}{0,514}$ =347,822 KN

a_{s1} = 500 π mm2/m

$\varepsilon_{s1} = \frac{Fs1}{as1 .Es} = \frac{347,822}{500\,\pi*200}$ = 1,107 mm/m

Assume ε_c>-2 mm/m
By balancing of the internal forces, $F_{s1} = F_c$

F_c= b. x. f_{ck}. α_R = b. f_{ck}. $(\frac{-\varepsilon c}{-\varepsilon c+\varepsilon s1})$.ds1 .$(\frac{-\varepsilon c}{2} - \frac{\varepsilon c^2}{12})$= 1000. 50. $(\frac{-\varepsilon c}{-\varepsilon c+1,107})$. 540. .$(\frac{-\varepsilon c}{2} - \frac{\varepsilon c^2}{12})$= 347,822 *103

Therefore ε_c=-0,185 mm/m > -2 mm/m, then the assumption was correct.

$K_a = \frac{8+\varepsilon c}{24+4.\varepsilon c} = \frac{8-0,185}{24-4*0,185}$ =0,336

$\xi = (\frac{-\varepsilon c}{-\varepsilon c+\varepsilon s1}) = (\frac{0,185}{0,185\ \ +1,107})$= 0,143

a = K_a.ξ.d_{s1} = 0,336*0,143*540 =26,021 mm

Z_{s1}= ds1-a= 540 - 26,021=513,979 mm

Exact moment = Z_{s1}. F_{s1}= 513,979 *347,822 *10 -3 = 178,770 KN.m
4.2.2 Flexural strengthening

Assume t_L =2 mm, b_L=40 mm and n_L=5

$a_L = t_L * b_L * n_L$ =2*40*5=400 mm2/m

$\Delta c_{dev} = \Delta c_{Gerät} + \Delta c_{Schnitt} + \Delta c_{Bauteil}$ =1+2+0= 3 mm

$t_s \leq c - \Delta c_{dev}$→ t_s =50-3=47 mm

$d_L = h - (t_s \frac{bL}{2})$ = 600 – (47 - $\frac{40}{2}$)= 573 mm

$\varepsilon_{Lud} = \frac{fLuk}{EL\ .\gamma LL} = \frac{2200}{170*1,2}$ = 10,784 mm/m

$\varepsilon_{LRd,max} = K_\varepsilon * \varepsilon_{Lud}$= 0,8 * 10,784=8,627 mm/m

$F_{LRd} = \varepsilon_{LRd,max} * a_L * E_L$=8,627*400*170*10-3=586,667 KN

$F_{s1d} = \frac{as1\ .\ fyk}{\gamma S} = \frac{500\,\pi\,.500}{1.15}$ *10-3= 682,955 KN

$\varepsilon_{L,0} = \varepsilon_{s1,0} + \frac{dL-ds1}{ds1}\ .\ (\varepsilon_{s1,0} - \varepsilon_{c,0}) = 1,107 + \frac{573-540}{540}$. (1,107+0,185)= 1,186 mm/m

Total strain= $\varepsilon_{L,0} + \varepsilon_{LRd,max}$ =1,186 +8,627=9,814 mm/m

By balancing the internal forces, $F_{s1d} + F_{LRd} = F_{cd}$
Assume ε_c>-2 mm/m
F_{cd}=682,955+586,667 =1269,622 KN/m

F_{cd}= b.x.f_{cd}.α_R = b.$\frac{-\varepsilon c}{-\varepsilon c+\varepsilon L,0+\varepsilon Ld,max}$.$d_L$.$\alpha_{cc}$.$\frac{fck}{\gamma C}$.$(\frac{-\varepsilon c^2}{12} - \frac{\varepsilon c}{2})$ =1000. $\frac{-\varepsilon c}{-\varepsilon c+9,814}$. 573 . 0,85 . $\frac{50}{1,5}$.$(\frac{-\varepsilon c^2}{12} - \frac{\varepsilon c}{2})$ =1269,622 *10 3
Therefore ε_c= -1,547 mm/m > -2 mm/m, then the assumption was correct.

$\xi = \frac{-\varepsilon c}{-\varepsilon c+\varepsilon L,0+\varepsilon Ld,max} = \frac{1,547}{1,547\ \ +9,814}$ = 0,136

$k_a = \frac{8+\varepsilon c}{24+4.\varepsilon c} = \frac{8-1,547}{24-4*1,547}$ = 0,362

a = K_a.ξ.d_L= 0,362*0,136*573 =28,275 mm

Z_{s1}= d_{s1}-a=540-28,275 =511,725 mm

Z_L= h-a=600-28,275 =571,725 mm

m_{Rd}=z_{s1}.F_{sld} + z_L.F_{LRd}=511,725 *682,955*10-3+571,725 *586,667 *10-3 =684,897 KN.m > 682,857 KN.m (maximum value for the ultimate moment)→ safe

$\omega = \frac{as1\ .\sigma sd-NEd}{b.d.fcd}$ where $\sigma_{sd} = \frac{fyk}{\gamma s} = \frac{500}{1,15}$ =434,782

$\omega = \frac{500\,\pi*10^{-6}.434,782-0}{1.0,540.28,3}$ = 0,045 → μ_{Eds}= 0,042

$M_{Rd,old}$=μ_{Eds}*b*d_2*f_{cd}=0,042*1*0,5402*28,3*1000=346,596 KN.m

Therefore the strengthening factor that this bridge will be strengthened with $=\frac{mRd}{MRd,old}=\frac{684,897}{375,412}$ =1,976

4.2.3 Yielding point

$F_{s1d}=\frac{as1 \cdot fyk}{\gamma s}=\frac{500\,\pi\,.500}{1.15}$ *10-3= 682,955 KN

$\varepsilon_{s1}=\frac{fyd}{Es}=\frac{500/1,15}{200000}$ *1000= 2,175 mm/m

Assume ε_c>-2 mm/m
By balancing of the internal forces, F_{s1d} = F_c
F_c= b. x. f_{ck}. α_R = b. f_{ck}. $(\frac{-\varepsilon c}{-\varepsilon c+\varepsilon s1})$.$d_{s1}$.$(\frac{-\varepsilon c}{2}-\frac{\varepsilon c^2}{12})$= 1000. 50. $(\frac{-\varepsilon c}{-\varepsilon c+2,175})$. 540. .$(\frac{-\varepsilon c}{2}-\frac{\varepsilon c^2}{12})$= 682,955*103

Therefore ε_c= -0,512 mm/m > -2 mm/m, then the assumption was correct.
$K_a=\frac{8+\varepsilon c}{24+4.\varepsilon c}=\frac{8-0,512}{24-4*0,512}$ = 0,341

$\xi=(\frac{-\varepsilon c}{-\varepsilon c+\varepsilon s1})=(\frac{0,512}{0,512\ \ +2,175})$= 0,191

a = K_a.ξ.d_{s1} = 0,341*0,191*540 =35,106 mm

Z_{s1}= d_{s1}-a=540 - 35,106 =504,894 mm

$M_{Rdy,0}$= Z_{s1}. F_{s1}= 504,894 *682,955*10 -3 = 344,820 KN/m
→ x($M_{Rdy,0}$) = 3,830 m at the cantiliver
→x($M_{Rdy,0}$) =5,476 m at the midspan

4.2.4 Anchorage

M_{cr} = κ_{fl} .$f_{ctm,surf}$.$W_{c,0}$

κ_{fl} =1.6 $-\frac{h}{1000}$=1.6 $-\frac{600}{1000}$ =1 and $W_{c,0}$ $=\frac{b.h^2}{6}$ $=\frac{1000.600^2}{6}$= 60000000

therefore M_{cr} =1*4,1*60000000*10-6=246 KN

$M_{LFl,perm}$ at x($M_{Rdy,0}$) at the cantilever ≈ 144,140 KN.m< M_{cr}

$M_{LFl,perm}$ at x($M_{Rdy,0}$) at the midspan ≈ 169,189 KN.m< M_{cr}

a_l=1*d_L=1*573=573 mm

Assume a_L=6300 mm

At the cantiliver
L_{bL} = a_L- a_l - x =6300-573-3,830 *1000=1897 mm

τ_{bGk}=k_{sys}.$\sqrt{\left(2.fGtk-2\sqrt{(fGtk^2+fGck.fGtk)}+fGck\right).fGtk}$
=0,8 . $\sqrt{\left(2.30-2\sqrt{(30^2+90.30)}+90\right).30}$ = 24 N/mm2

τ_{bck} = k_{bck} .$\sqrt{fcm}$ =4,5 .$\sqrt{58}$ = 34,271 N/mm2

τ_{bLd} =$\frac{1}{\gamma BE}$. min { τ_{bGk} . α_{bG} or τ_{bck} . α_{bc} } =$\frac{1}{1,3}$. min { 24. 0,5 or 34,271 . 0,9 } =9,231 N/mm2

$a_r=\frac{bw}{nL+1}=\frac{1000}{5+1}$ = 166,667 mm

Since l_{bL}> 115 mm→ F_{bLRd} = b_L.τ_{bLd} .$\sqrt[4]{ar}$. (26,2 + 0,065. tanh$(\frac{ar}{70})$.(l_{bL} -115)). 0,95
= 20. 9,231 .$\sqrt[4]{166,667}$. (26,2 + 0,065. tanh$(\frac{166,667}{70})$.(1897-115)). 0,95 = 176,529 KN

$F_{bLRd,ges}$= F_{bLR}d*n_L =176,529 *5=882,647 KN

F_{s1d}+F_{Ld}=F_{cd}→ 1

F_{LEd}=a_L *E_L*ε_L→ 2

F_{s1d}=a_{s1}*E_s*ε_{s1}→3

Using equation 1,2 and 3 and by iterations, ε_L and ε_s could be determined
ε_L≈ 1,9 mm/m

ε_c≈ -0,464 mm/m

F_{LEd}=a_L *E_L*ε_L=200*170*1,9/1000=129,2 KN< F_{bLRd}

At the midspan
L_{bL} = a_L- a_l - x =6300-573-5,476 *1000=251 mm

Since l_{bL}> 115 mm→ F_{bLRd} = b_L.τ_{bLd} .$\sqrt[4]{ar}$. (26.2 + 0.065. tanh$(\frac{ar}{70})$.(l_{bL} -115)). 0.95
= 20. 9,231 .$\sqrt[4]{166,667}$. (26.2 + 0.065. tanh$(\frac{166,667}{70})$. (251-115)). 0.95= 43,973 KN

FbLRd,ges= FbLRd*nL =43,973 *5=219,865 KN > FLEd
4.2.5 Shear strengthening

a_i=min. of $(\frac{h}{2}$ or $\frac{t}{2})$ =min. of $(\frac{0,6}{2}$ or $\frac{1,4}{2})$=0,3 m

Maximum shear acting on the structure acts at a distance a_i+d_{s1} so from the given values of the shear acting on the structure, the shear at x=5+0,3+0,549=6,849 m → $V_{Ed,red}$≈275,117 KN

$V_{Rd,c}$ = [$C_{Rd,c}$ .k .(100.ρ_1 .f_{ck})$^{\frac{1}{3}}$ + 0.12 .σ_{cp}].d

k = 1+$(\frac{200}{d})^{0.5}$ = 1+$(\frac{200}{549})^{0.5}$= 1,604

$\sigma_{cp}=\frac{NEd}{Ac}$ =0

$C_{Rd,c}=\frac{0.15}{\gamma c}=\frac{0.15}{1,5}$ = 0,1

$100.\rho_1 = \frac{as1}{d} = \frac{500\ \pi*10^{\wedge}2}{54,9} = 0,280$

Therefore $V_{Rd,c} = [0,1.\ 1,604.(\ 0,280.50)^{\frac{1}{3}}].540 = 209,229$ KN

$V_{Rd,c\ min} = [\ \frac{0.0525}{\gamma c}\ .\ \sqrt{k^3.fck}\ + 0.12\ .\ \sigma_{cp}\ ].d.b_w = [\ \frac{0.0525}{1,5}\ .\ \sqrt{1,604^3.50}\ ].540 = 272,653$ KN > $V_{Rd,c} \rightarrow V_{Rd,c\ min}$ is used

$a_{LE} = 500 + a_i = 500 + 0,3*1000 = 800$ mm

$V_{Rd,c,LE} = 0.75.(1 + 19.6.\ \frac{(100.\rho s1)^{0.15}}{aLE^{0.36}}).\ V_{Rd,c} = 0.75.(1 + 19.6.\ \frac{(0,280)^{0.15}}{800^{0.36}}).\ V_{Rd,}c$

$= 385,894$ KN> $V_{Ed,red}$

$\frac{VEd,red}{VRd,c\ min} = \frac{275,117}{272,653} \approx 1 < 1 \rightarrow$ acceptable, no need to be strengthened due to the shear loads.

4.2.6 Limitation of stresses

$\varepsilon_{Lo} = \varepsilon_{s1} + \frac{dL-ds1}{ds1}\ .\ (\mathcal{E}s1,0\ -\ \mathcal{E}c,0)\ =\ 1,107 + \frac{573-540}{540}\ .\ (1,107 + 0,185) = 1,186$ mm/m

First assume $\varepsilon_{L,SLS}$ and ε_c

$F_{s1}d + F_{Ld} = F_{cd} \rightarrow 1$

$F_{LEd} = a_L\ {*}E_L{*}\varepsilon_L \rightarrow 2$

$\varepsilon_{S,SLS} = \frac{-\varepsilon c+\varepsilon L,sLs+\varepsilon Lo}{dL*ds1} + \varepsilon c \rightarrow 3$

$F_{s1d} = a_{s1}{*}E_s{*}\varepsilon_{s1} \rightarrow 4$

Using equation 1,2,3 and 4 and by iterations, ε_c and $\varepsilon_{L,SLS}$ could be determined.
$\varepsilon_c = -0,390$ mm/m

$\varepsilon_{L,SLS} = 0,6$ mm/m < 2 mm/m → safe

therefore $\varepsilon_{S,SLS} = \frac{0,390\ +0,6+1,186}{573*540}\ -0,390 = 1,562$ mm/m < 2,5 mm/m → safe

The bridge needed only flexural strengthening as it was strong enough in shear; the concrete alone can sustain the shear loads acting on the section so it did not need to be shear-strengthened.

This bridge needs 5 bars with thickness 2 mm and width 40 mm every meter but this time, the CFRP will be placed in the grooves at the top of the section as shown in the following figure.

The following figures will show how the CFRP will look like when it is mounted inside the bridge

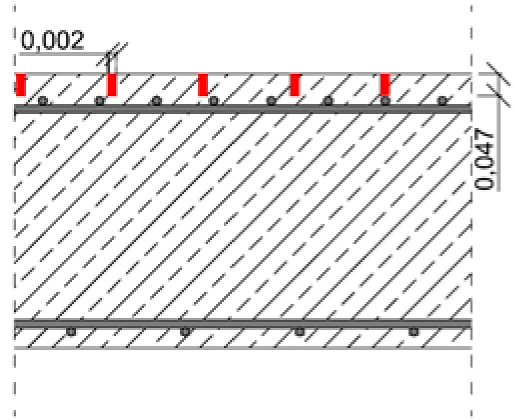


Fig.16. *Transverse section of bridge 2 after the FRP has been inserted*

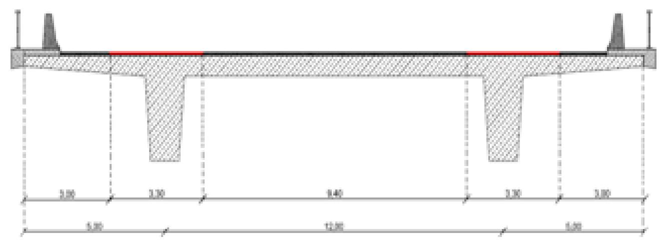


Fig.17. *Strengthened cross section A-A*

Fig.16 shows the NSM for bridge two in the longitudinal section, the length of the shown FRP is 3,30 m and it is placed as shown at the support. Fig.17 cross sectional view of bridge two showing the NSM grooved inside the bridge, 5 bars of the CFRP are placed every 1 m as shown.

5 Results and Conclusion

The type of attaching FRP can differ according to the current situation and environment of the structure that needed to be strengthened; for example in the two bridges designed earlier, the first one needed externally bonded FRP because there was no enough concrete cover so no enough space for the FRP to be placed in it while the second bridge was quite the opposite, it needed near surface mounted FRP as there was enough cover, the FRP in that case will be invisible and that is an advantage when it comes to attaching FRP at the top as it will be protected from the traffic so higher durability. Not all the bridges needed to be flexure and shear strengthened; based on the loads coming and the resistance of the structure and by comparing their values together; the strengthening due to shear or flexural loads or both would be designed. The results of the designed two bridges showed that both of them needed only to be flexure strengthened as the current structures cannot sustain the acting loads on them but they are strong enough to withstand the shear loads acting on them so they do not need to be strengthened due to the shear loads. For many years, the civil engineering community has tried to use the Fiber Reinforced Polymers as a strengthening material and they have succeeded in doing this but because it is still considered a new material used for strengthening so eyes must be kept open in every single detail about this process. Strengthening using FRP can be very effective and efficient but then again it should be used with good care in order to get over its disadvantages so that it can be used in the right way, hence engineers must improve their knowledge with respect to the actual behavior of the FRP to have a good, serviceable and cost effective structure.

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