Carbon fiber strengthening and jacking of engineering structures


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Förstärkning av betongkonstruktioner med kolfiberkompositer
Utveckling av mothåll för lyft av tunga konstruktioner
med hydraulcylindrar

Amélie Grésille

2009

Abstract

This work is divided in two parts related to repair and strengthening of engineering structures: the first part deals with the design for strengthening of concrete structures by carbon fiber composites. The work focuses on a design program that had to be corrected before being released on the market according to the following steps: presentation of design rules and materials, development of an own program, comparison of the results of this program with a similar commercial program and last, verification of the commercial program to be released.

The second part of this work deals with improvement of the design of reinforced concrete jacking consoles that are used at lifting bridge decks in order to carry out repair or replace bearing devices. In the first step, rules governing the fixing of consoles to bridge piers by pre-stressing bars were studied. Next, calculations have been carried out to reduce weight without detrimental effects on the load bearing capacity of the consoles. Finally, specifications and drawings need to manufacture the consoles have been produced.

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Summary

Title: Carbon fiber strengthening and jacking of engineering structures

Development of a program for calculation of carbon fiber reinforcement needed for strengthening;
Design of new jacking consoles

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Company supervisor: Fabien Cousteil, Method Engineer, VSL France

Problematics: 1. A program for design of carbon fiber strengthening exists, but has not been tested and therefore is not usable yet.
2. The existing jacking consoles are too heavy, difficult to handle.

Aim: 1. Carry out all the tests in order to correct the program if needed, so that it gives right results.
2. Design new jacking consoles, easier to handle and durable.

Method: 1. Create my own calculation program on Excel and use other programs to compare the results with those of the tested program. Run the program on many examples and correct display mistakes.
2. Get information about the prestressing bars that the company uses, study the tension losses due to the prestressing, choose the suitable material, design the console according to the reglementation and draw plans on AutoCAD.

Conclusion:

1. All the mistakes I noticed in the program are now corrected. The program can now be used for real projects of carbon fiber strengthening.
2. The consoles I designed have not been improved as much as I was expecting at the beginning. Nevertheless all the plans and calculations needed for the construction of the new consoles are now ready.

Key words: Carbon fiber, Ultimate Limit State (ULS), Serviceability Limit State (SLS), stress, deformation, structure, bending, strengthening, jacking, prestressing bars, reinforced concrete.

Preface

My internship at VSL France started on January 19th, 2009 and ended on June 19th, 2009, that is to say five months within the head office in Labège. My advisor in the company was Mr. Fabien Cousteil, the Method Engineer of the company.

I want to thank him, he has always been here when I needed him, when I had questions or when I needed some directions. Even though he was very busy with his own projects in the company, he always took time for me to explain the points I did not understand.

I also want to thank all my colleagues who were working on the same open space as me. Without them and their friendliness, the good conditions and atmosphere would not have been the same. These colleagues are: Marina, Morgane, Valérie, Ghislaine, Laurent, Grégory, Francis, Samy, Fabrice, and of course the two other trainees: Matthieu and Julien.

Thank you to Christian Rollet, my contact at Bouygues TP who I sent many emails to, but who was always patient and who always answered to me. Thanks to him we have been able to get a correct version of the program I was working on.

Finally, I want to thank Mr. Paul Vilar, director of VSL France, who accepted me in his company to do my internship.

Amélie Grésille

Toulouse, September 2009

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1. Introduction

1.1 Background

Many structures in reinforced concrete, like buildings or bridges, have been built 40, 50, 60 years ago and needs now to be strengthened or repaired. Their initial function can have changed or they can have been damaged by accidents, chemical reactions or any other reason. These structures can either be strenghtened in order to increase their initial strength, or they can be repaired to help them to recover their initial strength and protect them from further damages.

Carbon fiber strengthening is one of the solutions to reinforce structures. This increases the resistance of the structure by applying carbon fiber composites on the tensed parts of the structure. This carbon fiber strengthening is the subject of the first part of this work. There are several other ways to strengthen structures, as it will be developped further on.

Jacking of structures, which is the subject of the second part of this work, is necessary for different kinds of reperation. It can be used for instance to lift a building to reposition it in a correct position, or to lift a bridge deck to change old bearing devices underneath.

1.2 Objectives

Two different subjects were developped during this work.

The first one was to correct a program that gives the amount of carbon fiber needed to reinforce a structure. This program already existed when I arrived, and I had to make it run, to test it with all possible situations, to compare its result with my own results and to make a list of everything that had to be corrected. The aim was to be finally able to use it without any doubt about the results it gives, and therefore to use the results in real official design projects.

My second objective was to design new jacking consoles (the definition of jacking consoles will be given in the following chapters), in order to make them lighter so that they are easier to handle for the workers on site. In particular I had to consider the possibility of having metal consoles instead of the existing consoles made of reinforced concrete.

1.3 Presentation of the company

1.3.1 VSL France

Figure 1.1: logo of VSL France

VSL France is a company of Civil Engineering specialised in prestressing, jacking-pushing, repair of structures, heavy lifting and guying systems.
VSL France's head office is in Labège, very close to Toulouse (South West of France).

Thirty-four employees work for VSL France. The company got a turnover of €9,500,000 in 2008, which gave to the company an benefit of €480,000.
VSL France is part of the Bouygues group, which is an international industrial group present in 80 countries all over the world, with more than 145,000 employees and with a turnover of €32,7 billion in 2008.

Here is the position of VSL France in the Bouygues group.

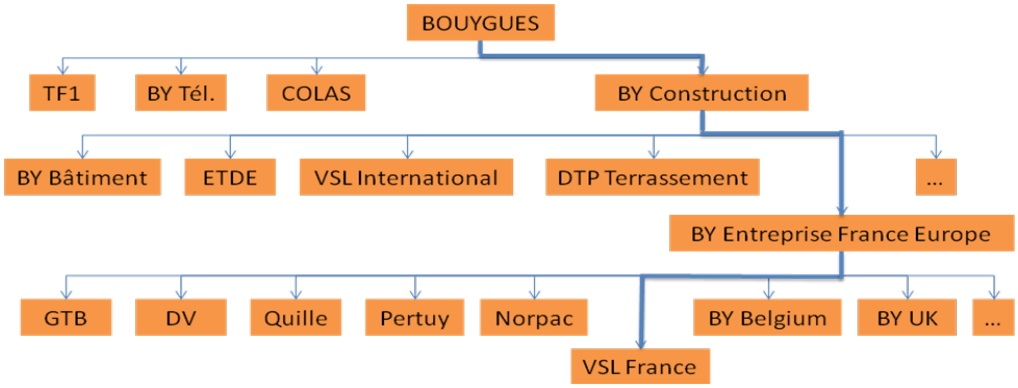


Figure 1.2: position of VSL France in the Bouygues group

As it is shown in the figure above, one of the subsidiary companies of Bouygues Construction is VSL International. VSL International is present all over the world, it has got 3,000 employees of which 900 are engineers or technicians. VSL (for *Vorspann System Losinger*) is one of the world leaders in the prestressing field. It was

a subsidiary company of Losinger, which is the third biggest group of construction in Switzerland. Bouygues Construction bought Losinger in 1990, that is how VSL became part of the Bouygues group. VSL International is now directly attached to Bouygues Construction, while VSL France is attached to Bouygues France Europe which is itself attached to Bouygues Construction.

1.3.2 Bouygues Construction

Figure 1.3: logo of Bouygues Construction

Bouygues Construction belongs to the Bouygues group, one of the largest French industrial groups, gathering more than 145,000 people all over the world and acting in the following fields:

- Buildings and Public Works with Bouygues Construction (owned at 100%)
- Real Estate with Bouygues Immobilier (100%)
- Roads with Colas (96,8%)
- Telecommunications with Bouygues Telecom (89,5%)
- Media with TF1 (43%)
- Energy – Transport with Alstom (30%)

In 2008, Bouygues Construction had 53,700 employees (including 24,700 in France) and had a turnover of €9,5 billions.

1.4 Note about this document

This paper is divided into two main parts: the first one concerns carbon fiber strengthening while the second part deals with jacking consoles.
In this second part, some calculations about the reinforcemement bars of consoles will be willfully missing because of the confidential character they have. The formulas used for these calculations, as well as the results, will be given in the report, but not the details of the calculation.
This was asked by the company since they want to use these calculations to justify the consoles they use, and they do not want everybody to have access to them.

2. Structural strengthening by carbon fiber

2.1 Principle of the strengthening

2.1.1 Different kinds of strengthening

A structural strengthening is needed when a structure has been built to resist to a particular system of loads and when this system of loads has changed with time. For example, this can happen for bridges when the trafic loads increase as the number of heavy vehicles increases, or when the deck has to be widened in order to have an additional lane. In these cases, the bridge has to be strengthened. The same with buildings: a room designed for an office for example needs to be strengthened if we change its function into a storage room.
A strengthening can also be needed when new regulations come up with more restrictive safety factors.
Sometimes, the structure needs to be *repaired* (not reinforced) when it has been damaged chemically (carbonation, alkali-aggregate reaction, corrosion of reinforcement bars…) or by accident, for instance because of earthquakes, car accidents or fire.

This work only considers strengthening of reinforced concrete.

Different kinds of strengthening are possible: shotcrete with additional reinforcement, external prestressing, and of course carbon fiber which will be studied in this report.

Shotcrete is used for the strengthening (if the loads have increased) or the repairing of the structure (if the structure's resistance has decreased because of damages and deteriorations). This method consists in applying a small layer of concrete onto a prepared surface so that the new concrete and the old concrete constitute one homogenous structure with a higher resistance than the old one, thanks to the larger amount of concrete and to a weldmesh set inside the new concrete.

Figure 2.1: shortcrete technology

Another kind of strengthening is external prestressing. As it is shown in the *Figure 2.2* below, this method consists in fixing cables at different places along the element that needs to be reinforced. These cables are then tensioned in order to have a new stressing distribution in the structure.
VSL is one of the leaders in prestressing technology, besides it has a European Technical Approval (ETA-06/0006) that gives VSL an international recognition.

Figure 2.2: external prestressing – Incarville's Viaduc

Carbon fiber strengthening will be now presented with more details, since it is the subject of this work.

2.1.2 Presentation of carbon fiber products

Two different kinds of products are available for carbon fiber strengthening: fabrics and laminates. Fabrics, also called carbon sheets (C-sheets), can be either uni-directional or bi-directional, which means that the C-sheets can take the stresses in only one direction for the uni-directional fabric or in two perpendicular directions for the bi-directional fabric.

Figure 2.3: Carbon sheet on the left picture and laminates on the right picture

For carbon fiber strengthening, carbon fibers are imbedded in a matrix of epoxy resin. The composite is bonded to the structure that has to be strengthened.
When the carbon fibers are supplied as a pre-cured laminate, the laminate is already embedded into the epoxy matrix and it is directly pasted on the surface. On the contrary, when they are supplied as a fabric, the embedment into the epoxy matrix takes place on site by hand lamination.

2.1.3 Carbon fiber: how it works

Strengthening by carbon fiber is very interesting when the loads to be taken are not too large. This strengthening method is relatively easy and fast to put in place, because carbon is a very light material and the application is very simple. Moreover, contrary to a strengthening with steel, the carbon fiber composite is not sensible to corrosion, which is favourable from a durability point of view.

Carbon fiber works by resisting a *load difference* in comparison with an initial state. Two different load states need to be considered: *at the moment* when the structure is to be strengthened, and *after* the strengthening. Indeed, the carbon fiber composite needs a load difference to work efficiently: if the structure is already fully loaded at the moment of the strengthening, it means that the carbon fiber composite is applied

on a surface that is already stretched at its maximum and the carbon fibers will therefore not be tensioned, that is to say they are useless.

The important point is then to unload the structure as much as possible before the strengthening, for example by stopping the traffic and removing a part of the permanent load by taking away the cover layer of the road. Ideally, only the dead load of the structure should remain. Once the carbon fiber strengthening has been put in place, the road cover can be applied again and the traffic is permitted. These additional loads will tend to deform the strengthened structure, but these deformations can now be taken by the carbon fiber composite through tension stresses.

2.1.4 Situations when the carbon fiber strengthening is possible

The main use of carbon fiber is to strengthen structures against bending moments and shear forces. These strengthenings are carried out according to official documents such as the *Avis Technique* in France, delivered by the CSTB (CSTB = Scientific and Technical Centre for Buildings) where the companies describe their products, their process, the terms of use. Design of carbon fiber strengthening in France is regulated by the AFGC (French Agency of Civil Engineering), whose conclusions are written in a book: *Réparation et renforcement des structures en béton au moyen des matériaux composites – Recommandations provisoires*, which means: *Repair and strengthening of concrete structures using composite materials – Temporary recommendations.*
This document also describes the process of confinement of columns by carbon fiber, in order to increase the column's resistance to a normal load.

A fourth application, currently under study by the working team of the AFGC, concerns the strengthening to punching. The carbon fiber strengthening should then replace the punching shear strengthening when it is needed.

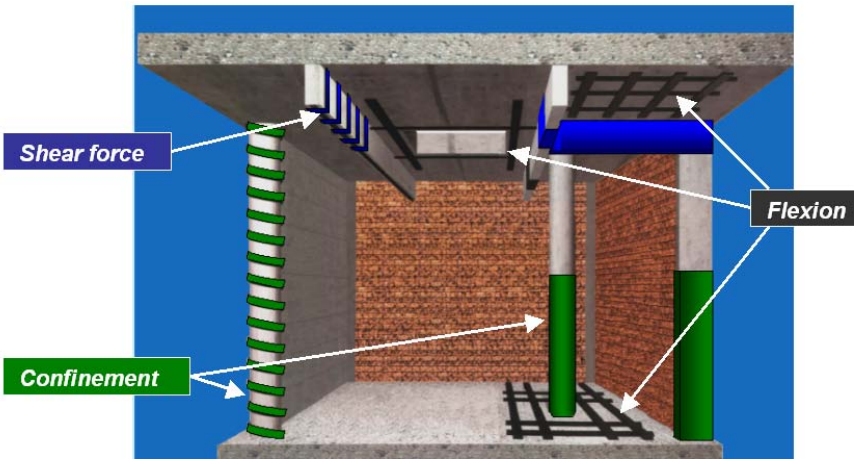


Figure 2.4: different kinds of carbon fiber strengthening

2.1.5 Calculation principle

Concerning the strengthening to bending stresses, the carbon fiber composite is applied on the surface of the member that is subject to tensile stresses to reduce deformations. The calculation principle, to know how much carbon fiber is needed, is to choose an arbitrary section of carbon fiber and to check whether the strains of each material (concrete, steel, carbon) are below the limit values or not for the Ultimate Limit State (ULS), and also to check whether the stresses in each material are below the limit values at the Serviceability Limit State (SLS). These limit values will be given in the next chapter.

Concerning the strengthening to shear forces, the carbon fiber composite is applied on the web of the beam to increase its resistance to shear forces. The calculation is very similar to that carried out to calculate the steel stirrups of reinforced concrete. In this document, I will only talk about the strengthening to bending moment, because it is the task that asked the most work.

2.2 The V2C process of VSL France

2.2.1 Carbon fiber products used by VSL France

VSL France uses two different kinds of carbon fiber products: the first ones are rolls of carbon fiber sheet, and the second are carbon fiber laminates, where the carbon fibers are already in an epoxy matrix (see also section 2.1.2).

Figure 2.5: beams and slabs reinforced by carbon fiber laminates (black strips)

The products used in the V2C process of VSL France are S&P CFK laminates and S&P C-sheets. They are manufactured by the company S&P Reinforcement. The *Avis Technique* (cf. Appendix 1) gives their main characteristics. Carbon fibers have a linear elastic brittle behaviour, with characteristics shown below:

Laminates:

	Laminate CFK 150/2000	Laminate CFK 200/2000
Modulus of elasticity E_f	165 GPa	205 GPa
Available thickness	1,2 and 1,4 mm	1,2 and 1,4 mm
Maximum strain – ULS: ε_{fd}	7 ‰	6,5 ‰
Maximum tensile stress – ULS: f_{fd}	1148 MPa	1332 MPa
Maximum tensile stress – SLS: $\sigma_{f,SLS}$	400 MPa	500 MPa

C-sheets:

	C-sheet 240 (200 g/m²)	C-sheet 240 (300 g/m²)	C-sheet 640 (400 g/m²)
Modulus of elasticity E_f	84 GPa	76,8 GPa	173 Gpa
Theoretical thickness	0,334 mm	0,550 mm	0,678 mm
Maximum strain – ULS: ε_{fd}	7 ‰	7 ‰	1,6 ‰
Maximum tensile stress – ULS: f_{fd}	590 MPa	540 MPa	280 MPa
Maximum tensile stress – SLS: $\sigma_{f,SLS}$	280 MPa	254 MPa	140 MPa

The calculation is similar to a traditional calculation of reinforced concrete and will be described more precisely further in this work.

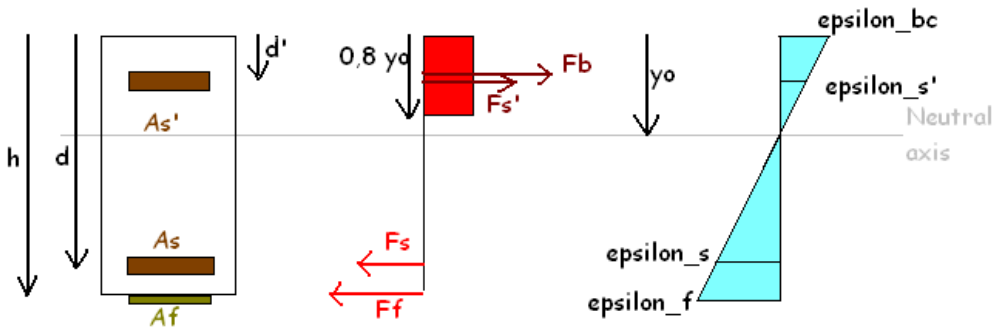


Figure 2.6: model for calculation in the Ultimate Limit State

The section of carbon fiber, A_f, should be large enough so that the total section (concrete + steel + carbon) can resist the bending moment in the ULS. The following characteristic values are known:

- Steel:
  - Modulus of elasticity: $E_s = 200\ GPa$
  - Yield stress: f_e
  - Safety factor for steel: $\gamma_s = 1{,}15$
  - Design value of the yield stress at the ULS: $f_{ed} = f_e / \gamma_s$
  - Maximum strain in the ULS: $\varepsilon_s = 10$ ‰
- Concrete:
  - Caracteristic compressive strength after 28 days: f_{c28}
  - Safety factor for concrete: $\gamma_c = 1{,}5$
  - Design value of compressive strength at the ULS: $f_{cd} = 0{,}85\ f_{c28} / \gamma_c$
  - Modulus of instantaneous longitudinal deformation of concrete after *j* days: $E_{ij} = 11000\ f_{cj}^{1/3}$ with f_{cj} in MPa

  - Modulus of long term longitudinal deformation of concrete after *j* days: $E_{vj} = 3700\ f_{cj}^{1/3}$
  - Maximum shortening of concrete at the ULS: $\varepsilon_{cu} = 3{,}5$ ‰

- Carbon fiber:
  - Linear stress-elongation curve: $\sigma_f = E_f \cdot \varepsilon_f$ (these values are given in the tables on the previous page, they depend of the product).

2.2.2 Calculation in the Ultimate Limit State (ULS)

A first formula gives the minimum section needed of carbon fiber, with the assumption that both the compressed and the tensioned steel bars are in the plastic domain in the ULS (which means: $\sigma_s = f_{ed} = f_e / \gamma_s$).

Equation of bending moments around the carbon fiber composite:

$$(1)\ M_u = A_{s'} \cdot f_{ed} \cdot (h - d') - A_s \cdot f_{ed} \cdot (h - d) + b_0 \cdot 0{,}8 y_0 \cdot f_{cd} \left(h - 0{,}4 y_0\right)$$

where:

- M_u = bending moment at the ULS
- $A_{s'}$ = section of the compressed steel bars
- A_s = section of the tensioned steel bars
- h = height of the concrete beam section
- b_0 = width of the concrete beam section
- d = distance between the upper concrete fiber (the most compressed) and the center of gravity of the tensioned steel bars
- d' = distance between the upper concrete fiber and the center of gravity of the compressed steel bars
- y_0 = distance between the upper concrete fiber and the neutral axis of the section

The simple rectangle diagram is used to describe the stress distribution in the concrete: the stress is equal to f_{cd} from the upper fiber of the concrete until $0{,}8\ y_0$, and it is zero elsewhere, as it is explained in the *BAEL* (article A.4.3,42) which is the French reglementation about reinforced concrete.

If we call z the distance between the resultant of the compressive stresses of the concrete (which is $0{,}4\ y_0$ far from the upper concrete fiber) and the carbon fiber composite (which is at a distance h from the upper concrete fiber), we have the following relation: $z = h - 0{,}4y_0$.

The *Avis Technique* introduces a new parameter:

$$\mu_u = 0{,}8\ y_0\ (h - 0{,}4y_0) / h^2 = (2h - 2z) \cdot z / h^2$$

Equation (1) can then be written:

$$(2)\quad M_u = A_{s'} \cdot f_{ed} \cdot (h - d') - A_s \cdot f_{ed} \cdot (h - d) + b_0 \cdot \mu_u h^2 \cdot f_{cd}$$

Which can also be written:

$$(3)\quad \mu_u = \frac{M_u + A_s \cdot f_{ed} \cdot (h - d) - A_{s'} \cdot f_{ed} \cdot (h - d')}{b_0 \cdot f_{cd} \cdot h^2}$$

In this equation (3), all the elements of the right part are known, which means that we can calculate the numerical value of μ_u.

Then, the equation: $\mu_u = (2h - 2z) \cdot z / h^2$

which can also be written: $z^2 - hz + \frac{\mu_u h^2}{2} = 0$

has got $z = 0,5h\left(1 + 2\sqrt{1 - 2\mu_u}\right)$ as a solution, which gives the numerical value of z, since h and μ_u are known.

Equation of bending moments around the resultant of compressive stresses in the concrete (which is at a distance z from upper concrete fiber):

$$M_u = A_s \cdot f_{ed} \cdot (z - (h - d)) + A_{s'} \cdot f_{ed} \cdot ((h - d') - z) + A_f \cdot z \cdot f_{fd}$$

where A_f is the minimum section of carbon fiber needed to resist to the bending moment M_u.

This last equation gives A_f:

$$A_f = \frac{M_u - A_s \cdot f_{ed} \cdot \left(z - \left(h - d\right)\right) - A_{s'} \cdot f_{ed} \cdot \left(\left(h - d'\right) - z\right)}{z \cdot f_{fd}}$$

Now that the section A_f is known, it is possible to know the strains in each material and to make sure that they are lower than the limits:

- strain of the carbon fiber: $\varepsilon_f = \varepsilon_{fd}$ (maximum elongation at the ULS);
- strain of the upper concrete fiber: $\varepsilon_c \leq 3,5‰$;
- strain of the tensioned steel bars: $\varepsilon_{s,inf} \leq 10‰$;

The initial assumptions ($\varepsilon_{s,inf} > f_{ed} / E_s$ and $\varepsilon_{s,sup} > f_{ed} / E_s$) can now be checked. If the first assumption is not valid, it means that the tensile steel bars have not reached their yield limit when the failure occurs. The section will then have a brittle failure in the ULS, which is not wanted. This is a drawback of carbon fiber strengthening: the ductility of the section decreases, since the maximal strain of carbon fiber ε_{fd} is lower than 10‰, and that the final stress in tensile steel bars $\varepsilon_{s,inf}$ is lower than ε_{fd} .

2.2.3 Calculation in the Serviceability Limit State (SLS)

With the section A_f of carbon fiber given in the previous paragraph, the section can resist to the bending moment at the ULS. It is now necessary to make sure that this section is sufficient to stay below the limit stress values of each material at the SLS.

This calculation is done in two steps. The first one consists in calculating the stresses in the different materials *just before* the strengthening, when the structure is ready for the strengthening but not reinforced yet, which means when the structure has been unloaded as much as possible. These stresses should be lower than the limit values (they will be detailed below). Otherwise the carbon fiber strengthening is useless.

Then, if the stresses are acceptable *just before* the strengthening, it is time for the second step: to check that the structure *with* carbon fiber composite can resist to *the final bending moment* at the SLS, when all the loads are taken into account (dead weight + service loads). These additional loads will create a variation of the bending moment: $\Delta M_s = M_{sf} - M_{s0}$ where M_{sf} is the final bending moment and M_{s0} is the bending moment just before the strengthening. The idea is to calculate the stresses in each material (including in the cartbon fiber composite, which was not present in the first step) corresponding to a bending moment ΔM_s. The final stresses will then be equal to the sum of the stresses of the second step (section with carbon fiber with a bending moment ΔM_s) and the stresses of the first step (section without carbon fiber and with a bending moment M_{s0}). These final stresses have to be smaller than the limit values given below:

- Upper concrete fiber: $\sigma_{c,ELS} = 0{,}6\, f_{c28}$

- Compressed and tensioned steel bars:
  - Non-damaging cracks: $\sigma_{sinf,ELS} = \sigma_{ssup,ELS} = f_e$
  - Damaging cracks:

    $$\sigma_{sinf,ELS} = \sigma_{ssup,ELS} = Min\left\{\frac{2}{3} f_e \,;\, Max\left(0{,}5 f_e \,;\, 110\sqrt{\eta f_{tj}}\right)\right\}$$

    where $\eta = 1$ for plain bars, 1.6 for high adhesion bars with a diameter higher than 6 mm, and 1.3 mm for high adhesion bars with a diameter lower than 6 mm (*BAEL*, A.4.5,33),
    and $f_{tj} = 0{,}6 + 0{,}06\, f_{cj}$ (MPa) is the tensile strength of a concrete that has a compressive strength of f_{cj} MPa (*BAEL*, A.2.1,12).

- Carbon fiber:
  - Laminate CFK150/2000: 400 MPa;
  - Laminate CFK200/2000: 500 MPa;
  - C-sheet 240 (200g/m²): 240 MPa;
  - C-sheet 240 (300g/m²): 254 Mpa;
  - C-sheet 640 (400g/m²): 140 MPa.

If the calculated final stresses in the materials are larger than the limits given above, it means that more carbon fiber is needed, the section A_f has to be increased in order to satisfy the SLS.

2.2.4 Verification of the slip stress

The shear stress τ_{slip} between the carbon fiber composite and the concrete has to be lower than the limit stress τ_u at the ULS.

According the *Avis Technique* V2C, the limit slip stress is:

$$\tau_u = \min\left(2\,MPa\,;\frac{f_{tk}}{1,5}\right)$$

where f_{tk} is the characteristic superficial cohesion of concrete, that can be found by doing cohesion tests on the concrete (on site).

The slip stress in the composite at the ULS is calculated with the following formula:

$$\tau_{slip} = 4 \times \frac{A_f \times \sigma_{f,ELU}}{b_f \times L_f}$$

where: $A_f = t_f \times b_f$ is the section of carbon fiber composite;

$\sigma_{f,ELU}$ is the stress in the composite of section A_f, submitted to the bending moment in the ULS;

b_f is the composite's width in contact with the concrete surface;

L_f is the composite's length in contact with the concrete surface;

t_f is the composite's thickness.

2.2.5 Verification of the anchorage of laminates

The ancorage length is the minimum length necessary so that the stresses can be transferred from the composite to the structure. According to the *Avis Technique V2C*, studies have been carried out about anchorage zones of carbon fiber laminates, and they showed that there is a peak of stresses in this area: they are much larger than the stresses average and they can lead to the delamination of the composite. The area where strengthening is needed is called the effective strengthening zone. When the structure is strenghened by carbon fiber laminates, an anchorage length L_{bd} is needed on both sides of this effective strengthening zone, to ensure the transfer of stresses to the concrete structure.

According to the *Avis Technique* V2C, the anchorage length L_{bd} is given by:

$$L_{bd} = \sqrt{\frac{E_f \times t_f}{f_{ctd}}}$$

where E_f is the modulus of elasticity of the composite, in MPa;
t_f is the thickness of the composite, in mm;
$f_{ctd} = \tau_u$ is the design value of the superficial cohesion of the concrete, in MPa;

$$f_{ctd} = \min\left(2\,MPa\,;\frac{f_{tk}}{1,5}\right).$$

The anchorage length for sheets is much smaller than for laminates, since they are much thiner and wider. According the *Avis Technique*, a 10 cm anchorage length is sufficient.

The maximum force that the anchorage zone can resist is:

$$F_{bd} = 0,5 \times b_f \times k_f \times k_t \times \sqrt{E_f \times t_f \times f_{ctd}}$$

where b_f is the composite's width in contact with the concrete, in mm;
$k_t = 1$ if the structure is indoor and $k_t = 0,9$ outdoor;
k_f is a shape factor (for example the larger b_f is compared to b_0, the less diffusion of stress can occur in the concrete, the smaller the capacity force F_{bd} is):

$$k_f = 1,06 \times \sqrt{\frac{2 - \frac{b_f}{b_0}}{1 + \frac{b_f}{400}}}\ ;$$

where b_0 is the width of the concrete section that needs to be reinforced, in mm.

We need to check that the force in the composite at the beginning of the anchorage zone (at a point called point E, which is at a distance X_E from the support) is lower than the maximum acceptable force F_{bd}. The position of X_E is shown below:

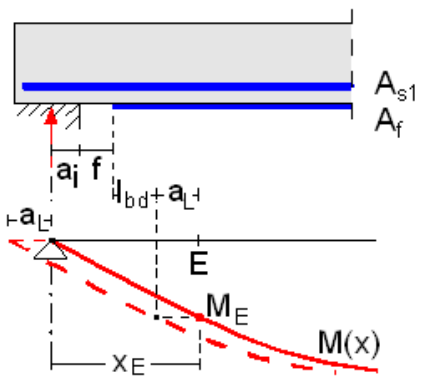


Figure 2.7: position of the point E, where the verification of the anchorage is done

$X_E = a_i + f + L_{bd} + a_L$

With:
a_i = off-set due to the real support, which is not just a theroretical point;

f = distance between the support extremity and the laminate extremity;
L_{bd} = anchorage length;
a_L = off-set of the curve of moments, due to the presence of cracks.

To calculate the force in the composite at the position X_E, we need to know the bending moment at X_E, then we calculate the stress in the laminate $\sigma_{f,E}$ at this point. The force at the position X_E is:

$$F_{f,E} = \sigma_{f,E} A_f$$

Finally we need to check that:

$$F_{f,E} < F_{bd}$$

2.3 Development of a calculation program

A special calculation program on Excel was created in order to test the V3C program.

An existing design software was also used to compare the results of the V3C program: *S&P FRP Lamella* ("FRP" stands for "Fiber Reinforced Polymer"), which is a program created by the Swiss company *S&P Clever Reinforcement Company AG*, provider of carbon fiber elements for VSL France. The problem is that this program does not take into account the French standard, concerning for example stress limits of materials. This software was however very useful to test the Excel program that was created for this work, and then the V3C program. The objective was to check that the results were similar between the three programs: the V3C that had to be tested, the existing program *Lamella*, and the Excel program developped in this work.
Creating a personal Excel program is a good tool for the correction of the V3C program: first, it is a way to understand and apply the design method that is described in the document of the AFGC, and more particularly in the *Avis Technique V2C*. Then, once it is running correctly (the results are similar to those of the software *S&P Lamella* mentioned above), it is an essential way to detect possible mistakes in the V3C program: formulas of the Excel program could be changed until it gives the same result as the V3C program (a factor 2 missing for example, or a problem of units…).

The following part will describe this Excel program.

2.3.1 Data that the program needs to run

The data that the program needs to run are the same as the ones the V3C program needs (the V3C will be presented further). The user has to enter them.
First, the geometry of the section has to be entered (it can be a rectangular beam or a T-shaped beam).

Data:

Geometry		
	h (m)	*0,800*
	b (m)	*0,500*
	Cover layer (m)	*0,060*
	d' (m)	*0,060*
	d (m)	0,740

Then, we need to know the characteristics of the concrete: the characteristic compressive strength, the modulus of elasticity, the characteristic tensile strength, from which we can then calculate the design values (ULS and SLS):

Concrete	
fcj (Pa)	*2,500E+07*
gamma_c	*1,500E+00*
Ec (Pa=N/m²)	1,082E+10
fcd (Pa)	1,417E+07

The same for the tensioned and compressed steel bars: we need to know the characteristic yield limit of the steel, the modulus of elasticity and the total section of steel.

Tensioned steel bars	
fe (Pa)	*4,000E+08*
Es (Pa=N/m²)	*2,000E+11*
gamma_s	*1,150*
Section As1 (m²)	*6,980E-04*
fed (Pa)	3,478E+08
epsilon_elast ELS	2,000E-03
n	18,486

Compressed steel bars	
fe (Pa)	*4,000E+08*
Es (Pa)	*2,000E+11*
gamma_s	*1,150*
Section As2 (m²)	*1,570E-04*
fed (Pa)	3,478E+08
epsilon_elast ELS	2,000E-03

Then we need to know the characteristics of the carbon fiber composite: the modulus of elasticity and the limit stress at the ULS and the SLS (given in the *Avis Technique*)

Carbon	
Ef (Pa)	*1,650E+11*
ffd (ELU) (Pa)	*1,148E+09*
sigma_f lim (ELS) (Pa)	*4,000E+08*
n'	15,251

Finally we need to know the loads that apply on the structure: the bending moment at the ULS (M_u), the bending moment at the SLS just before the strengthening (M_{s0}), and the bending moment at the SLS with the final loads (M_{sf}).

Loads		
	Mu (N.m)	*6,000E+05*
	Msf (N.m)	*2,500E+05*
	Ms0 (N.m)	*1,000E+05*

2.3.2 Minimum section of carbon fiber

Thanks to the formulas of the *Avis Technique* that have been explained previously (cf. section 2.2.2), we can find the minimum section of carbon fiber that is needed to fulfill the ULS. This section appears in dark grey in the table below.
In the next cell (light grey in the table below), the user decides the section of carbon fiber they want to have (preferably higher than the minimum section above). It is this last value that will be used to do all the following calculations.
In the example below, the user chose to take the minimum section of carbon fiber.

Results:

V3C ELU

mu_u	1,267E-01
z	7,456E-01
section Af mini	5,068E-04
Af chosen	5,068E-04

2.3.3 Verification of the stresses at the SLS

The stresses in each material are first calculated just before the strengthening. It means that the whole section has for the moment only concrete and steel bars, there is no carbon fiber composite yet. The bending moment is M_{s0}.

The stresses are calculated as it is explained in the *Avis Technique* V2C: first the position y_0 of the neutral axis is found thanks to the modular ratio steel/concrete (called *n* in the *Avis Technique* V2C). The section is supposed to be cracked.

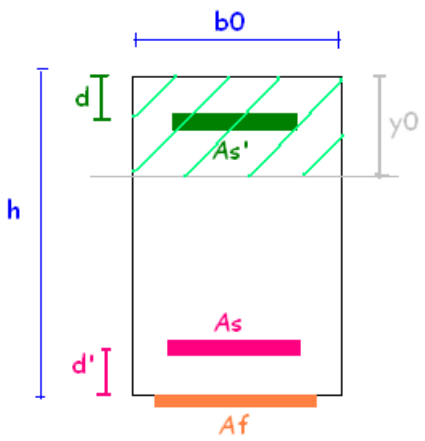


Figure 2.8: model of a reinforced concrete section

Moment equilibrium around the neutral axis y_0 gives:

$$b_0 y_0 \times \frac{y_0}{2} + nA_{s'}\left(y - d'\right) = nA_s\left(d - y_0\right) + n' A_f\left(h - y_0\right)$$

This gives an equation of degree 2 for y_0, and the solution of this equation is:

$$y_0 = \frac{-n\left(A_s + A_{s'} + \frac{n'}{n} A_f\right) + \sqrt{\Delta}}{b_0}$$

where: $$\Delta = \left[n\left(A_s + A_{s'} + A_f \frac{n'}{n}\right)\right]^2 + 2nb_0\left(A_s d + A_{s'} d' + A_f \frac{n'}{n} h\right)$$

At that time, there is no carbon fiber yet, so we have $A_f = 0$.

Δ	9,972E-03
y0	1,681E-01
inertia I0	5,046E-03

Once the value of y_0 is known, it is possible to get the moment of inertia I_0 of the section without carbon fiber. Then it is easy to have the stresses in each material:

- concrete: $\sigma_{bc0} = \frac{M_{S0}}{I_0} y_0$
- tensioned steel bars: $\sigma_{\sin f\,0} = n \times \frac{M_{S0}}{I_0}\left(d - y_0\right)$

- compressed steel bars: $\sigma_{s\sup 0} = n \times \frac{M_{S0}}{I_0}\left(y_0 - d'\right)$

sigma_bc0 (Pa)	3,332E+06
sigma_s10 (Pa)	2,095E+08
sigma_s20 (Pa)	3,961E+07

Then it is time for the second step: after strengthening with the final loads at the SLS. The variation of moment $\Delta M_s = M_{s0} - M_{sf}$ creates new stresses in the materials. We do the same calculations as in the first step, but this time with the presence of carbon fiber. The modular coefficient carbon composite / concrete is called *n'*. There is therefore a new moment of inertia I_1 due to the carbon fiber composite, which implies a new position y_1 of the neutral axis.

Δ	1,646E-02
y1	2,095E-01
inertia I1	7,924E-03

The additional stresses in each material, due to the variation of bending moment ΔMs, are:

- concrete: $\Delta\sigma_{bc} = \frac{\Delta M_S}{I_1} y_1$
- tensioned steel bars: $\Delta\sigma_{s\inf} = n \times \frac{\Delta M_S}{I_1}\left(d - y_1\right)$
- compressed steel bars: $\Delta\sigma_{s\sup} = n \times \frac{\Delta M_S}{I_1}\left(y_1 - d'\right)$
- carbon fiber: $\Delta\sigma_f = n' \times \frac{\Delta M_S}{I_1}\left(h - y_1\right)$

delta sigma_bc (Pa)	3,966E+06
delta sigma_s1 (Pa)	1,856E+08
delta sigma_s2 (Pa)	5,232E+07
delta sigma_f (Pa)	1,705E+08

Finally we can have the final stresses in each material, by adding up stresses of the first step (before strengthening) with stresses of the second step due to the variation of bending moment (superposition principle):

- concrete: $\sigma_{bc} = \sigma_{bc0} + \Delta\sigma_{bc}$
- tensioned steel bars: $\sigma_{s\inf} = \sigma_{s\inf 0} + \Delta\sigma_{s\inf}$
- compressed steel bars: $\sigma_{s\sup} = \sigma_{s\sup 0} + \Delta\sigma_{s\sup}$

- carbon fiber: $\sigma_f = 0 + \Delta\sigma_f$

sigma_bc (Pa)	7,298E+06	< 0,6*fc28	1,500E+07	OK
sigma_s1 (Pa)	3,952E+08	< fe	4,000E+08	OK
sigma_s2 (Pa)	9,193E+07	< fe	4,000E+08	OK
sigma_f (Pa)	1,705E+08	< sigma_f lim	4,000E+08	OK

The program shows whether these final stresses are lower than the limit values or not. If this is correct, it means that the chosen section of carbon fiber is correct for the SLS. If the final stresses are higher than the limit values, the user needs to enter a larger section of carbon fiber and to do the verification again, until the final stresses are low enough to fulfill the SLS requirements.

2.3.4 Verification of the ULS

As it has already been said, the *Avis Technique V2C* gives a minimum value of the section of carbon fiber that is needed so that the whole section can resist to a bending moment M_u at the ULS.

The program developed in this work can calculate the resistance moment M_{Rd} of the reinforced beam at the ULS thanks to the "pivot method". Once this moment is known, it is simple to verify that M_{Rd} is larger than the bending moment M_u in the ULS.

Here is how the "pivot method" works: all the sections of the materials are known (the user gives them himself in the program). The only thing missing to be able to calculate the deformations in each material is the position of the neutral axis y_0 at the ULS. The idea is to work by iteration: the user chooses arbitrarily a value for y_0. Once we have y_0, we can find out which material will be the "pivot", it means which material will reach its limit deformation first. If it is the tensioned steel bars that will reach their limit first (ε_s = 1%), it will be the Pivot A. For the concrete, it will be the Pivot B (ε_c = 0.35%), and for the carbon, it will be the Pivot D.

Figure 2.9 below shows the way we know what will be the pivot once we have the position of the neutral axis. This figure represents the strain distribution in a section of reinforced concrete without carbon fiber, to be easier to understand. However the principle for a section with carbon fiber is the same.

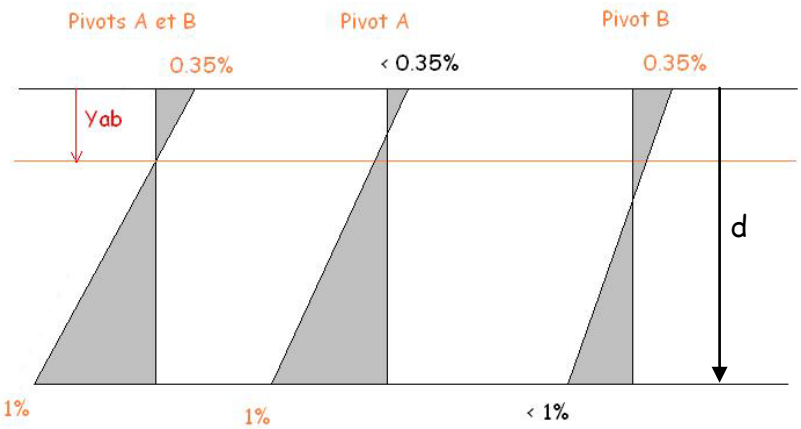


Figure 2.9: choice of the pivot according to the position of the neutral axis

If the position of the neutral axis y_0 is equal to $y_{AB} = \frac{0,35}{0,35+1} \times d$ where d is the position of the tensioned steel bars, then both steel and concrete are pivots: they reach their limit at the same time, for the same bending moment (this is what is called balanced failure).
If $y_0 < y_{AB}$, then the tensioned steel bars reach their limit first, so it is Pivot A.
If $y_0 > y_{AB}$, then the concrete reaches its limit first, so it is Pivot B.

For a section with carbon fiber underneath, the linearity of the strain distribution implies that the elongation in the carbon fiber will be higher than the elongation in the tensioned steel bars. Yet the limit value of elongation of the carbon fiber is 0.7% maximum, while the limit value of elongation of the tensioned steel bars is 1%. This means that the steel can never reach its limit value of 1% elongation, because the carbon fiber will always reach its limit value of 0.7% first.
In a section with carbon fiber strengthening, the pivot is therefore either Pivot B (concrete) or Pivot D (carbon fiber composite).

To know whether the pivot is Pivot B or Pivot D, the position of the neutral axis has to be compared not to y_{AB} but to $y_{BD} = \frac{0,35}{0,35+\varepsilon_{fd}} \times h$ where h is the beam's height and ε_{fd} is the design limit elongation at the ULS (this value is given in the Avis Technique, it depends on the kind of carbon fiber composite that is chosen).

To put in a nutshell:

1. The user chooses a position for the neutral axis y_0;
2. By comparison to y_{BD} they know what material is the Pivot, which means that they know the strain in this material;

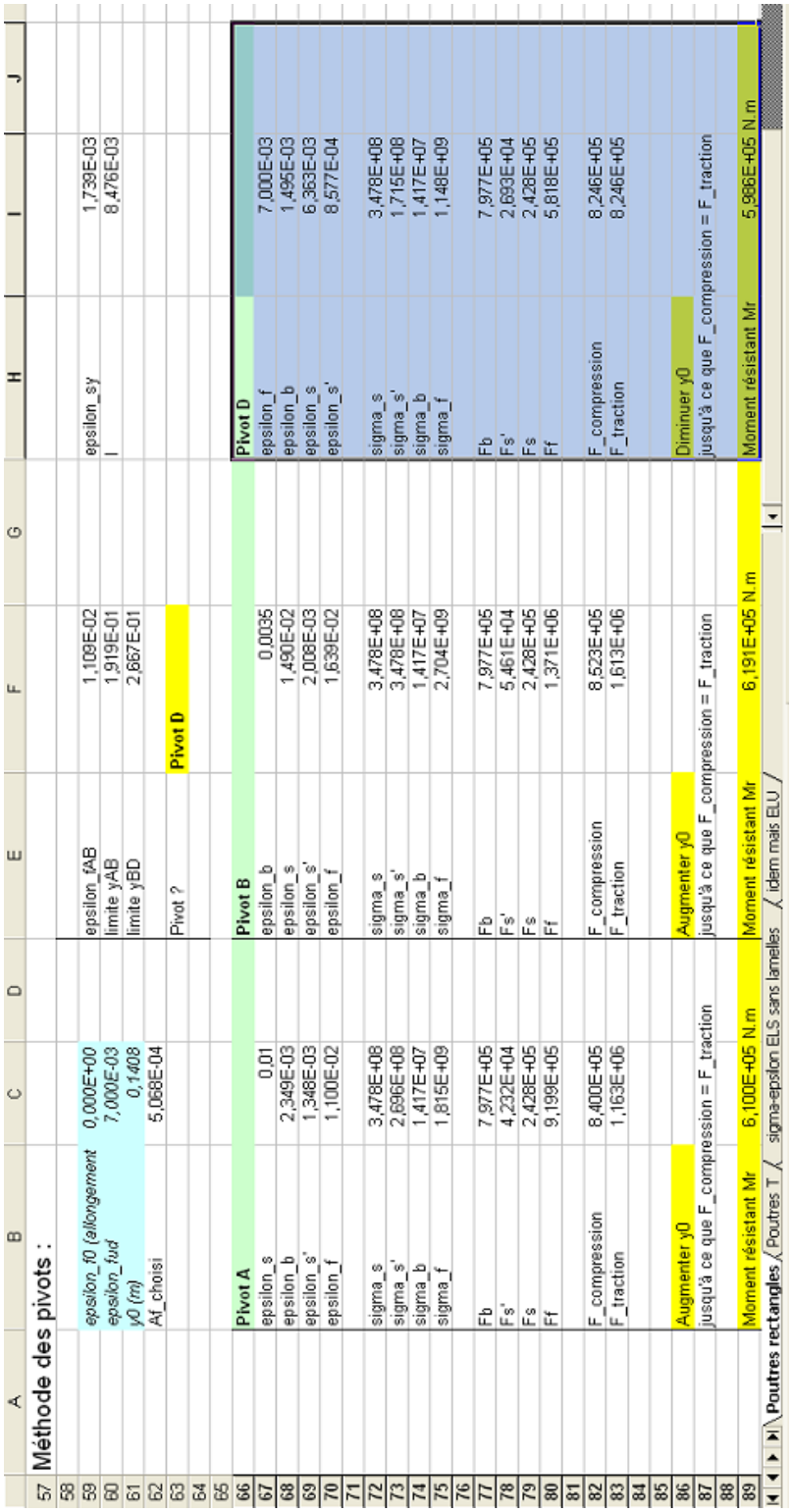
3. By knowing one strain and the position of the neutral axis, they can deduct the strains of all the other materials;
4. By knowing all the strains, they can deduct all the stresses since they know the constitutive laws for each material;
5. From these stresses, they can deduct all the resultant forces:
   - Concrete: rectangular diagram is used for the distribution of stresses. This gives a resultant compressive force: $F_b = 0.8 y_0 \times b \times f_{cd}$, at the distance $z = 0.4\, y_0$ from the upper concrete fiber
   - Compressed steel bars: $F_{s,\sup} = A_{s,\sup} \times \sigma_{s,\sup}$
   - Tensioned steel bars: $F_{s,\inf} = A_{s,\inf} \times \sigma_{s,\inf}$
   - Carbon fiber composite : $F_f = A_f \times \sigma_f$
6. The first assumption about the position of the neutral axis has now to be checked: if this position were correct, the equilibrium of forces should give: $F_{compression} = F_{tension}$, or in other words: $F_b + F_{s,sup} = F_{s,inf} + F_f$.
7. If the equilibrium above is correct (this is very rare after the first attempt…), it means that the assumption about y_0 was correct, and that all calculations and deductions from point 2 to point 6 were correct. If the equilibrium does not work with the values of point 5, it means that the chosen y_0 was wrong. The user should start again this list of steps with a new chosen y_0, having in mind that if $F_{compression} > F_{tension}$, the value for y_0 should be decreased, and if $F_{compression} < F_{tension}$, the value for y_0 should be increased.

At the end of the iteration, all the strains, stresses and forces are known. The resistant moment at the ULS is therefore (moment equilibrium around the carbon composite):

$$M_{Rd} = F_b \times \left(h - 0,4 y_0\right) + F_{s,\sup} \times \left(h - d'\right) - F_{s,\inf} \times \left(h - d\right)$$

It is now possible to compare the bending moment at the ULS M_u to the resistant moment M_{Rd}. If all goes well, the result should be $M_u < M_{Rd}$.

The example on the following page shows the steps of the calculation. All the characteristics of the materials, the loads and the geometry do not appear on this figure, they are in other cells.



	A	B	C	D	E	F	G	H	I	J
57	Méthode des pivots :									
58										
59		*epsilon_f0 (allongement*	*0,000E+00*		epsilon_fAB	1,109E-02		epsilon_sy	1,739E-03	
60		*epsilon_fud*	*7,000E-03*		limite yAB	1,919E-01		I	8,476E-03	
61		*y0 (m)*	*0,1408*		limite yBD	2,667E-01				
62		Af_choisi	5,068E-04							
63					Pivot ?	**Pivot D**				
64										
65										
66		**Pivot A**			**Pivot B**			**Pivot D**		
67		epsilon_s	0,01		epsilon_b	0,0035		epsilon_f	7,000E-03	
68		epsilon_b	2,349E-03		epsilon_s	1,490E-02		epsilon_b	1,495E-03	
69		epsilon_s'	1,348E-03		epsilon_s'	2,006E-03		epsilon_s	6,363E-03	
70		epsilon_f	1,100E-02		epsilon_f	1,639E-02		epsilon_s'	8,577E-04	
71										
72		sigma_s	3,478E+08		sigma_s	3,478E+08		sigma_s	3,478E+08	
73		sigma_s'	2,696E+08		sigma_s'	3,478E+08		sigma_s'	1,715E+08	
74		sigma_b	1,417E+07		sigma_b	1,417E+07		sigma_b	1,417E+07	
75		sigma_f	1,815E+09		sigma_f	2,704E+09		sigma_f	1,148E+09	
76										
77		Fb	7,977E+05		Fb	7,977E+05		Fb	7,977E+05	
78		Fs'	4,232E+04		Fs'	5,461E+04		Fs'	2,693E+04	
79		Fs	2,428E+05		Fs	2,428E+05		Fs	2,428E+05	
80		Ff	9,199E+05		Ff	1,371E+06		Ff	5,818E+05	
81										
82		F_compression	8,400E+05		F_compression	8,523E+05		F_compression	8,246E+05	
83		F_traction	1,163E+06		F_traction	1,613E+06		F_traction	8,246E+05	
84										
85										
86		Augmenter y0			Augmenter y0			Diminuer y0		
87		jusqu'à ce que F_compression = F_traction			jusqu'à ce que F_compression = F_traction			jusqu'à ce que F_compression = F_traction		
88										
89		Moment résistant Mr	6,100E+05 N.m		Moment résistant Mr	6,191E+05 N.m		Moment résistant Mr	5,986E+05 N.m	

Poutres rectangles / Poutres T / sigma-epsilon ELS sans lamelles / idem mais ELU

Figure 2.10: results given by the "Pivot method"

2.4 The calculation program V3C

2.4.1 History of the program

The V3C program was created by the computation department of Bouygues TP. Like the program developed during this work and described in the previous section, the V3C program is supposed to tell the user if a given section of carbon fiber will be sufficient to make the beam resist the loads in the ULS and the SLS.

The V3C program was in an evaluation period when this work started: a version of the program was available for VSL France but it was not guaranteed that it was running correctly, so it could not be used in official calculation notes, for real projects. The goal of this work was then to check the validity of the results, in order to be able to use the V3C program in real design projects.

The interface of the V3C program is an Excel file, but this Excel file uses a more complicated program called “SBAEL” that does all the calculations of stresses and deformations. I did not have any access to the program code, I could only run the “SBAEL” and see the results on the Excel sheet. However here comes a short description of how the “SBAEL” program works.

The program “SBAEL” works like a finite element program: once the section is known, the program creates a mesh.

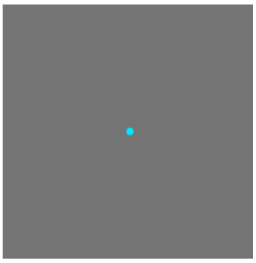


Figure 2.11: piece of mesh made of concrete, with the element point in the middle

Figure 2.12: section of a concrete beam (in grey) with four steel points of mesh (in black)

For the bending design, a first calculation is done for the SLS whithout carbon fiber, at the initial state. Then another calculation is done on the deformed beam, this time with carbon fiber composites, at the final state. A third calculation is done for the ULS, without the initial step.

The calculation is done this way: a small variation of strain is applied and the stresses are calculated. Then, forces and moments in the whole beam are calulated by integration. These forces and moments are compared with the external bending moment and normal force. If the equilibrium is not reached, another small variation of strain is applied, and the calculations are done again, until the equilibrium is reached or the limit number of iterations is reached. In this last case, the equilibrium is supposed imposible to reach, which means failure of the beam.

My task was to run the programs for various configurations and to identify differences. Identified differences were reported to the creator of the V3C program at Bouygues TP for further verifications.

2.4.2 Description of the V3C program

The user interface of the V3C program has four sheets: one called “Démarrage” that can be translated by “Start”, “Flexion” meaning “Bending”, “Ancrage” meaning “Anchorage” and “Tranchant” meaning “Shear force”.

“Start” sheet: the user can write the name of the project, its identification number and other kinds of information. It is similar to a “book cover”.

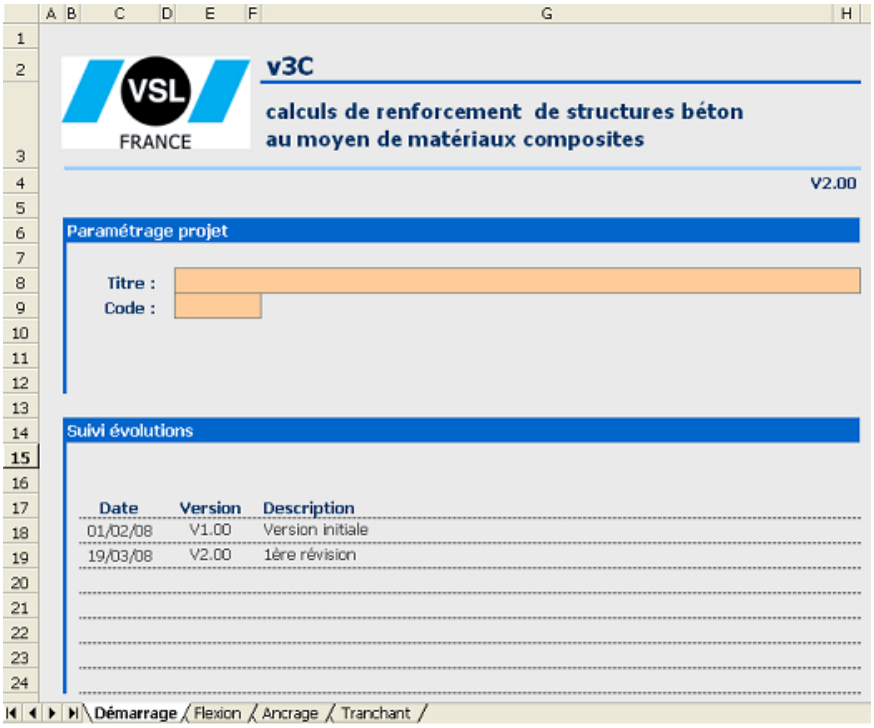


Figure 2.13: “Start” sheet of the Excel file V3C

“Bending” sheet: is the tab that the Excel program created in this work and presented in section 2.3 is supposed to check. The user chooses a section (geometry, materials) with loads applying to this section and by clicking on the macroinstruction “SBAEL” at the bottom of the page, deformations and stresses in each materials are calculated.

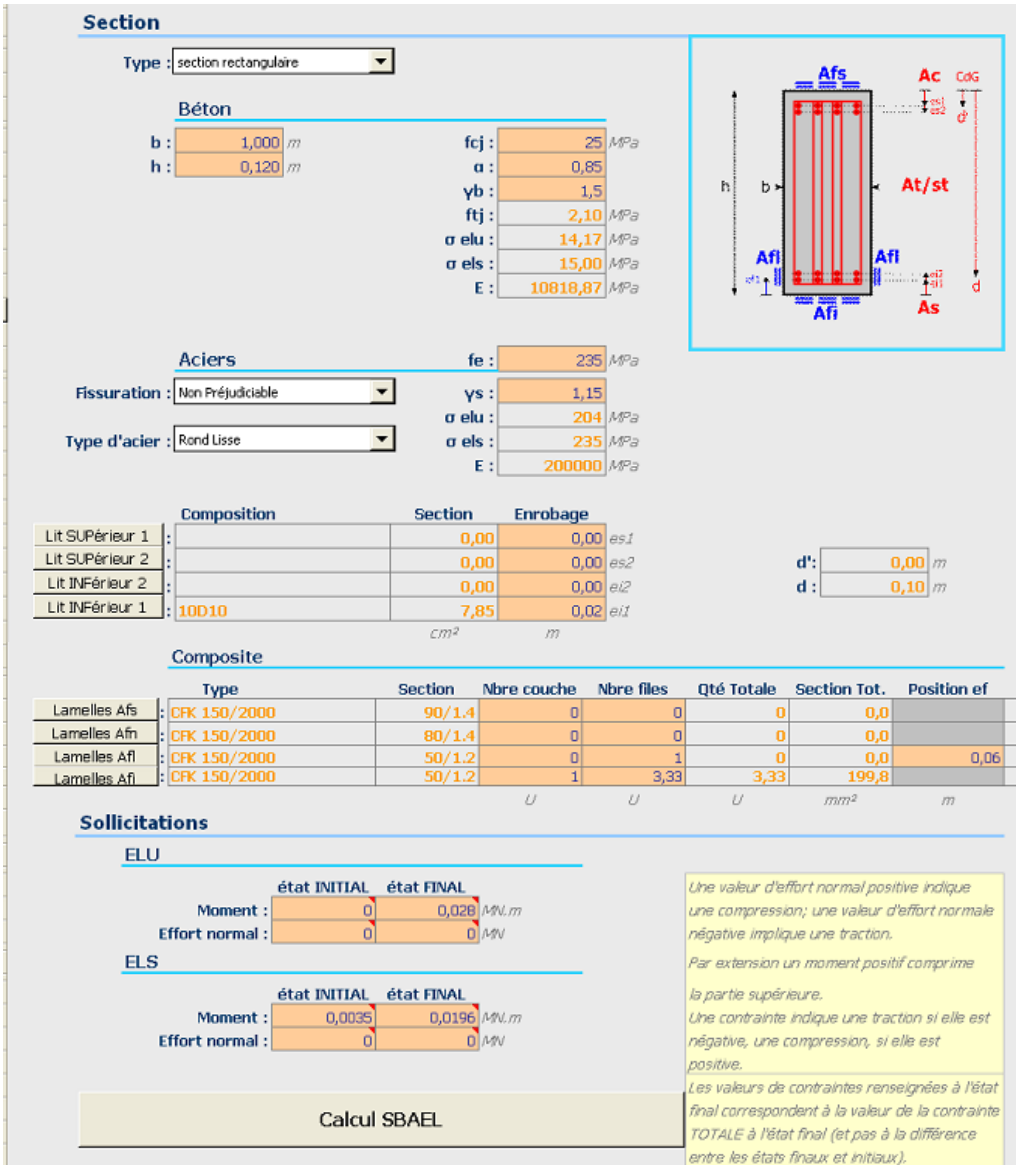


Figure 2.14: V3C information about the section, chosen by the user, in the "Bending" sheet

Here are the results obtained after having clicked on the macroinstruction “Calcul SBAEL”:

Calcul SBAEL

Les valeurs de contraintes renseignées à l'état final correspondent à la valeur de la contrainte TOTALE à l'état final (et pas à la différence entre les états finaux et initiaux).

Vérifications OK

Contraintes ELU

	état INITIAL	état FINAL	LIMITES	
Béton supérieur :	0,00	13,60	14,17	MPa
Béton inférieur:	0,00		14,17	MPa
Aciers Supérieurs :				MPa
Aciers Inférieurs :	0,00	-204,35	204,35	MPa
V3C Afs :				MPa
V3C Afl :				MPa
V3C Afi :		-649,11	1148,00	MPa

Déformations ELU (en pour 1000)

état INITIAL	état FINAL	valeur limite:
0,00	1,60	3,5
0,00		3,5
0,00	-2,99	10,0
	-3,93	7,0

Contraintes ELS

	état INITIAL	état FINAL	LIMITES	
Béton supérieur :	1,97	9,92	15,00	MPa
Béton inférieur :			15,00	MPa
Aciers Supérieurs :				MPa
Aciers Inférieurs :	-51,70	-229,34	235,00	MPa
V3C Afs :				MPa
V3C Afl :				MPa
V3C Afi :		-201,73	400,00	MPa

Déformations ELS (en pour 1000)

état INITIAL	état FINAL	valeur limite:
0,18	0,92	1,4
		1,4
-0,26	-1,15	1,2
	-1,22	2,4

Figure 2.15: V3C results from the SBAEL calculation in the “Bending” sheet

Verifications are “OK” if the calculated values are under the limit values. If the limit values are overstepped, a red “NO” appears instead of the green “OK”. In that case, the carbon fiber section needs to be increased and the program should be run again.

The verification of the slip stress is also done in this “Bending” tab, see also section 2.2.5:

Glissement ELU

ftk : 2,00 MPa

	Longueur	σ elu	Section	Nbre Files	Larg. Fibre	Contrainte	Limite
V3C Afs :	0	0,00	0	0	90	0,00	1,33
V3C Afl :	0	0,00	0	1	50	0,00	1,33
V3C Afi :	5,91	649,11	199,8	3,33	50	0,53	1,33
	m	MPa	mm²	U	mm	MPa	MPa

Figure 2.16: V3C verification of the slip stress in the “Bending” sheet

<u>“Anchorage” sheet:</u> enables to check that the force in the carbon composite in the anchorage zone (position X_E) is lower than the limit force F_{bd} (called “Effort Adm.” in the program), see also section 2.2.6.

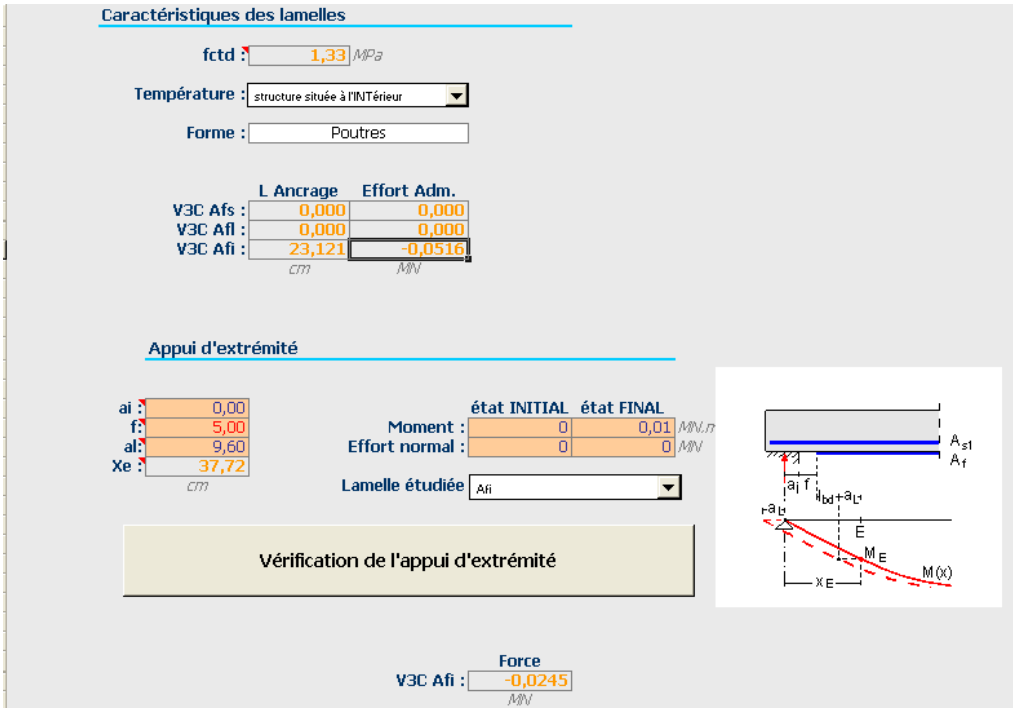


Figure 2.17: "Anchorage" sheet of the V3C

<u>"Shear force" sheet:</u> is used to check the resistance of a section to shear forces. I also made an Excel program to check these results, but this is not the point in this report since I focussed on the strengthening to bending moment.

2.4.3 Corrections of the program

After I have done my own Excel program, I could compare my results with those of the V3C program. Most of the results were similar, and for those that were different, the mistake was often quite easy to find just by looking to the formulas in the cells a little more carefully.

Most of the mistakes were not calculation mistakes but only display problems, like a wrong color coming up with the results, wrong limit values, wrong characteristics for the materials…

A sample of the mistakes I found in the program is given below:

<u>Flexion sheet:</u>

- The limit strain for concrete was set to *2‰* instead of *3,5‰*. Once this was corrected, there was still a problem: the value 2,5‰ was not taken into account for the verification. Indeed when the final strain in concrete was

between 2‰ and 3,5‰, the program said there was a failure in the concrete while it should have been OK.

- The numerical value for the neutral axis in the V3C program was very different from the one of the program developed in this work. There was apparently a mistake in the “SBAEL” program code.
- In the SLS, when the limit stresses were exceeded in steel or carbon fiber, there was no warning from the program: it said that the section was OK. The mistake was caused by an error of sign: the calculated stress was negative (traction) while the limit value was given with a positive sign (for example f_e for steel). Therefore the calculated stress was always lower than the limit stress, so the section was always OK concerning steel and carbon fiber composite.
- Problems of display: some cells were visible while they should have been hidden, some were green instead of red, some had the wrong text.
- Some mistakes in the carbon fiber library of the program: when the user chooses a type of carbon fiber product, it is related to a library containing all the necessary datas about the composite: the modulus of elasticity, the section, the limit elongation at the ULS/SLS, the limit stress at the ULS/SLS. Some of these values were wrong.

The other sheets had the same kinds of mistakes. They were relatively easy to correct, but often quite difficult to detect.

All the mistakes that have been notified during this work have been corrected by Bouygues TP. I cannot be sure that I saw all the mistakes and that all the results will be correct now, but I tried to test as many examples as I could, and at least for those ones the program gives correct results.

3. Design of new jacking consoles

3.1 Definition of “jacking”

3.1.1 Principle

“Jacking” consists in lifting up or lowering a whole structure or a part of it by means of hydraulic jacks. The reason can be to lift up a bridge deck in order to change its bearing device, or to lower a bridge deck that have been built above its final position and that needs to be laid on the bearings. Jacking can also be used on a building, when fundations have been affected by settlements. In this case, jacking the building can be a way to reposition it.

Jacking can be temporary or permanent, depending if the jacks are removed at the end of the operation or not.

Figure 3.1: jacking of a building: the middle part of the building has been jacked up to reposition it in a horizontal level

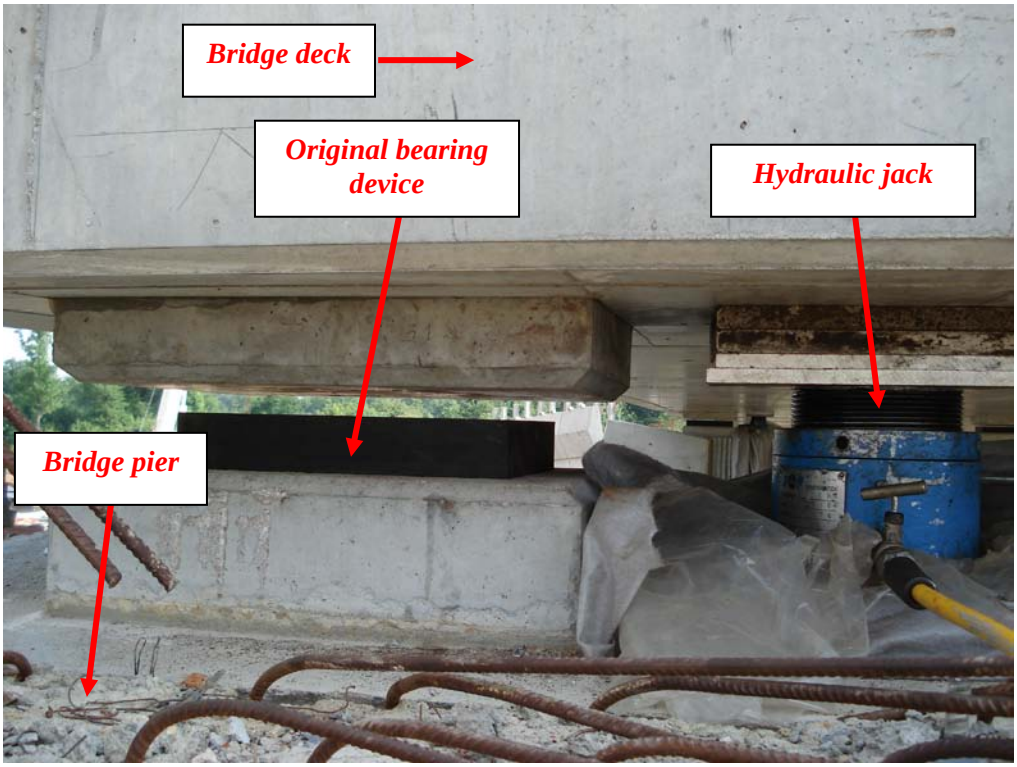


Figure 3.2: jacking used for renewal of bearing device

3.1.2 Purpose of jacking consoles

On the picture above, the jack lies on the top of the bridge pier, close to the bearing device. This is possible when there is enough space for it. There is no problem for new bridges: almost all of them are deseigned with this space. When it comes to older bridges, most of the time there is no space for a jack.

In this case, jacking consoles are needed: concrete or metallic consoles are fixed on the sides of the bridge piers, and jacks are put on top of them and can lift the bridge deck.

These are temporary consoles: they will be set in place for the jacking and the bridge pier will be restored after the jacking operation.

3.1.3 Console types

The following console is used by VSL France:

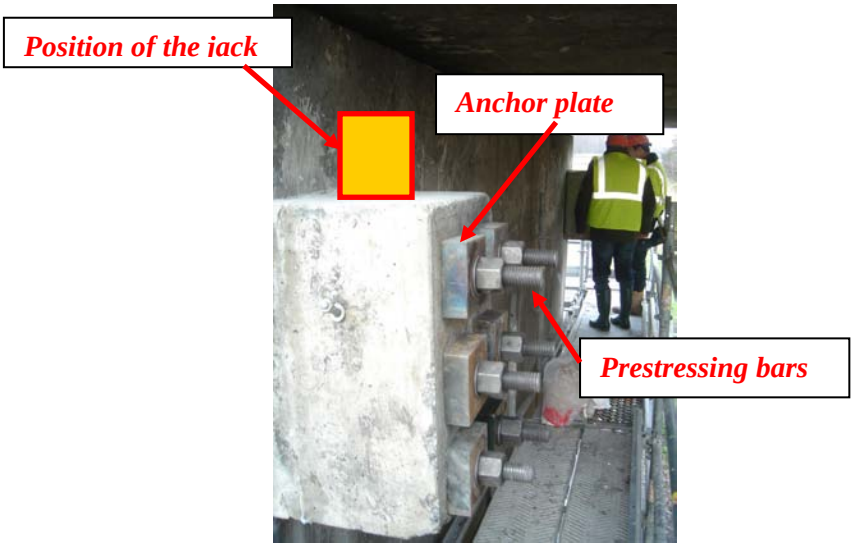


Figure 3.3: reinforced concrete console

These reinforced concrete consoles are fixed to the pier by prestressing bars. Then the friction coefficient between the console and the pier enables the console to take a certain load coming from the jack (cf. section 3.2.2 below).

Figures 3.4 and *3.5* below show the typical VSL console and the way it is fixed to the bridge pier.

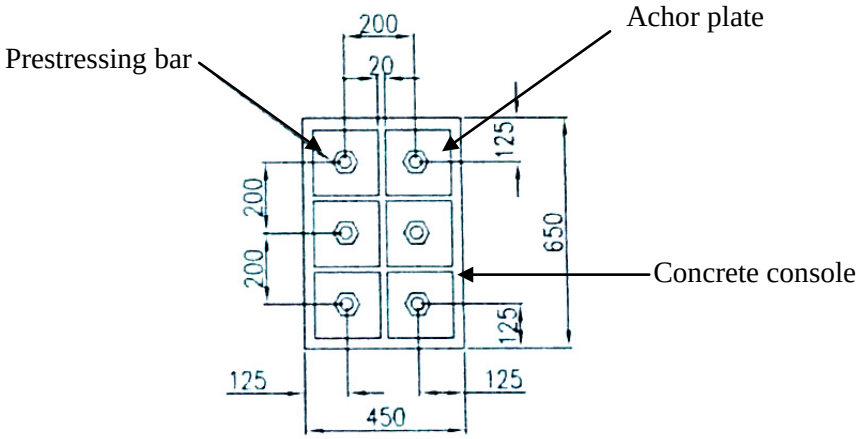


Figure 3.4: VSL console (front elevation)

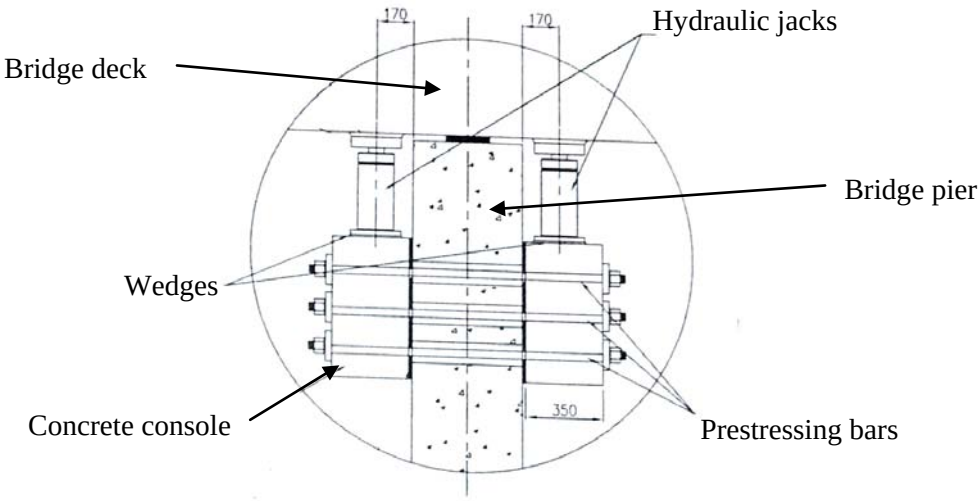


Figure 3.5: VSL console fixed on a pier (cross section)

Similar consoles in steel can be found too:

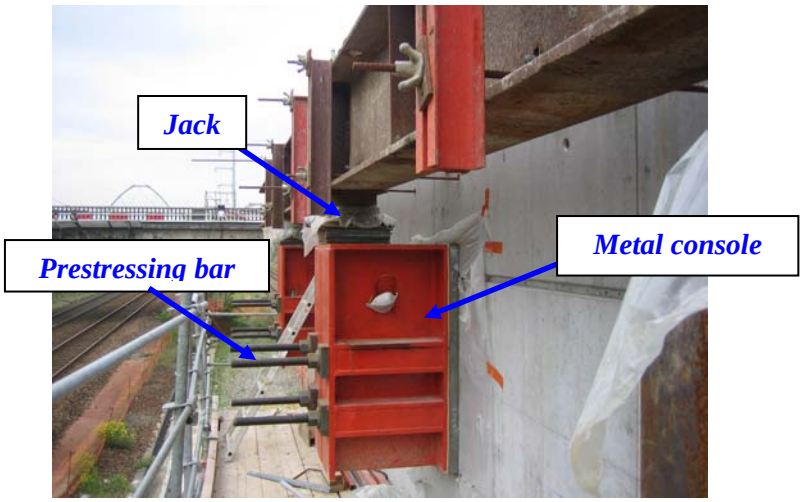


Figure 3.6: metal console

3.2 Fixing of the consoles

Two possibilities exist to fix the console to the bridge pier: either the load is transferred to the structure through friction forces between the console and the pier due to the prestressing bars that press the console against the pier, or the console works in tension-shear force. In this case, the bars are not prestressed but they must have a sufficient shear strength to resist to the vertical load coming from the jack, and a sufficient tensile strength to resist to the tensile loads caused by the excentricity of the jack in relation to the interface console-pier.
The VSL France consoles are concrete consoles that work with prestressing bars (*Figure 3.3*). In this work, the first important part was to study prestressing bars and losses, in order to be able to design new ones.

3.2.1 Prestressing bars

The prestressing bars used by VSL France are bars of either Macalloy, Dywidag or Annahütte type. All bar types have a certification on a European level: a *European Technical Approval* (*ETA*), delivered by the European Organisation for Technical Approvals (EOTA).

The properties of Macalloy bars are given in the *ETA 07-0046*. VSL France uses six kinds of threadbars, corresponding to six different diameters. They all have a characteristic tensile strength of 1030 MPa. They can be tensioned to 80% of the tensile strength maximum (i.e. 0,8×1030=824MPa). Here are some properties:

Diameter (mm)	25	26.5	32	36	40	50
Characteristic Tensile Load Capacity F_{pk} (kN)	506	568	828	1048	1294	2022
Maximum Prestressing force = $0,8F_{pk}$ (kN)	405	454	662	834 (838?)	1035	1618

Table 3.1: properties of Macalloy threadbars according to ETA 07-0046

The properties of Dywidag bars are given in the *ETA 05-0123*. VSL France uses threadbars with five different diameters. They all have a characteristic tensile strength of 1050 MPa and can be tensioned up to 80% of the characteristic tensile strength (=840 MPa). It is even possible to have an overtension going up to 95% of the characteristic 0.1% proof-stress of prestressing steel if the force in the prestressing jack is accurate enough:

Diameter (mm)	26 WR	32 WR	36 WR	40 WR	47 WR
Characteristic Tensile Load Capacity F_{pk} (kN)	580	845	1070	1320	1821
Maximum prestressing force = $0{,}8F_{pk}$ (kN)	464	676	856	1056	1457
Maximum overstressing force = $0{,}95F_{p0,1k}$ (kN)	499	722	912	1131	1566

Table 3.2: properties of Dywidag threadbars according to ETA 05-0123

The properties of Annahütte prestressing bars are the same as the Dywidag's. They are given in the *ETA 05-0122* which is almost exactly the same as the *ETA 05-0123* of Dywidag.

3.2.2 Load transfer

The normal load that is needed in the prestressing bars in order to press the console against the pier and to resist to the load from the jack is given in the standard *NF P 95-104* called: "Repair and strengthening of concrete and masonry structures".
The section 6.5 of this standard gives the formula with the relation between the horizontal prestressing force N_p that is needed to resist to the vertical load from the jack P_m, when there is a friction coefficient φ between the console surface and the pier surface.

At the ULS:

$$\gamma_P \times N_P \times \frac{\varphi}{\gamma_\varphi} \geq 1{,}35 P_m$$

where $\gamma_P = 0{,}85$ is a safety factor due to the uncertainty about the prestressing force, and $\gamma_\varphi = 1{,}2$ is another safety factor due to the uncertainty about the friction coefficient φ.

According to this standard, the usual value for φ is *0.5* for a console made of concrete and with the application of a resin at the interface console/pier.

Thanks to this formula, if the vertical load P_m coming from the jack is known, then the normal force N_p that is needed in the prestressing bars can be deducted. In this way, the number of prestressing bars needed can be found, keeping in mind that there will be prestress losses:

(Total prestressing force on the console) – (prestress losses) > N_p

It is then very important to estimate prestress losses accurately. In the following section, prestress losses for post-tensioning will be detailed, on the one hand according to the French standard called *BPEL*, and on the other hand according to the European standard *Eurocode 2*. The company had an Excel file that was done to

calculate these losses according to the formulas in the *BPEL*, and one of my tasks was to update this file in order to calculate the losses according to *Eurocode 2*. This is the reason why a detailed comparison of the two standards will be done in the following section.

3.2.3 Prestressing losses for post-tensioning

Immediate losses of prestress for post-tensioning to take into account are:

- Losses due to friction, with parameters that can be found in the *ETA* of each kind of bar;
- Losses at anchorage, due to a wedge draw-in and to a deformation of the anchorage itself;
- Losses due to the instantaneous deformation of concrete.

Time-dependent losses of prestress for post-tensioning due to:

- the creep of concrete;
- the shrinkage of concrete;
- the relaxation of steel.

VSL France had an Excel file that could help to calculate prestressing losses according to the *Règles BPEL 91 modifiées 99* which are the last French rules about prestressed concrete before the *Eurocode*.

This file has been updated in order to have the losses according to the *Eurocode 2* and not the *BPEL* any more.
In the following subsections *a.* to *g.*,a comparison between the losses according to *BPEL* and the losses according to the *Eurocode 2* is carried out.

a. Losses due to friction

<u>BPEL, Article 3.3,11 :</u>

For a bar with a length L between anchorages and with a prestressing stress σ_{p0} at one extremity, the stress at the other extremity is:

$$\sigma_{p0}(L) = \sigma_{p0} e^{-f\alpha - \varphi L}$$

Which means that the losses due to friction are:

$$\Delta\sigma_{p0} = \sigma_{p0}\left(1 - e^{-f\alpha - \varphi L}\right)$$

where:

- f = coefficient of friction between the tendon and its duct;
- α = sum of the angular displacements over the distance L;

- φ = coefficient of tension losses per unit length.

f and φ were given in the old brochures from the bar suppliers, before the publication of the *ETA*. α is supposed to be equal to zero for a straight bar.

Eurocode 2, Article 5.10.5.2
Losses due to friction are now:

$$\Delta\sigma_{p0} = \sigma_{p0}\left(1 - e^{-\mu(\theta + kL)}\right)$$

where:
- μ = coefficient of friction between the tendon and its duct (= f of the BPEL);
- θ = sum of the angular displacements over the distance L (= α du BPEL) ;
- k = unintentional angular displacement for internal tendons, per unit length ($\mu k = \varphi$ of BPEL).

The numerical values for μ and k are given in the *ETA*, and θ is equal to zero.

The two formulas, from *BPEL* and *Eurocode 2*, are therefore identical.

b. Losses at anchorage

When the load is transferred from the jack to the anchorage, there is a displacement of the wedges, called draw-in slip. This draw-in slip is written g and is given in mm in the *ETA*. In order to reduce the losses due to the draw-in slip of the anchorage, it is advisable to repeat the tensioning at least twice. In this way, most of the draw-in happens during the first tensioning, and only a small draw-in happens during the second tensioning.

The draw-in slip implies a reduction of the bar length, and therefore a loss of tension stresses in this bar.

Draw-in value: g (mm)
Deformation of the bar: g / L (‰, with L in meters)
Tension losses in the bar:

$$\Delta\sigma_g = \frac{g}{L} \times E_p$$

with E_p the modulus of elasticity of the prestressing bar in GPa. In both the *BPEL* and *Eurocode 2*, it is explained that the g value is given in the technical approval of the bar provider.

c. Losses due to the instantaneous deformation of concrete

BPEL, Article 3.3,13 :

$$\Delta\sigma_e = E_p \times \frac{k\Delta\sigma_{bj}}{E_{ij}}$$

where:

- $\Delta\sigma_{bj}$ = stress variation at the center of gravity of the prestressing bars;
- $k = (n-1) / 2n$ where n is the number of prestressing bars;
- E_{ij} = modulus of instantaneous longitudinal deformation of concrete (Article 2.1,42 of the *BPEL*).

Eurocode 2, Article 5.10.5.1 :

$$\Delta\sigma_e = \frac{\Delta P_{el}}{A_p} = E_p \times \frac{j\Delta\sigma_c}{E_{cm}}$$

where:

- $j = k$ of the *BPEL*;
- $\Delta\sigma_c = \Delta\sigma_{bj}$ of the *BPEL*;
- E_{cm} = modulus of elasticity of concrete (given in table 3.1 of the article 3.1.2 of the *Eurocode 2*).

The formulas are identical in theory, but the value given to the modulus of elasticity of concrete is slightly different.

d. Losses due to the relaxation of steel:

BPEL, Annex 2, Article 3 :

For a given value of the relaxation loss (in %) at 1000 hours after tensioning at a temperature of 20°C, written ρ_{1000}, the BPEL has a formula giving the losses due to relaxation j days after the tensioning and at the distance x of the stressing anchorage:

$$\Delta\sigma_\rho(x, j) = 0{,}008 \times \rho_{1000} \left(\frac{j \times 24}{1000}\right)^{0{,}75(1-\mu)} \times e^{\frac{10\mu - 7{,}5}{1{,}25}} \times \sigma_{pi}(x)$$

where:

- $\mu = \sigma_{pi}(x) / f_{prg}$ where $\sigma_{pi}(x)$ is the initial tension in the bar, after deduction of immediate losses, and f_{prg} is the characteristic tensile strength of the bar $(=f_{pk})$;
- $\rho_{1000} = 3\ \%$ according to the *ETA*;

- *j = 500 000 / 24 = 20833 days* is the value that should be taken for *j* if we consider long-term losses.

Eurocode 2, Article 3.3.2 :

Hot rolled bars are part of Class 3 according to this article of the Eurocode. *j* days after the tensioning, losses due to the relaxation are:

$$\Delta\sigma_{pr}(x, j) = 1{,}98 \times \rho_{1000} \times e^{8\mu}\left(\frac{j \times 24}{1000}\right)^{0{,}75(1-\mu)} \times 10^{-5} \times \sigma_{pi}(x)$$

where the parameters have the same meaning as in the *BPEL*'s formula.

The expression: $e^{\frac{10\mu - 7{,}5}{1{,}.25}}$ in the *BPEL* can be written: $e^{8\mu} \times e^{-6}$, and considering that: $0{,}008 \times e^{-6} = 1{,}98 \times 10^{-5}$, we can point out that the two formulas are in fact the same.

e. Losses due to the creep of concrete

BPEL, Article 3.3,22 :

This is a simplificated formula of the long-term tension losses due to the creep of concrete:

$$\Delta\sigma_{fl} = 2{,}5\sigma_b \frac{E_p}{E_{ij}}$$

where σ_b is the final stress in concrete, due to the permanent loads (including prestressing loads).

Eurocode 2, Article 3.1.4 :

The long-term deformation of concrete because of creep is:

$$\varepsilon_{cc}(\infty, t_0) = \varphi(\infty, t_0) \cdot \frac{\sigma_c}{E_c}$$

where:

- $\varphi(\infty,t_0)$ = final creep coefficient given by the Figure 3.1 of the Article 3.1.4 of *Eurocode 2*;
- t_0 = age of concrete at the time of the loading (in days).

In order to know the strain of the concrete *j* days after tensioning (not the long-term deformation), the complex formulas of the Annex B of the Eurocode 2 could be used, but to make it easier I used the *r(t)* function of the *BPEL* (Article 2.1,51) which is a

way to estimate time-dependent losses *t* days after the tensioning if the long-term losses are known (Article 3.3,24 of *BPEL*).

Thus, the strain in concrete, *j* days after tensioning, is:

$$\varepsilon_{cc}(j,t_0) = r(j)\cdot\varepsilon_{cc}(\infty,t_0)$$

With: $r(j) = j / (j+9r_m)$, where $r_m = B/u$ with B = *section of the element* and u = *external perimeter of the element*.

Losses due to creep *j* days after tensioning are then:

$$\Delta\sigma_{cc}(j) = \varepsilon_{cc}(j,t_0)\cdot E_p = r(j)\cdot\varphi(\infty,t_0)\cdot\sigma_c\cdot\frac{E_p}{E_c}$$

f. Shrinkage

BPEL, Article 3.3,2 :

The long-term tension loss due to the shrinkage of concrete is:

$$\Delta\sigma_r = \varepsilon_r\left[1 - r(t_0)\right]E_p$$

Where:

- ε_r = final shrinkage of concrete, as given in the Article 2.1,51 of the *BPEL*:
  - ε_r = 0,15 ‰ in very wet regions;
  - ε_r = 0,2 ‰ in wet regions,like France, except the South-East region;
  - ε_r = 0,3 ‰ in temperate-dry regions, like in the South-East France;
  - ε_r = 0,4 ‰ in warm and dry regions;
  - ε_r = 0,5 ‰ in very dry or desertical regions.
- t_0 = age of concrete at the time of tensioning;
- $r(t_0)$ = function giving the evolution ot shrinkage with time, as explained in the Article 2.1,51 of the *BPEL*: $r(t_0) = t_0 / (t_0 + 9r_m)$,

Eurocode 2, Article 3.1.4 :

The total strain due to the shrinkage at the time *t* is:

$$\varepsilon_{cs}(t) = \varepsilon_{cd}(t) + \varepsilon_{ca}(t)$$

where:

- $\varepsilon_{cs}(t)$ = total shrinkage deformation;
- $\varepsilon_{cd}(t)$ = drying shrinkage deformation;
- $\varepsilon_{ca}(t)$ = autogenous shrinkage deformation;
- t = age of concrete, in days.

Formulas giving $\varepsilon_{cd}(t)$ and $\varepsilon_{ca}(t)$ are in the Article 3.1.4 of the *Eurocode 2*.

Thus the tension losses due to shrinkage *j* days after tensioning (tensioning at the time t_0) are:

$$\Delta\sigma_{cs}(j) = (\varepsilon_{cs}(t) - \varepsilon_{cs}(t_0)) \cdot E_p$$

Note: relation between parameters: $t = t_0 + j$

g. Total of time-dependent losses

BPEL, Article 3.3,2 :

Total value of the long-term losses (time-dependent losses):

$$\Delta\sigma_d = \Delta\sigma_r + \Delta\sigma_{fl} + \frac{5}{6}\Delta\sigma_\rho$$

The coefficient 5/6 just before the losses due to the relaxation of steel bars is explained by the diminution of these relaxation losses with the creep and shrinkage of concrete.
Total value of time-dependent losses, *j* days after tensioning:

$$\Delta\sigma_{dj} = r(j) \cdot \Delta\sigma_d$$

where *r(j)* is once more the function of evolution given in the Article 2.1,51 of the *BPEL*.

Eurocode 2, Article 5.10.6 :

Total value of time-dependent losses, *j* days after tensioning:

$$\Delta\sigma_{p,c+s+r}(j) = \frac{\Delta\sigma_{cs}(j) + 0{,}8 \cdot \Delta\sigma_{pr}(L, j) + \Delta\sigma_{cc}(j)}{1 + \frac{E_p}{E_{cm}} \frac{A_p}{A_c}\left(1 + \frac{A_c}{I_c} z_{cp}^2\right)[1 + 0{,}8\varphi(t, t_0)]}$$

were:

- I_c = moment of inertia of the concrete section;
- z_{cp} = distance between the center of gravity of the concrete section and the prestressing bars.

All the other parameters have already been defined.

The numerator of this formula is similar to the formula in the *BPEL* (with 0,8 ≈ 5/6).
The main compared to the *BPEL* is the presence of the denominator which is larger than 1 and therefore reduces the total value of the time-dependent losses.

3.3 Design of new consoles

3.3.1 Choice of the material

The present consoles are made of reinforced concrete and are 65 cm high, 45 cm wide and 35 cm thick. If we consider that the density of concrete is 2500 kg/m^3, this means that these consoles weigh 256 kg. Handling these consoles are therefore not very easy, a lifting device is necessary.

One of my tasks during my internship was to design new jacking consoles that are easier to handle, which means basically lighter and smaller.

The first idea was to think about steel consoles instead of concrete consoles. Indeed steel has higher strength than concrete so the amount of material could be considerably reduced. However two objections convinced us that steel would probably not be better than concrete.

The first objection concerned the friction coefficient φ between the console and the pier, mentioned in section 3.2.2. Indeed according to the standard *NF P 95-104*, the value given to φ for a concrete console would be *0.5* while it would be only *0.37* for a metal console.
This means that to resist to the same vertical load, if 4 prestressing bars are sufficient for a concrete console, then 6 bars would be necessary if the console was made of metal. This implies more work and construction costs.

The second objection concerns the high density of steel compared to the concrete's (7850 kg/m^3 for steel, 2500 kg/m^3 for concrete). We quickly realised that steel plates and stiffeners had to be thick enough in order to resist to the large loads and large enough to avoid risks for compressive failure at the bearing surfaces.
Finally, considering all these elements, it was obvious that a metal console would not be much better than a concrete console.

For these reasons, a reinforced concrete design was decided for the consoles.

3.3.2 Geometry of consoles

As it has already been mentioned before, VSL France uses bars of the type Macalloy, Dywidag or Annahütte (the two last ones have the same characteristics). A diameter of 40 mm is common for these bars during jacking operations, that is why it was decided that the new consoles should be able to be used with bars of this diameter. I designed consoles for 6 bars, when heavy loads need to be jacked, but also consoles for 4 bars that are often sufficient at lower load levels.

The aim was to minimise the dimensions of consoles as much as possible. The problem is that they cannot be too small, because they need to offer space for 4 or 6

anchor plates, and the section of the console in contact with the pear should be large enough to avoid the risk of failure at bearing surfaces.

Standard Macalloy anchor plates are squares of 160 x 160 mm², while Dywidag plates are squares of 220 x 220 mm². The Dywidag plates are much larger than the Macalloy's, and they would need larger consoles than the Macalloy's. In order not to have such large consoles, it was decided that the bar diameter would be limited to 36 mm for Dywidag (and still 40 mm for Macalloy). For a bar of 36 mm diameter, the Dywidag anchor plate is a square of 200 x 200 mm².

That is why the geometry of consoles is:

- 4 bars console: concrete consoles with two rows of two bars, 450 mm wide, 450 mm high, and 300 mm thick;
- 6 bars consoles: concrete consoles with three rows of two bars, 450 mm wide, 660 mm high and 300 mm thick.

Moreover the console is protected by a steel shell, on the lateral surfaces (cf. *Figures 3.7* and *3.8*). This shell is 20 mm thick on the upper and lower faces (in order to spread the load from the jacks into the concrete of the console), and 5 mm thick on the side faces (where hooks will be welded for lifting the console).

Figure 3.7 below shows a general view of the 4-bars console with Dywidag anchor plates (bars 36 mm maximum):

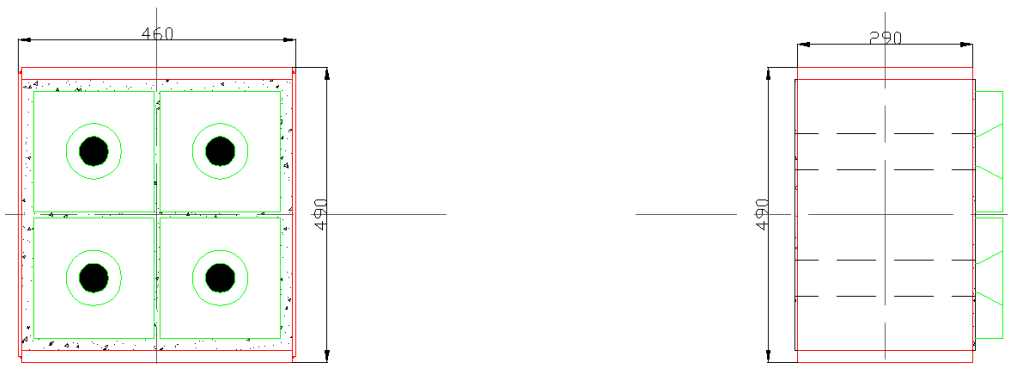


Figure 3.7: 4-bars console with Dywidag anchor plates

General view of the 6-bars console with Dywidag anchor plates (bars 36 mm maximum):

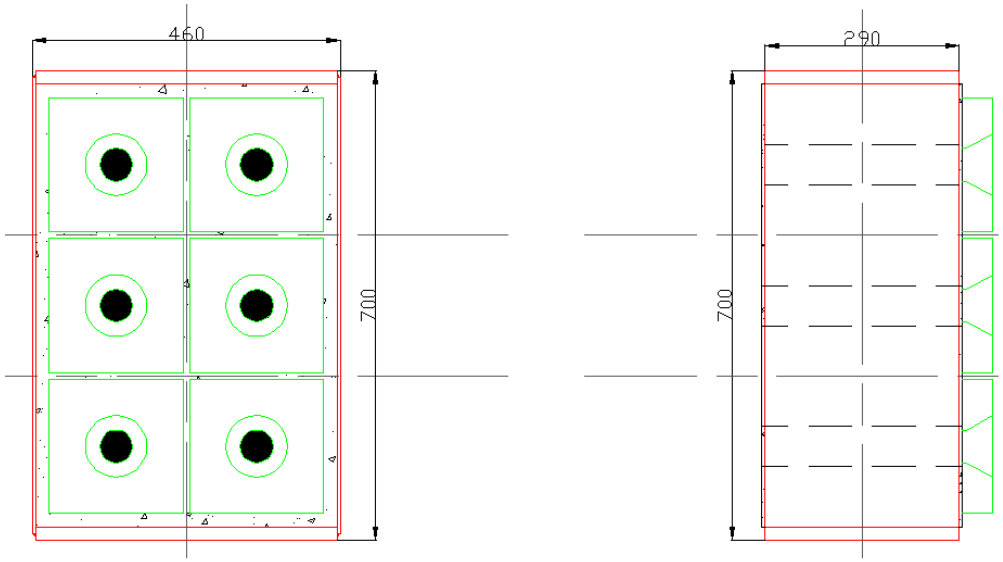


Figure 3.8: 6-bars console with Dywidag anchor plates

3.4 Design of reinforcement bars

The amount and disposition of reinforcement bars that are needed are designed according to the Annex 4 of the *BPEL*.

All the details of the calculations are not given in this paper, since the company would like to keep them confidential. However, all the formulas and principles of the calculation that can be found in the *BPEL* are given below.

Figure 3.9 below is a plan section of a console that would be fixed against the pier thanks to a prestressing force F_0. The picture shows roughly the stress distribution in this section, which also indicates where reinforcement bars are needed.

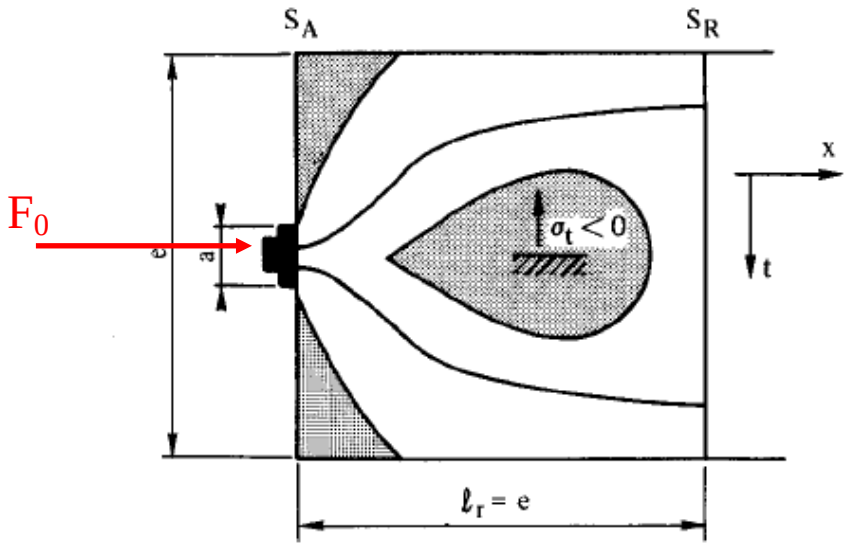


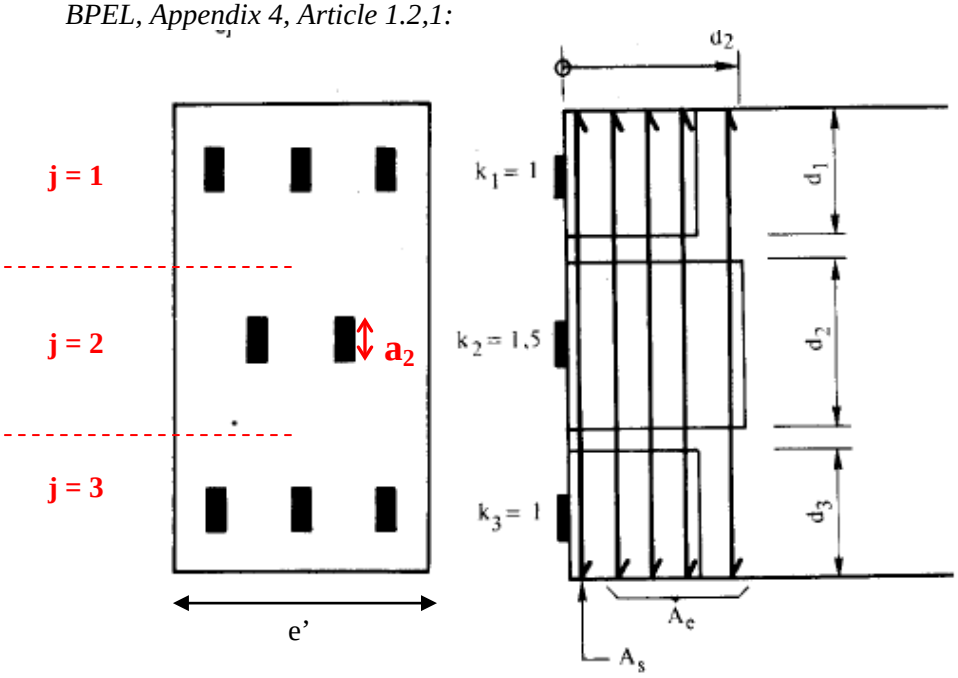
Figure 3.9: stress distribution in a prestressed section
(picture from BPEL*)*

The area between S_A and S_R is called "regularization zone". In this zone, areas with compression stresses are in white while areas with tension stresses are in grey. Two different areas with tension stresses are shown on this picture, which means that two different kinds of steel reinforcement bars will be necessary: the first ones, called "surface reinforcement bars", are necessary just behind the section S_A where the prestressing force is applied. The other reinforcement bars, called "split reinforcement bars", are necessary inside the section in order to resist to the nucleus of tension stresses that tend to make the concrete crack.

Then, other reinforcement bars are necessary in order to satisfy the general equilibrium of the structure: minimum section of reinforcement bars to resist to the normal force, and minimum section to resist to the shear force.

3.4.1 Surface reinforcement bars: A_s

BPEL, Appendix 4, Article 1.2,1:



max. $(d_j) = d_2$

Figure 3.10: model for console design
(picture from BPEL*)*

The minimum section of surface reinforcement bars needed is:

$$A_s = 0,04 \times \frac{\max\left(F_{j0}\right)}{\sigma_{s\lim}}$$

where:

- j = level of prestressing bars (see *Figure 3.10* above);
- F_{j0} = total prestressing force at level j;
- $\sigma_{slim} = 2/3\ f_e$ = design tensile strength for steel.

3.4.2 Split reinforcement bars: A_e

BPEL, Appendix 4, Article 1.2,2:

Maximum split stress in the prism corresponding to the anchorage level j:

$$\sigma_{tej} = 0,5\left(1 - \frac{a_j}{d_j}\right)\frac{F_{j0}}{e'd_j}$$

where:

- a_j = dimension of the anchorage plate (cf. *Figure 3.10*);
- d_j = dimension of the symmetric prism (cf. *Figure 3.10*);
- *e'* = dimension of the console, perpendicular to the study plan (cf. *Figure 3.10*);

Average compressive stress:

$$\sigma_{xmj} = \frac{F_{j0}}{e'd_j}$$

These two following relations must be verified before going further:

$$\sigma_{tej} \leq 1,25\ f_{tj} \quad \text{and} \quad \sigma_{xmj} \leq \frac{2}{3}\ f_{cj}$$

If one of these two conditions is not satisfied, the section of concrete needs to be increased or the prestressing force has to be smaller. Otherwise there is a risk of splitting the concrete, even with split reinforcement bars.

If these conditions are satisfied, then it is possible to calculate the amount of split reinforcement bars that is needed:

Resultant force of split stresses at level *j*:

$$R_j = 0,25\left(1 - \frac{a_j}{d_j}\right)F_{j0}$$

For each level *j* is calculated a section A_{ej} of split reinforcement bars:

$$A_{ej} = \frac{R_j}{k_j \sigma_{s\lim}}$$

where:

- $k_j = 1$ if *j* is a "side level";
- $k_j = 1,5$ if *j* is an "intern level", which means that in case of failure in concrete at this level, the levels around can still help to the resistance (cf. *Figure 3.10*).

The section A_e that is necessary in the section is:

$$A_e = \sup \begin{cases} \max\left(A_{ej}\right) \\ 0{,}15 \dfrac{\max\left(F_{j0}\right)}{\sigma_{s\lim}} \end{cases}$$

3.4.3 Longitudinal reinforcement bars: A_l

Once the design of reinforcement bars in the regularization zone has been done, the consoles need traditional longitudinal and transversal reinforcement bars to resist to compression and shear forces.

The Article A.8.1,2 of the *BAEL*, which is the French standard concerning reinforced concrete, recommends a minimal section of longitudinal reinforcement bars for a compressed element:

$$A_l \geq 4 \times p \qquad \text{cm}^2$$

where p is the length of the edge of element which is perpendicular to these reinforcement bars.

3.4.4 Transversal reinforcement bars: A_t

The Article A.5.1,22 of the *BAEL* also recommends a minimal section of transversal reinforcement bars (stirrups):

$$\frac{A_t}{s_t} \geq 0{,}4 \times \frac{b_0}{f_e}$$

where:

- s_t = distance between two consecutive stirrups (m)
- b_0 = width of the element – console in our case (m)
- f_e = yield limit for steel (MPa)

The AutoCAD drawings below illustrate the results of these calculations. By using the formula of section 3.2.2, it is also possible to calculate the maximum load that a console can take: around 100 tonnes for 4 bars consoles, and 150 tonnes for 6 bars consoles.

It is important to note that if consoles are cast in-situ and not precast, then the friction coefficient is better, so the maximum load that can be taken by the jacks is larger. However it costs more money and take more time to cast in-situ instead of having precast consoles.

3.4.5 Disposition of reinforcement bars in the console

Reinforcement bars in the 4-bars console:

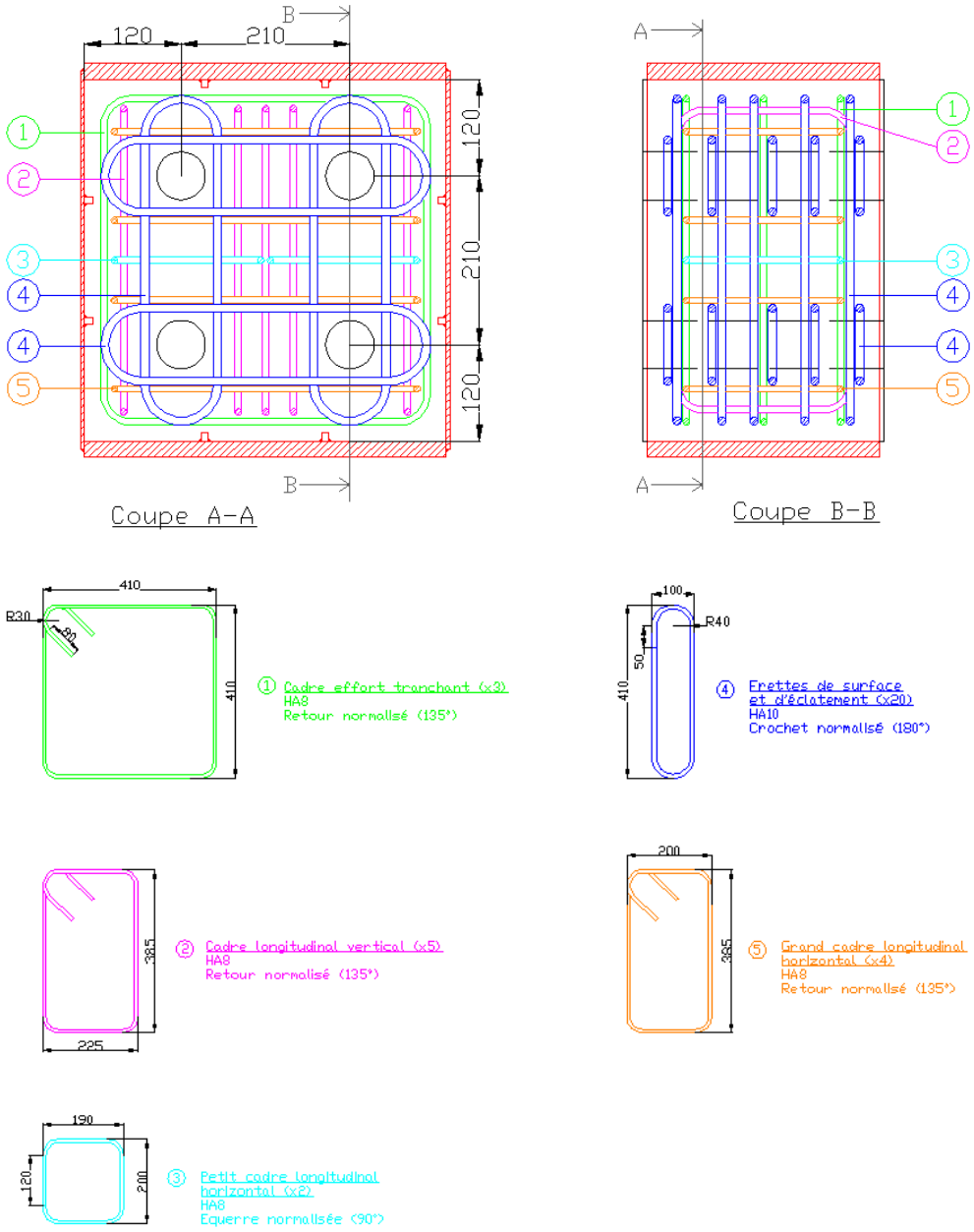


Figure 3.11: reinforcement bars in the 4-bars console

Reinforcement bars in the 6-bars console:

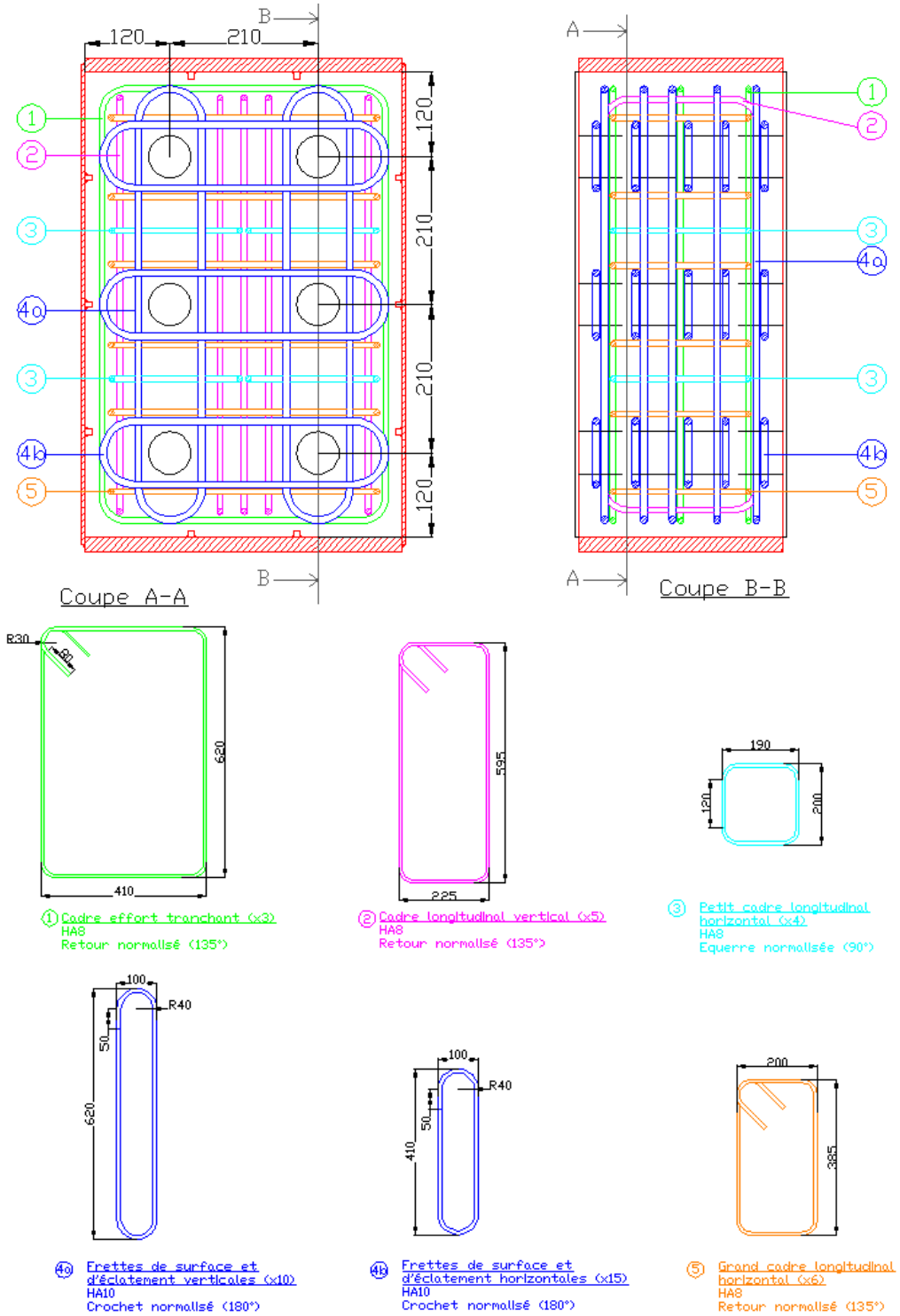


Figure 3.12: reinforcement bars in the 6-bars console

4. General conclusion

As it was developped in the first part of this work, carbon fiber strengthening is a very interesting solution to reinforce old structures: it is easy to apply, it is durable (no risk of corrosion) and it has high mechanical characteristics. It is now a relatively common solution to strengthening and it is important for a company to have a trustful tool to calculate the amount of carbon fiber needed.
The V3C program I worked on is one of these tools, designing carbon fiber composites according to the French standard *BAEL*. This kinf of program must probably exist in the other companies using carbon fiber strengthening, but every company tends to keep it in an internal area.
The final version of the V3C, after I worked on it, has satisfied the tests I run on it. I tried to cover as many situations as I could but there are certainly some particular cases that I did not think of, that is why the user should still be careful with the results if he uses the program in a situation lightly different than common ones. But this kind of programs need time and experience in order to be totally correct.

Concerning jacking consoles, the discussion is still open. The new consoles I designed are a bit smaller and probably more durable than the old ones, but it did not offer the improvement that we were expecting at the beginning. They are not much lighter, they still need a lifting device to be fixed on the pier. The question is then to wonder if it would not be better to cast the console in place, instead of having precast consoles. Then the friction coefficient would be better, so we would need maybe less prestressing bars. On the other hand this means that we cannot use the consoles again, more time is needed for casting and drying of concrete, so it costs more money.
The reflexion between precast consoles or cast in place consoles has to be thought for each project. Both have advantages and disadvantages.
Another direction could be to study the solution of a lighter metal, like aluminium. However the knowledge about aluminium structures, concerning their design and their durability, was not sufficient for us to cary out a study with a result for the company during the short time I had, and we prefered to focus on reinforced concrete consoles than aluminium consoles.

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Appendix 1: Avis Technique 3/07-524 - V2C®

Avis Technique 3/07-524

Annule et remplace l'Avis Technique 3/05-437

Éléments de structure renforcés par un procédé de collage de fibres carbone

V2C®

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Commission chargée de formuler des Avis Techniques
(arrêté du 2 décembre 1969)

Groupe Spécialisé n° 3
Structures, planchers et autres composants structuraux

Vu pour enregistrement le 21 janvier 2008

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le futur en construction

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Appendix 1: Avis Technique 3/07-524 – V2C®, page 2/11

Le Groupe Spécialisé n° 3 "STRUCTURES, PLANCHERS ET AUTRES COMPOSANTS STRUCTURAUX" de la Commission chargée de formuler les Avis Techniques, a examiné le 25 juin 2007 le dossier de demande de révision de l'Avis Technique 3/05-437 sur le procédé de renforcement d'éléments de structures par collage de tissus de fibres de carbone, dénommé V2C®, exploité par l'Entreprise VSL FRANCE SA. Il a formulé sur ce procédé l'Avis Technique ci-après qui révise l'Avis Technique 3/05-437.

1 – Définition succincte

Procédé de renforcement d'éléments de structure, consistant à coller sur la surface des éléments visés un tissu ou des lamelles de fibres de carbone à l'aide d'une résine époxydique synthétique à deux composants.

Ce procédé est destiné à augmenter la capacité portante des éléments concernés, par fonctionnement mécanique conjoint élément-tissu, grâce à l'adhérence conférée par la résine après son durcissement, entre les deux matériaux.

1.1 - Identification des composants

Les références commerciales des différents composants sont les suivantes:

- pour ce qui concerne les résines : SRS-S3D, SRS-S4D pour les tissus de carbone et SRS-P204 pour les lamelles;
- pour ce qui concerne les lamelles : S&P CFK 150/2000 ou S&P CFK 200/2000 ;
- pour ce qui concerne les tissus : C-Sheet S&P 240, C-Sheet S&P 640.

2 - L'AVIS

L'Avis qui est émis prend en compte le fait que la conception et le dimensionnement du renforcement sont effectués par ou sous la responsabilité de VSL France SA, l'exécution des travaux étant effectuée par VSL France SA.

2.1 - Domaine d'emploi accepté

Le domaine d'emploi accepté par le Groupe Spécialisé n°3 est celui couvrant les éléments en béton armé ou en béton précontraint entrant dans la constitution des bâtiments courants (habitations, bureaux, etc.) et des bâtiments industriels (supermarchés, entrepôts, etc.).

Les éléments concernés sont sollicités par des charges à caractère principalement statique, comme c'est le cas dans les bâtiments administratifs, commerciaux, scolaires, hospitaliers, d'habitation, de bureaux, parkings pour véhicules légers (30 kN de charge maximale à l'essieu).

L'Avis n'est valable pour les différents composites que si la température n'excède pas :

- 40°C en température de service continu* et 45° en température de pointe* pour la résine S3D
- 45°C en température de service continu* et 55°C en température de pointe* pour les résines S4D et P204
- 55°C en service continu* et 60°C en température de pointe* pour la résine MHT.

(*) : voir nota à la fin du paragraphe.

L'utilisation de ce procédé est limitée au renforcement des structures vis-à-vis des actions rapidement variables.

L'utilisation en bâtiments industriels est admise tant que l'agressivité chimique ambiante peut être considérée comme normale et que les charges non statiques ne sont pas de nature répétitive entretenue pouvant donner lieu à fatigue. On peut citer, à titre d'exemple de charges exclues, les machines tournantes et les passages intensifs et répétés de camions.

Les utilisations autres que celles prévues au présent domaine d'emploi, notamment les renforcements d'éléments constitués de matériaux autres que le béton (maçonnerie ou bois) sortent du champ du présent Avis.

Le CPTP (paragraphe 2.3 du présent Avis) précise les conditions dans lesquels le renforcement par le procédé V2C peut être envisagé.

L'Avis est émis pour les utilisations en France européenne.

L'utilisation en zone sismique et le cas des sollicitations susceptibles de changer de sens ne sont pas examinés dans le cadre du présent Avis Technique.

Nota :

Température de pointe : température maximale susceptible d'être présente sur le support et de façon non permanente.

Température de service : température usuellement constatée sur le support renforcé

2.2 - Appréciation sur le procédé

2.21 Aptitude à l'emploi

2.211 Stabilité

L'examen des performances du complexe tissu-résine, lamelle-résine, au travers des essais effectués par le demandeur, permet de conclure que le procédé conduit à l'augmentation des capacités résistantes des éléments renforcés, conformément aux modèles de calcul développés dans le Dossier Technique établi par le demandeur, à condition de respecter strictement les prescriptions données dans le CPTP du présent Avis.

2.212 Sécurité au feu

Réaction au feu :

Des essais effectués au laboratoire du CSTB (**PV n° RA05-0047**) ont fourni le classement de réaction au feu des complexes tissu-résine et lamelle-résine **avec une protection de CAFCO 300 de 40mm d'épaisseur**, sur support M0 non isolant, pour les 5 complexes suivantes :

- résine SRS-P204 et lamelles CFK 150/2000 (dimensions : 50 x 1,2mm)
- résine SRS-S3 et tissus en fibres de carbone C-Sheet 240 (4 couches)
- résine SRS-S3 et tissus en fibres de carbone C-Sheet 240 (1 couche)
- résine SRS-S4 et tissus en fibres de carbone C-Sheet 640 (4 couches)
- résine SRS-S4 et tissus en fibres de carbone C-Sheet 640 (1 couche)

Les proportions des produits mis en œuvre sont les suivants :

- résines : SRS P204 1,5kg/m² ; résine SRS-S3 0,3 à 0,4 kg/m² par couche ; résine SRS-S4 0,3 à 0,4 kg/m² par couche.
- Masses surfaciques des tissus C-Sheet 240 : 200g/m² et 300g/m² ; C-Sheet 640 : 400g/m².

Classement annoncé par le PV : **M1**, valable sur support M0 non isolant.

Résistance au feu :

En ce qui concerne la résistance au feu, le procédé V2C ne participe pas à la tenue des éléments renforcés. Lorsqu'une protection au feu est prévue par-dessus le composite, elle doit justifier d'un essai de résistance au feu effectué sur un support identique, par un laboratoire agréé par le Ministère de l'Intérieur. Une attention particulière doit être apportée au fait que les caractéristiques mécaniques de la colle diminuent rapidement lorsque la température augmente.

2.213 Prévention des accidents lors de la mise en œuvre ou de l'entretien

Pour la manipulation de la colle et son application, il y a lieu de respecter les prescriptions du Code du travail concernant les mesures de protection relatives à l'utilisation des produits contenant des solvants, utilisés pour le nettoyage des outils. En dehors de ce point, les conditions de mise en œuvre ne sont pas de nature à créer d'autre risque spécifique.

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2.22 Durabilité – Entretien.

La durabilité des éléments renforcés est normalement assurée, exception faite pour les utilisations en locaux (ou ambiances) suivants :

1. atmosphère agressive
2. lorsque la température est susceptible de dépasser régulièrement la température de service défini au §2.1 du présent Avis.

En effet, pour le premier cas, la stabilité des caractéristiques mécaniques de la colle n'est pas démontrée. Pour le second cas, il n'existe pas d'essai de fluage présenté par la société VSL mais le retour d'expérience de cette entreprise sur ces procédés de renforcement défini dans le §1.1 laisse présager d'un bon comportement.

Dans le cas où des dégradations (chocs, abrasion, etc.) sont possibles, une protection mécanique du renforcement est à prévoir.

2.23 Fabrication et contrôles.

Les éléments entrant dans la constitution du procédé sont fabriqués dans des usines spécialisées :

- Le tissu et les lamelles sont fabriqués par la Société suisse S&P Reinforcement,;
- La résine provient de la société Résipoly.
- La fabrication du tissu, ainsi que celle de la colle, font l'objet d'un plan d'assurance-qualité dans les usines concernées.

2.24 Finitions.

Lorsque des revêtements (notamment peintures) sont prévus sur le renforcement, ils doivent avoir fait l'objet d'essais préalables validant leur adhérence sur la matrice époxydique du V2C.

2.3 - Cahier des Prescriptions Techniques Particulières

2.32 Conditions de conception et de calcul.

2.321 Justification à la rupture.

Cette justification consiste en une vérification de l'élément à la rupture, toutes redistributions effectuées, sans tenir compte du renforcement, sous la combinaison ELS rare, considérée conventionnellement dans les calculs comme combinaison ELU fondamentale. Il sera tenu compte, s'il y a lieu, des charges climatiques et de celles dues aux instabilités.

Toutefois, cette justification n'est pas à effectuer si :

- **(R_1) ≥ 0,63 (S_2)**, dans le cas d'un élément principal, dont la rupture est susceptible d'entraîner celle d'autres éléments (poutre porteuse, par exemple),
- **(R_1) ≥ 0,50 (S_2)**, dans le cas d'un élément secondaire, dont la rupture n'est pas susceptible d'entraîner celle d'autres éléments (panneaux de dalles de planchers posés sur poutres, par exemple).

Avec, dans ces expressions :

R1 : capacité résistante à l'ELU de l'élément non renforcé.

S2 : sollicitation agissante à l'ELU sur l 'élément renforcé.

2.322 Renforcement vis-à-vis du moment de flexion

Les justifications à effectuer, vis-à-vis du moment de flexion, pour les éléments en béton renforcés par le procédé V2C sont les suivantes :

- Calcul **à l'ELS** : ce calcul est effectué selon les hypothèses classiques du béton armé, en tenant compte de l'historique du chargement et du renforcement (y compris un éventuel déchargement ou vérinage provisoire en cours de travaux). Ceci conduit à superposer les états de contraintes relatifs aux deux situations suivantes :
- ouvrage non renforcé, soumis aux sollicitations initiales, appliquées au moment où l'on entame les travaux de renforcement,
- ouvrage renforcé, soumis aux sollicitations additionnelles.

Cette justification est menée en prenant en compte un coefficient de sécurité de 2.15 sur la contrainte à rupture des lamelles V2C, respectivement 3 pour le renforcement par tissu V2C, et en limitant la contrainte finale dans les armatures tendues existantes aux valeurs suivantes :

- cas de la fissuration peu préjudiciable : f_e,
- cas de la fissuration préjudiciable : la limitation prévue à l'article A.4.5,33 des Règles BAEL91 ,
- cas de la fissuration très préjudiciable : la limitation prévue à l'article A.4.5,34 des Règles BAEL91.

La contrainte de compression dans le béton est limitée à 0,6 f_{ej}.

- Calcul **à l'ELU** : ce calcul est mené conformément aux détails donnés dans le dossier technique établi par le demandeur : en plus des hypothèses classiques sur le béton et l'acier, la déformation du V2C est limitée à :
  - 6,5 ‰ pour renforcement C240 le coefficient de sécurité adopté sur la contrainte à cet allongement est de 2
  - 1,6 ‰ pour renforcement C640 le coefficient de sécurité adopté sur la contrainte à cet allongement est de 1,6
  - 7 ‰ pour renforcement par lamelles S&P 150/2000, le coefficient de sécurité adopté sur la contrainte à cet allongement est de 2
  - 6,5 ‰ pour renforcement par lamelles S&P 200/2000, le coefficient de sécurité adopté sur la contrainte à cet allongement est de 1,8
- **Vérification du glissement à l'interface composite-béton :** cette vérification consiste à s'assurer que la contrainte de cisaillement due au moment de flexion, à l'interface composite-béton n'excède pas la valeur limite de la contrainte limite de cisaillement. Cette valeur limite s'appuie dans tous les cas par des essais de pastillage à effectuer in situ sur le support en préparation, dans l'état dans lequel il est destiné à recevoir le renforcement.

La valeur de la contrainte de cisaillement limite à retenir pour le dimensionnement est calculée de la manière suivante, à partir de la résistance caractéristique ftk obtenue par les essais de pastillage :

A l'ELS : $\bar{\tau} = \mathrm{Min}\left(1{,}5\,\mathrm{MPa} ; \frac{f_{tk}}{2}\right)$

A l'ELU (fondamental et accidentel) : $\bar{\tau}_u = \mathrm{Min}\left(2\,\mathrm{MPa} ; \frac{f_{tk}}{1{,}5}\right)$

2.323 Renforcement vis-à-vis de l'effort tranchant.

Le renforcement des dalles vis-à-vis de l'effort tranchant n'est pas visé dans le cadre du présent Avis Technique.

Utilisation de plaques d'ancrage total dans la table: dans ce cas, il y a lieu de tenir compte des capacités résistantes des plaques ancrées, la poutre ainsi renforcée pouvant être justifiée sur la totalité de sa section (hauteur de table comprise).

Renforcement sans plaques d'ancrage : Ce type de renforcement n'est pas admis dans le cas d'un moment négatif sur l'appui considéré. Dans ce cas, la poutre ainsi renforcée doit être justifiée vis-à-vis de l'effort tranchant, en ne tenant compte que de la retombée sous dalle.

Dans tous les cas, les deux vérifications données dans le dossier technique établi par le demandeur sont à effectuer. Il s'agit :

- de la vérification en traction du composite,
- de la vérification de non-glissement du plan de collage.

2.33 Conditions de mise en œuvre

La mise en œuvre est effectuée exclusivement par l'entreprise VSL FRANCE. Elle doit être effectuée dans les strictes conditions définies dans le dossier technique établi par le demandeur, notamment pour ce qui concerne le nettoyage et la préparation des supports ainsi que la réalisation des essais de convenances sur ce même support. Il est précisé que ces essais doivent être effectués pour chaque chantier et pour tous les supports visés par le présent Avis Technique.

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Conclusions

Appréciation globale

L'utilisation du procédé de renforcement V2C dans le domaine d'emploi accepté est appréciée favorablement.

Validité

Jusqu'au 30 juin 2012

Pour le Groupe Spécialisé n° 3
Le Président

J.-P. BRIN

3. Remarques complémentaires du Groupe Spécialisé

Le Groupe Spécialisé n°3 tient à souligner que le procédé présenté, bien que comparable au procédé de renforcement par tôles collées, présente des possibilités supérieures à ce dernier, notamment en ce qui concerne l'exécution et la maîtrise de l'encollage.

Pour ce qui concerne le dimensionnement, le Groupe Spécialisé n°3 considère que la valeur de la résistance caractéristique à la traction f_{tk} du béton, issue de l'essai de pastillage, est conservative pour le calcul de $\overline{\tau}$ et de $\overline{\tau_{II}}$ par rapport au procédé visant les tôles collées.

Concernant les températures en service continu et températures de pointe, le groupe a limité ces températures pour tenir compte de l'incertitude que l'on a à l'heure actuelle sur le vieillissement des matériaux soumis à des températures élevées.

Le Rapporteur du Groupe Spécialisé n° 3

M.CHENAF

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Dossier Technique
établi par le demandeur

A. Description

1. Présentation du procédé

1.1 Le procédé V2C

Le procédé V2C est un procédé de renforcement de structures en béton armé ou précontraint par matériaux composites en fibres de carbone (lamelles et tissus), collés par une résine époxydique, afin d'accroître leur résistance.

Cette méthode est comparable au procédé l'Hermite ou collage de tôle en acier.

Le procédé V2C est dimensionné et mis en œuvre par la société VSL France.

1.2 Les matériaux composites

Issus de l'industrie aéronautique, les matériaux composites FRP à matrice polymérique renforcée de fibres s'utilisent depuis une dizaine d'année dans le domaine du BTP. La fibre de carbone se caractérise par un module d'élasticité élevé (au moins 165 GPa), sa faible densité (1.6 g/cm^3), l'absence de risque de corrosion, sa facilité de manutention et ses propriétés mécaniques. Les fibres noyées dans une matrice époxyde forment un composite (lamelle /tissus).

1.3 Domaine d'application

Structure en béton armé et précontraint soumise à un chargement statique et dont la température en service n'excède pas :

- résine S3D : 40°C en service - 45°C en accidentel
- résine S4D ou S4DT : 50°C en service - 55°C en accidentel
- résine P204 : 50°C en service - 55°C en accidentel
- résine MHT : 75°C en service - 80°C en accidentel

1.4 Avantage du procédé

- Mise en œuvre facile
- Encombrement réduit
- Le carbone étant un matériau inerte, il est insensible à la corrosion, et aux agents chimiques

2. Choix du système composite FRP

2.1 Les lamelles S&P CFK

Les lamelles CFK sont fabriquées par filage à chaud. Pendant leur fabrication, les fibres de carbone d'un module d'élasticité différent et d'une résistance à la traction différente sont étirées puis plongées dans une matrice époxyde et subissent un durcissement progressif sous l'influence d'une source de chaleur.

Produits de base :

- fibres : Tenax UTS, Toray 700S ou Tenax IMS (selon le type de produit)
- Résine : époxy thermodurcissable produite par CIBA GEIGY selon une formulation mise au point pour ISOSPORT.

Fabricant :

S&P reinforcement C/O Isosport verbundbauteile GmbH en Autriche

Principales caractéristiques :

	Lamelle CFK 150/2000	Lamelle CFK 200/2000
% vol volumique des fibres	68%	68%
Densité [g/cm^3]	1.6	1.6
Module d'élasticité [GPa]	>165	>205
allongement à la rupture	1.4%	1.2%
résistance à rupture en traction [MPa]	>2500	>2500

2.2 – Les tissus S&P

Les tissus C-Sheet sont des matériaux unidirectionnels composés de fibres droites noyées dans une matrice en polymère.

Produits de base :

- Tenax STS ou Toray 600 (C Sheet 240)
- Mitsubishi K63712 (C Sheet 640)

Fabricant : S&P Reinforcement (entreprise certifiée ISO 9001) à Brunnen en Suisse.

Principales caractéristiques : (fibres seules)

	C-Sheet 240 (200 g/m2)	C-Sheet 240 (300g/m2)	C-Sheet 640 (400g/m2)
Module E [GPa]	240	240	640
Allongement à la rupture	1.55%	1.55%	0.4%
Densité [g/cm^3]	1.7	1.7	2.1
Masse [gr/m^2]	200	300	400
Epaisseur théorique [mm]	0.117	0.176	0.19

2.3 Résines utilisées pour la mise en œuvre :

Les résines et autres produits utilisés pour la réparation des surfaces proviennent de la société RESIPOLY.

Pour la mise en œuvre des lamelles et des tissus, les températures d'application des résines époxy sont les suivantes.

tissus :	- S3D	+5°C à +30°C
	- S4D ou S4DT	+10°C à +40°C
	- MHT	+5°C à +30°C
lamelles :	- P204	+5°C à +35°C

2.4 Caractéristiques des composites

L'ensemble fibres de carbone noyées dans une matrice époxy est appelé composite.

Les lamelles définies précédemment constituent donc un composite composé de 68% de fibres et de 32% de résine époxyde (valeur minimale garantie).

Le tissu mis en œuvre est composé de (valeurs minimales garanties)

- 35% de fibres, 65% de matrice époxy pour C-sheet 240 (200 g/m2)
- 32% de fibres, 68% de matrice époxy pour C-sheet 240 (300 g/m2)
- 27% de fibres, 73% de matrice époxy pour C-sheet 640 (400 g/m2)

	C-Sheet 240 (200 g/m^2)	C-Sheet 240 (300 g/m^2)	C-Sheet 640 (400 g/m^2)
Densité [g/cm^3]	1.7	1.7	2.1
Module E [MPa]	84 000	76 800	173000
Epaisseur [mm]	0.334	0.550	0.678
allongement à la rupture	1.25%	1.25%	0.25%
résistance à rupture en traction [MPa]	>1050	>960	>432

3. Principe de dimensionnement

3.1 Hypothèses de calculs

Les calculs sont menés conformément au BAEL 91 et BPEL en retenant les hypothèses fondamentales du calcul béton :

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- Les sections droites restent planes après déformation (hypothèse de Navier Bernoulli)
- Il n'y a pas de glissement relatif entre les armatures existantes, la fibre de carbone et le béton
- La résistance à la traction du béton est négligée
- Le comportement des matériaux aciers et béton, les coefficients de sécurité et les combinaisons des charges sont donnés dans les règlements usuels (BAEL – BPEL).

3.2 Notations utilisées

Géométrie :

bo : largueur de la section
h : hauteur de la section
d : hauteur utile
As (As') : section d'acier tendus (comprimés)
Af : section de fibres utilisées
bf : largeur du composite
Lf : longueur du composite
tf : épaisseur du composite
At : section des armatures transversales
St : espacement des armatures transversales
α : angle des armatures transversales avec la fibre moyenne de la poutre
β : angle d'inclinaison des bielles d'about

Matériaux :

fcj (ftj) : résistance caractéristique à la compression (traction) du béton à j jours
fe : limite d'élasticité de l'acier
fed : contrainte de dimensionnement retenue pour l'acier (fed=fe/γs)
fcd : contrainte de dimensionnement retenue pour le béton (fcd=0.85 fc28/(θxγb))
ffd : contrainte de dimensionnement retenue pour les lamelles
σbc : contrainte dans le béton
σs : contrainte dans l'acier tendu
σs' : contrainte dans l'acier comprimé
σf : contrainte dans les fibres
n : coefficient d'équivalence acier /béton (n=Es/Eb) pris égal à 15 (BAEL A4.5,1)
n' : coefficient d'équivalence fibres/béton (n=Ef/Eb)
Ef : module d'élasticité de la fibre utilisée

Sollicitations :

Mu : moment sollicitant la section à l'ELU
Ms : moment sollicitant la section à l'ELS
Vu : tranchant sollicitant la section à l'ELU
τu : contrainte tangente à l'ELU

Calcul BA ELU-ELS:

Yu : distance de l'AN à la déformation de la fibre la plus comprimée (ELU)
Z : bras de levier
I1 : moment d'inertie de la section homogène
yo : distance de l'AN à la déformation de la fibre la plus comprimée (ELS)
fctm : valeur de la résistance à la traction superficielle du béton
fctd : valeur de calcul de la résistance à la traction superficielle du béton fctd=fctm/ γ_b

Calcul BP ELU-ELS :

f_{prg} : charge de rupture garantie
η : coefficient de fissuration
Bt : aire du béton tendu
σ_{Bt} : valeur absolue de la contrainte maximale de traction
N_{Bt} : résultante des contraintes de traction correspondantes (calculées en section non fissurée en classe 2 et 3)
b_n : épaisseur nette
τ : contrainte tangente de l'élément calculée à partir de l'épaisseur nette bn
σ_x : contrainte normale de la poutre calculée à partir de la section nette
σ_t : contrainte normale transversale calculée à partir de l'épaisseur nette bn
F_{tu} : effort résistant des armatures de pré contraintes transversales
s_t' : espacement des armatures transversales précontraintes mesurées perpendiculairement à la fibre moyenne.
α' : angle des armatures transversales précontraintes avec la fibre moyenne de la poutre
β_u : angle des fissures d'effort tranchant avec la fibre moyenne de la poutre (β_u>30°)

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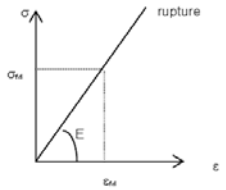
3.3 Caractéristiques techniques des produits VSL utilisées pour le dimensionnement

3.31 Loi de comportement des composites carbone (lamelles et tissus)



3.32 Lamelles CFK 150/2000 et CFK 200/2000

Utilisation : renforcement vis à vis de la flexion

	Lamelle CFK 150/2000	Lamelle CFK 200/2000
Module E [MPa]	165000	205000
Epaisseurs disponibles [mm]	1.2 et 1.4	1.2 et 1.4
$\varepsilon_{dim\,ELU}$	0.7%	0.65%
$\sigma_{dim\,ELU}$ noté f_{fd} [MPa]	1148	1332
$\sigma_{dim\,ELS}$ noté σ_f [MPa]	400	500

Lorsque l'adhérence totale est requise en tout point d'un renforcement, les contraintes dans les lamelles seront limitées aux valeurs suivantes :

	Lamelle CFK 150/2000	Lamelle CFK 200/2000
$\sigma_{adhérence\ limite}$ noté σ_{fdm} [MPa]		
e = 1.2 mm	367	-
e = 1.4 mm	339	378

3.33 C-sheet 240 & C-sheet 640

Utilisation : renforcement vis à vis de la flexion (solution alternative aux lamelles)

	C-Sheet 240 (200g/m²)	C-Sheet 240 (300g/m²)	C-Sheet 640 (400g/m²)
Module E [MPa]	84 000	76 800	173000
Epaisseur théorique [mm]	0.334	0.550	0.678
ε_{dim} ELU (flexion ou tranchant)	0.7%	0.7%	0.16%
σ_{dim} ELU noté f_{fd} [Mpa]	590	540	280
σ_{dim} ELS noté σ_f [MPa]	280	254	140

3.4 Structures en béton armé

3.41 Principe de calcul du renforcement vis à vis de la flexion

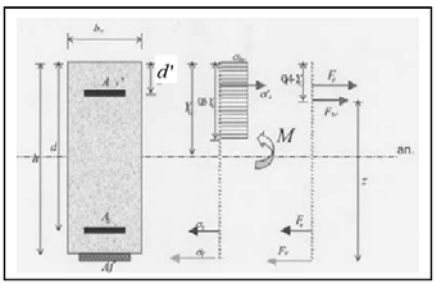
Condition préalable :

Le moment résistant (accidentel) de la structure non renforcée doit être supérieur au moment ultime (accidentel) de la structure sous le chargement final.

Sont visées dans ce paragraphe les structures soumises à la flexion pure ou flexion composée. Le cas de la flexion pure est détaillé ci-après, étant entendu que le principe de calcul sera identique pour les structures sollicitées en flexion composée.

L'étude est menée à l'ELU et à l'ELS en considérant les différentes phases de chargement.

1/ Dimensionnement de la section de fibre à mettre en œuvre à l'ELU sous chargement final



L'équilibre de la section donne :

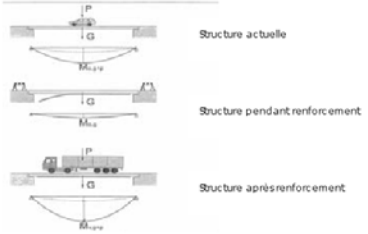
$$A_f = \frac{M_u - A_S \cdot f_{ed} \cdot (z-(h-d)) + A_{S'} \cdot f_{ed} \cdot (z-(h-d'))}{z \cdot f_{fd}}$$

avec $z = 0{,}5 \cdot h \cdot \left(1+\sqrt{1-2\mu_u}\right)$

et $\mu_u = \dfrac{M_u + A_S \cdot f_{ed} \cdot (h-d) - A_{S'} \cdot f_{ed} \cdot (h-d')}{b_0 \cdot f_{cd} \cdot h^2}$

2/ Vérification des contraintes à l'ELS en considérant les différentes étapes de chargement :

- Structure à l'état initial (avant renforcement)
- chargement de la structure renforcée (chargée par $M_{s\,final} - M_{s\,initial}$)
- structure à l'état final correspondant à superposition des états précédents



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- Calcul des contraintes :

  l'équilibre des forces donne :

$$0{,}5\cdot b_0\cdot y_0^2+n\cdot\left(A_s+A_{s'}+A_f\cdot\frac{n'}{n}\right)y_0-n\cdot\left(A_s\cdot d+A_{s'}\cdot d'+A_f\cdot\frac{n'}{n}\cdot h\right)=0$$

D'où :

$$y_0=\frac{-n\cdot\left(A_s+A_{s'}+\frac{n'}{n}\cdot A_f\right)+\sqrt{\Delta}}{b_0}$$

Avec

$$\Delta=\left[n\cdot\left(A_s+A_{s'}+Af\cdot\frac{n'}{n}\right)\right]^2+2\cdot n\cdot b_0\cdot\left(A_s\cdot d+A_{s'}\cdot d'+Af\cdot\frac{n'}{n}\cdot h\right)$$

On peut alors en déduire :

- le moment d'inertie :

$$I_1=\frac{b_0}{3}\cdot y_0^3+n\cdot As\cdot(d-y_0)^2+n\cdot As\cdot(d'-y_0)^2+n'\cdot Af\cdot(h-y_0)^2$$

- les variations de contraintes dans :

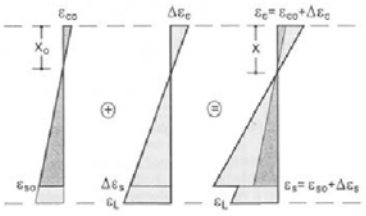
  le béton : $\sigma_{bc}=\frac{M_s}{I_1}\cdot y_0$

  l'acier comprimé : $\sigma_{sc}=n\cdot\frac{M_s}{I_1}(y_0-d')$

  l'acier tendu : $\sigma_s=n\cdot\frac{M_s}{I_1}(d-y_0)$

  les fibres : $\sigma f=n'\cdot\frac{(M_{s\,fmax}-M_{s\,mmax})}{I_1}(h-y_0)$

- superposition des contraintes



Les contraintes ne peuvent être

Finalement, les contraintes totales sont comparées aux contraintes admissibles données dans le BAEL

- le béton : $\sigma_{be\,tot}=\sigma_{be}+\Delta\sigma_{be}<0{,}6\cdot f_{c28}$
- l'acier comprimé : $\sigma_{sc\,tot}=\sigma_{sc}+\Delta\sigma_{sc}<\overline{\sigma_s}$
- l'acier tendu : $\sigma_{s\,tot}=\sigma_s+\Delta\sigma_s<\overline{\sigma_s}$
- le composite : $\sigma_f<\overline{\sigma_f}$

NOTA : La contrainte limite dans l'acier à considérer dépend du degré de fissuration retenu :

En fissuration peu préjudiciable, $\overline{\sigma_s}=fe$

En fissuration préjudiciable et préjudiciable (BAEL A.4.5,33 et A.4.5,34)

3/ Contrainte de glissement

On note :

S : l'effort d'entraînement qui varie comme l'effort tranchant

L'effort dans la lamelle s'exprime en fonction de la contrainte de glissement : $F_l=A_L\times f_{fd}=0{,}5\times S\times\frac{Lf}{2}$

La contrainte de glissement est égale à : $\tau_{glissement}=\frac{S}{b_f}=4\times\frac{A_f\times f_{fd}}{b_f\times L_f}$

La valeur obtenue est à comparer à la valeur de cisaillement limite à l'ELU .

4. Vérification de l'ancrage des lamelles carbone

Des recherches effectuées sur les zones d'ancrage des lamelles carbones ont montrées que la distribution des contraintes de cisaillement à l'interface béton – lamelle est très différente de la répartition moyenne des contraintes.

La différence de contraintes entre l'estimation moyenne et le pic réel de contrainte existant à cet endroit peut expliquer pour certaines poutres les problèmes de délamination.

- longueur d'ancrage des lamelles

  La longueur d'ancrage des lamelles est donnée par la formule suivante :

$$l_{bd}=0.6\times\sqrt{\frac{E_f\times t_f}{f_{ctd}}}$$

où : l_{bd} : longueur d'ancrage en mm

E_f : module d'élasticité du produit utilisé en MPa

t_f : épaisseur du produit utilisé en mm

f_{ctd} : valeur de calcul de la cohésion superficielle du support en Mpa

NOTA : La longueur d'ancrage des tissus est de 10 cm (C-Sheet 240 (200g/m² et 300g/m²) et C-Sheet 640.

- Effort maximal repris par la zone d'ancrage

  Il convient de vérifier que l'effort existant dans le renfort au niveau de l'ancrage est bien inférieur à l'effort que peut reprendre la lamelle à cet endroit, déterminé comme suit :

$$F_{bd}=0.5\times b_f\times k_f\times k_t\times\sqrt{E_f\times t_f\times f_{ctd}}$$

où F_{bd} : Effort maximal repris par la zone d'ancrage en N

E_f : module d'élasticité du renfort utilisé en MPa

t_f : épaisseur du renfort utilisé en mm

b_f : largeur totale du renfort utilisée en mm

f_{ctd} : cohésion superficielle du support existant en MPa

k_t : coefficient dépendant de la température

k_t =1 structure située à l'intérieur

k_t =0.9 structure située à l'extérieur

k_f : coefficient de forme.

$$k_f=1.06\times\sqrt{\frac{2-\frac{b_f}{b_a}}{1+\frac{b_f}{400}}}$$

b_a : largeur de la section

Les distances entre axes des renforts sous dalles doivent être tels que le rapport de la plus grande distance à la plus petite ne dépasse pas 2.

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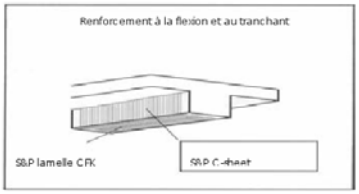
3.42 Principe de dimensionnement vis à vis de l'effort tranchant

En cas d'insuffisance d'armatures d'effort tranchant, on procède par analogie au calcul béton armé (méthode de BRESSON).



On détermine tout d'abord l'effort maximal pouvant être repris par les aciers existants et le béton le cas échéant. L'effort à reprendre par le tissu est alors égal à la différence entre l'effort tranchant calculé et l'effort tranchant repris par les aciers existant et le béton.

- Formulation générale :

$$\tau_u = \frac{V_u}{b_0 \times d}$$

$$V_u = V_{u\,acier} + V_{u\,béton} + V_{u\,fibres}$$

$$V_{u\,béton} = 0.3 \times k \times ftj \times ba \times d$$

$$V_{u\,acier} = \frac{Af}{st} \times 0.9 \times d \times \sigma_{sd} \times (\cot an\,\beta + \cot an\,\alpha) \times \sin\,\beta$$

$$V_{u\,fibres} = \frac{Af}{zf} \times z \times ffd \times (\cot an\,\beta + \cot an\,\alpha) \times \sin\,\beta$$

NOTA : l'allongement des aciers existant est limité à celui du tissu C-Sheet 640 soit 0.17%.

- Valeur à retenir pour le bras de levier:

  Lors du dimensionnement, il est intéressant de différencier les 2 mises en œuvre possibles :

  - le renforcement entoure entièrement la poutre, ou des dispositifs particuliers permettent l'ancrage du tissu dans la zone de béton comprimé.
  - Le renforcement est disposé sur les joues de la poutre, avec un retour horizontal et sans dispositifs d'ancrage particuliers. Cette disposition est retenue généralement dans le cas d'une poutre surmontée d'une dalle.

Reprise de l'effort tranchant dans le cas où l'ancrage dans la zone comprimée du béton est assuré

Enrobage complet
z=0.9 h

Ancrage dansla zone comprimée
z=min (0.9 h ;h retour)

Reprise de l'effort tranchant dans le cas où l'ancrage dans la zone comprimée du béton n'est pas assuré

On procède selon la méthode de l'AFGC : le bras de levier est pris égal à la retombée de la poutre diminuée de la longueur de collage.

$z = h_{retombée} - l_{collage}$ si retour horizontal

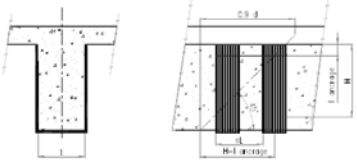
$z = h_{retombée} - 2xl_{collage}$ sinon

$l_{collage}$ =100mm

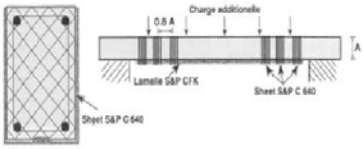
$Af = 2xt_f x b_f$

- dispositions constructives à respecter lors de la mise en œuvre du tissu de fibres de carbone

Les dispositions constructives à respecter sont les suivantes :



De manière à éviter qu'une fissure d'effort tranchant puisse se former entre 2 bandes de tissu, on limite l'espacement entre 2 bandes à 80% de la hauteur de la poutre.



Tissu considéré comme ancré dans la zone comprimée :

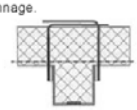
Cet ancrage peut être réalisé à l'aide d'une plaque en acier, collée sur le tissu, et de barres en aciers scellées dans la structure et soudées sur la plaque. Ces éléments en acier devront bien entendu être protégé de la corrosion. (peinture époxy par ex)

On peut également utiliser des bandes de tissus repliées plusieurs fois.

Cas particulier d'un élément soumis à des moments négatifs :

Dans le cas où la place est un facteur déterminant pour le type de renforcement, on procède de la manière suivante : les aciers traversent la dalle et sont repliés sur la dalle. Ils sont ensuite ligaturés et protégés par du béton.

On peut également disposer en partie supérieure une plaque d'acier maintenue par boulonnage.



3.5 Structures en béton précontraint

3.51 Calcul du renforcement vis à vis de la flexion

- Condition préalable :

  Les éléments en béton précontraint ne pourront être renforcés que si la vérification est effectuée en classe immédiatement supérieure à celle du dimensionnement d'origine : ainsi, une section de classe 1 sera renforcée de manière à vérifier la classe 2, de même, une section de classe 2 sera renforcée et vérifiée en classe 3.

  Cas des éléments de prédalles précontraintes et dalles alvéolées précontraintes : compte tenu des niveaux de traction admissibles pour ces éléments (selon le CPT plancher), il est admis que le renforcement de ces éléments n'est pas couvert par ce document.

- Vérification ELS :
  - en classe 2

    Le calcul des contraintes est effectué en section non fissurée, en vérifiant les contraintes données dans le BPEL (art. 6.1,24)

    Au moment du renforcement : ftj=0 (aucune traction n'est admise)

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En service : sous combinaison rare : ftj dans la section d'enrobage ; 1.5 x ftj ailleurs

Sous combinaison fréquente : 0 dans la section d'enrobage

- en classe 3 :

Le calcul est effectué en section fissurée : calcul en flexion composée en considérant l'historique du renforcement :

Etape 1 : structure à l'état initial (avant renforcement)

Etape 2 : chargement de la structure renforcée

Etape 3 : structure à l'état final correspondant à superposition des états précédents

Les limitations sont les suivantes :

- pour le béton :

  0.6 fcj (ou 0.5 fcj sous combinaison quasi permanente)

- pour les aciers passifs :

  En combinaison rare : $\overline{\sigma_s} = \max\left\{\frac{2}{3} fe ; 110\sqrt{\eta \times ftj}\right\}$

  En combinaison fréquente : 0.35fe

- pour les aciers de précontraintes : (exploitation)

  En combinaison rare : la surtension dans les armatures de précontrainte est limitée à : $0.1 f_{prg}$ pour la post tension

  $\min\{0.1 \times f_{prg} ; 150\ \eta_p\}$ pour la prétension

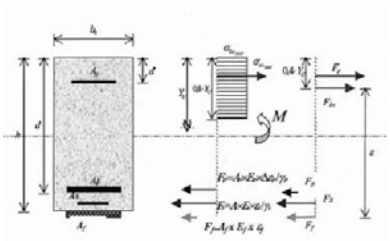
  En combinaison fréquente : la surtension dans les armatures de précontrainte est limitée à 100MPa

  En combinaison d'exploitation: aucune traction n'est admise dans la section d'enrobage

- pour les fibres de carbone : σ_f

- Vérification ELU :

  On vérifie que compte tenu de la géométrie de la section et de son ferraillage, le moment résistant de la section est supérieur au moment sollicitant.



- Aciers minimum :

L'article 6.1,32 du BPEL concernant le ferraillage mini à disposer en partie tendue est à respecter :

$$A_{s\,min\,t} = \frac{Bt}{1000} + \frac{N_{ut}}{\sigma_{st}} \times \frac{f_g}{f_e}$$

3.52 Calcul du renforcement vis à vis de l'effort tranchant

- Vérification ELS

Les contraintes doivent être vérifiées conformément à l'article 7.2.1 du BPEL. Les inégalités à vérifier sont rappelées ci dessous :

$$\tau^2 - \sigma_s \times \sigma_t \le 0.4 \times f_g \times \left[ftj + \frac{2}{3}(\sigma_s \times \sigma_t)\right]$$

et

$$\tau^2 - \sigma_s \times \sigma_t \le 2 \times \frac{f_g}{f_{cj}} \times \left[0.6 \times fcj - (\sigma_s + \sigma_t)\right] \times \left[ftj + \frac{2}{3}(\sigma_s \times \sigma_t)\right]$$

Vérification ELU

La méthode utilisée est celle du BPEL en inscrivant que l'effort tranchant réduit est repris par le béton, les aciers existants et les fibres rajoutées.

$$\tau_{u\,réduit} = \tau_{u\,acier} + \tau_{u\,béton} + \tau_{u\,fibres}$$

Où

$$\tau_{u\,acier} = \frac{At}{b_n \times s_t} \frac{fe}{\gamma_s} \frac{\sin(\alpha + \beta_u)}{\sin \beta_u} + \frac{F_{pu}}{b_n \times s'_t} \frac{\sin(\alpha + \beta_u)}{\sin \beta_u}$$

$$\tau_{u\,béton} = \frac{f_g}{3}$$

$$\tau_{u\,fibres} = \frac{Af}{b_n \times s_f} \times f_{fd} \times \frac{\sin(\alpha + \beta_u)}{\sin \beta_u}$$

$$\tan 2\beta_u = \frac{2 \times \tau_u}{\sigma_{xu} - \sigma_{tu}}.$$

Les bras de levier pour le calcul de la fibre sont pris égaux à : (cf §3.4)

- Min (0.9h ; $h_{renfort}$) si ancrage dans la zone comprimée est assuré
- $h - l_{collage}$ sinon (avec retour horizontal)
- $h - 2xl_{collage}$ sinon (sans retour horizontal)

La section de fibres à disposer est égale à : $A_f = 2xt_f x b_f$

4. Mise en œuvre du procédé VSL de renforcement par fibres de carbone

Ce paragraphe définit brièvement les principales étapes de la mise en œuvre. Pour de plus amples renseignements, se reporter au PAQ de VSL France.

4.1 Réception et stockage des produits S&P et RESIPOLY au dépôt et sur chantier

Chaque livraison fait l'objet d'une vérification de conformité à la commande accompagnée d'un contrôle visuel. Une fiche de contrôle sera émise pour chaque contrôle.

4.2 Condition d'application

Réception des supports

On s'assurera que les éléments suivants sont respectés :

- l'aptitude du support à supporter un renforcement par fibres de carbone
- le support existant doit présenter une cohésion superficielle de 1.5 MPa (ELU) au minimum. Un essai de traction sera effectué à l'aide de pastilles découpées dans une lamelle (le nombre est à définir en fonction du support).
- Les supports ne devront pas être contaminés en profondeur par des produits pouvant empêcher la bonne adhérence de la résine (hydrocarbures...).

Si l'état de surface ne répond pas au précédent critère, un traitement ou réparation du béton contaminé sera effectué conformément à la norme NF P 95-101 (réparation et renforcement des ouvrages en maçonnerie – reprise du béton dégradé superficiellement). On retiendra par exemple :

- injection des fissures supérieures ou égales à 0.3 mm
- traitement des armatures corrodées
- reconstitution du béton

Conditions ambiantes d'application :

- humidité de l'air inférieure à 85%
- température de l'air comprise entre 5°C et 45°C
- température du support supérieure à 5°C
- humidité du support inférieure à 5% (non ruisselant)

Le critère d'application du renforcement est défini grâce au tableau des points de rosées.

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4.3 Application des lamelles S&P CFK et du tissu S&P C-Sheet

Préparation des produits de carbone avant collage.

- Dégraissage des lamelles à l'aide d'un solvant (MEXYL).
- Découpe des lamelles et des bandes de tissus aux dimensions indiquées sur les plans.
- Mélange des 2 constituants de la résine époxy jusqu'à l'obtention d'une teinte homogène.

Application des lamelles par double encollage

- Imprégner le support avec la résine P204
- Etaler également une couche de résine P204 sur la lamelle
- Pose de la lamelle (pendant la DPU de la résine) par pression manuelle
- Marouflage : l'excédent de colle doit être expulsé latéralement
- Nettoyage des excès de colle au couteau
- Après polymérisation, nettoyage des lamelles.

Cas d'un croisement des lamelles :

Appliquer une résine de reprofilage (P204) avant la pose de la lamelle croisée.

Cas des lamelles insérées :

Remplissage de résine P204 la saignée nettoyée avant mise en place de la lamelle, et nettoyage du surplus.

Application des tissus

- Imprégner le support avec la résine S3D, S4D.
- Pose manuelle du tissu (pendant la DPU de la résine) en commençant par une extrémité
- Marouflage dans le sens des fibres jusqu'à l'apparition de la résine à travers la trame
- Application immédiate de la résine de fermeture

Cas de couches superposées de tissus :

Répéter les opérations précédemment décrites sur la couche de fermeture non polymérisée. Dans le cas contraire, la couche de fermeture doit être dépolie au papier abrasif, puis la surface doit être nettoyée et aspirée avant la répétition des opérations précédentes.

Cas d'un renforcement avec lamelles et tissus :

Suivre les instructions pour la pose des lamelles, puis appliquer sur la zone de recouvrement la résine P204 avant de suivre le processus décrivant la mise en place du tissu.

5. Dispositions particulières

5.1 Perméabilité à la vapeur d'eau

En effet, 30% à 50% de la surface de l'élément doit être perméable à la vapeur d'eau : la résine époxy ne peut donc pas recouvrir la totalité de la sous face d'une dalle de couverture recouverte d'une étanchéité en partie haute.

5.2 Protection au feu

Les fibres de carbone, contrairement à la colle, ont une bonne résistance aux températures élevées. La capacité portante du renforcement par fibres de carbone ne peut donc pas être justifié dans le cas d'un incendie.

Lorsque la stabilité au feu de la structure peut être justifiée par les régles FB en prenant en compte les aciers existants sous combinaison accidentelle, aucune disposition de protection des renforts n'est à prévoir.

5.3 Travaux de peinture et de revêtement

Le procédé VSL de renforcement des structures par fibres peut recevoir tout type de peinture, époxy, polyuréthane ou autre compatible avec le support.

B - Références

Quelques références de chantiers réalisés (liste non exhaustive) :

- LOOS 59 (Nord Est immobilière des chemins de fer) : renforcement de 12 poutres dans le cadre de la réhabilitation du bâtiment,
- OA20 Arcueil-Cachan 94 (D.D.E du Val de Marne) : renforcement des poutres en béton armé du tablier,
- Viaduc d'Incarville 27 (Société des autoroutes Paris Normandie) : renforcement de 28 poutres précontraintes,
- Viaduc de l'Yonne 89 (Autoroutes Paris Rhin Rhône) : renforcement de 12 poutres en béton précontraint du tablier,
- ARCELOR, colonne d'équilibre 57 (ARCELOR) : renforcement d'une colonne d'équilibre,
- Parking Saint Serge ANGERS 49 (Carrefour) : renforcement de 17 poutres en béton précontraint du parking aérien,
- Grand Plaisir PLAISIR 78 (SCI de la passerelle Est et Ouest) : réparation et renforcement des poutres principales de la passerelle franchissant le CD 161,
- Aéroréfrigérant DAMPIERRE 45 (EDF CNEPE) : renforcement de la coque externe de l'aéroréfrigérant,
- Terrasse CARPEAUX 92 (EPAD) : renforcement de poutres en béton armé dans la station RER de la Défense,
- Station d'épuration HOUPLIN ANCOISNE 59 (Communauté Urbaine de Lille) : renforcement de 6 passerelles en béton armé,
- BMS-UPSA Rueil-Malamison 92 (OTH bâtiment) : renforcement de poutres, d'une dalle et d'une trémie.