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French standard

NF P 18-710
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National addition to Eurocode 2 — Design of concrete structures: specific rules for Ultra-High Performance Fibre-Reinforced Concrete (UHPFRC)

F : Complément national à l'Eurocode 2 — Calcul des structures en béton : règles spécifiques pour les Bétons Fibrés à Ultra-Hautes Performances (BFUP)
D : Nationale Ergänzung zu Eurocode 2 — Bemessung von Betonstrukturen: spezifische Bestimmungen für faserbewehrte Ultrahochleistungsbetone

French standard approved

by decision of the Director General of AFNOR.

Correspondence

At the date of publication of this document, there is no international or european standardization works on the same subject.

Summary

This document, which can be considered as a national complement to Eurocode 2, applies to the design of UHPFRC structures (buildings and civil engineering). Consequently, this document is concerned with the requirements for resistance, serviceability, durability and fire resistance of these structures.

Descriptors

Technical International Thesaurus: construction, civil engineering, concrete structures, computation, stresses, limits, materials, mechanical strength, deformation, reinforcing steels, prestressing steels, durability, stress analysis, flexing, shear strength, tensile strength, strain, fatigue life, cracking (fracturing), anchorages, armatures, distance, covering, beams: supports, floors, slabs, partitions, columns, foundations, verification.

Modifications

Corrections

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The standard is regularly reviewed to assess its appropriateness over time.

Any standard is considered to be effective as from the date presented on the first page.

Understanding standards

The reader's attention is drawn to the following points:

Only the verbal form **shall** is used to express one or more requirements that shall be satisfied in order to comply with this document. Such requirements may be contained in the body of the standard or in a so-called "normative" annex. For test methods, the use of the imperative corresponds to a requirement.

Expressions involving the verbal form **should** are used to express a possibility that, while preferred, is not actually necessary in order to comply with this document. The verbal form **may** is used to express practical, but not mandatory advice or suggestions, or permission.

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The composition of the standardisation commission that prepared this document is provided below. If experts are representing a body other than the body to which they habitually belong, this information is presented using the following convention: body of membership (body represented).

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Contents

Foreword

The present standard NF P18-710 "Design of concrete structures: specific rules for ultra-high performance fibre-reinforced concrete (UHPFRC)" which constitutes a national complement to Eurocode 2, has been prepared within the BNTRA CN EC2 "Design of concrete structures" committee to meet the French requirement to standardise the use of ultra-high performance fibre-reinforced concrete (known as UHPFRC in the rest of the document) in building and civil engineering designs.

This standard is intended to be used in conjunction with the two other standards dealing with the topic of UHPFRCs:

- standard NF P18-470 "UHPFRC: Specification, performance, production and compliance" which deals more specifically with the UHPFRC material itself and which may be considered as an adaptation of the standard NF EN 206 to the case of UHPFRCs
- standard NF P 18-451 "Execution of concrete structures – Specific Rules for UHPFRCs"

The present standard is aimed at the design of structures in UHPFRC (buildings and civil engineering structures). This standard thus gives the requirements in terms of resistance, serviceability, durability and fire resistance for these structures. The characteristics of the UHPFRCs covered by the present standard are shown in the "Scope" section below.

The present standard has been drafted by adapting Eurocode 2[1] to the case of structures in UHPFRC. The table of contents for the present standard identically reproduces the whole of the table of contents for part 1-1 of Eurocode 2 and where there is no particularity with regards to UHPFRCs relative to the concretes dealt with in Eurocode 2, the word "Unchanged" is written[2]. With regard to the articles specific to UHPFRCs, these are mostly based on the "Structural design methods" section of the recommendations on UHPFRCs by the Association Française de Génie Civil (AFGC) (*French Association for Civil Engineering*) working group[3]. Compared with the concretes covered by Eurocode 2 (concretes C12/15 to C90/105), the main distinguishing features of UHPFRCs are a much higher compression strength and the fact the tensile strength of the material can be taken into account, including after cracking, which changes the use of traditional reinforcing steel or prestressing tendons. This involves defining new concepts, such as the partial factor for tensioned UHPFRC γ_{cf}, inclusion of the distribution and orientation of the fibres through an orientation factor K and a new non-brittleness check. UHPFRCs are also higher performance than the concretes covered by Eurocode 2 with regard to durability which is expressed in particular in this standard by the modification of the formulae for calculating concrete cover.

[1] The precise references of the standards forming Eurocode 2 are given in part 1.1 of the present standard.

[2] When the word "Unchanged" appears, the reader must in any case remember that the word "concrete" in Eurocode 2 is referring to UHPFRC.

[3] "Ultra-high performance fibre-reinforced concretes - Recommendations" Revised edition, June 2013

1 GENERAL

1.1 Scope

(1)P This standard applies to the design of buildings and civil engineering structures in unreinforced UHPFRC, reinforced UHPFRC or prestressed UHPFRC. It complies with the principles and requirements for safety and serviceability of structures and the design and data verification bases in EN 1990: Basis of structural design. If the structure comprises members produced in UHPFRC as well as others in other materials (steel, wood, ordinary concrete, composites, etc.), reference should be made to this standard for the justification on the members in UHPFRC.

NOTE Buildings are defined by their "area of use": see standard NF EN 1991-1-1 and its National Annex. Category E buildings (industrial activity and storage) along with non-standard parts of buildings in categories A to D may need specific requirements. These must be specified in special contract documents.

(2)P This standard only deals with requirements regarding strength, serviceability, durability and fire resistance of structures in UHPFRC. Other requirements, such as those relating to thermal and acoustic insulation for example, are not covered.

(3)P This standard is designed to be used in conjunction with the following standards:

Eurocode 2 "Design of concrete structures"

NF EN 1992-1-1 "General rules and rules for buildings" (October 2005) incorporating the 1st corrigendum (January 2008) and 2nd corrigendum (November 2010)

NF EN 1992-1-1/NA "National Annex to NF EN 1992-1-1" (March 2007)

NF EN 1992-1-2 "Structural fire design" (October 2005)

NF EN 1992-1-2/NA "National Annex to NF EN 1992-1-2" (October 2007)

NF EN 1992-2 "Concrete bridges - Design and detailing rules" (May 2006)

NF EN 1992-2/NA "National Annex to NF EN 1992-2" (April 2007)

NF EN 1992-3 "Liquid retaining and containment structures" (December 2006)

NF EN 1992-3/NA "National Annex to NF EN 1992-3" (July 2008)

Eurocode 0 "Basis of structural design"

NF EN 1990 "Basis of structural design" (March 2003)

NF EN 1990/A1 "Amendment 1" (July 2006)

NF EN 1990/NA "National Annex to NF EN 1990" (June 2004)

NF EN 1990/A1/NA "National Annex to NF EN 1990/A1" (December 2007)

Eurocode 1 "Actions on structures"

NF EN 1991-1-1 "General actions — Densities, self-weight and imposed loads for buildings" (March 2003) incorporating the 1st corrigendum (November 2009)

NF EN 1991-1-1/NA "National Annex to NF EN 1991-1-1" (June 2004)

NF EN 1991-1-2 "General actions – Actions on structures exposed to fire" (July 2003) incorporating the 1st corrigendum (August 2009) and the 2nd corrigendum (December 2012)

NF EN 1991-1-2/NA "National Annex to NF EN 1991-1-2" (February 2007)

NF EN 1991-1-3 "General actions - Snow loads" (April 2004) incorporating the 1st corrigendum (October 2009)

NF EN 1991-1-3/NA "National Annex to NF EN 1991-1-3" (May 2007)

NF EN 1991-1-3/NA/A1 "Amendment to the National Annex" (July 2011)

NF EN 1991-1-4 "General actions - Wind actions" (November 2005) incorporating the 1st corrigendum (May 2010) and the 2nd corrigendum (September 2010)

NF EN 1991-1-4/NA "National Annex to NF EN 1991-1-4" (March 2008)

NF EN 1991-1-4/NA/A1 "Amendment to the National Annex" (July 2011)

NF EN 1991-1-4/NA/A2 "Amendment 2 to the National Annex" (September 2012)

NF EN 1991-1-5 "General actions - Thermal actions" (May 2004) incorporating the 1st corrigendum (October 2010)

NF EN 1991/NA "National Annex to NF EN 05/01/1991" (February 2008)

NF EN 1991-1-6 "General actions - Actions during execution" (November 2005) incorporating the 1st corrigendum (February 2009) and the 2nd corrigendum (December 2012)

NF EN 1991-1-6/NA "National Annex to NF EN 1991-1-6" (March 2009)

NF EN 1991-1-7 "General accidents - Accidental actions" (February 2007) incorporating the 1st corrigendum (March 2011)

NF EN 1991-1-7/NA "National Annex NF EN 1991-1-7" (September 2008)

NF EN 1991-2 “Traffic loads on bridges” (March 2004) incorporating the 1st corrigendum (June 2010)

NF EN 1991-2/NA "National Annex to NF EN 1991-2" (March 2008)

NF EN 1991-3 "Actions induced by cranes and machinery" (April 2007) incorporating the 1st corrigendum (January 2013)

NF EN 1991-3/NA "National Annex to NF EN 1991-3" (January 2010)

NF EN 1991-4 "Silos and tanks" (May 2007) incorporating the 1st corrigendum (December 2012)

NF EN 1991-4/NA "National Annex to NF EN 1991-4" (November 2007)

Eurocode 7 "Geotechnical design" and its national application standards

NF EN 1997-1 "General rules" (June 2005) incorporating the 1st corrigendum (August 2011)

NF EN 1997-1/NA "National Annex to NF EN 1997-1" (September 2006)

NF P94-261 "National application standards for Eurocode 7 - Shallow foundations (June 2013)

NF P94-262 "National application standards for Eurocode 7 - Deep foundations (July 2012)

NF P94-281 "Gravity wall design" (April 2014)

UHPFRC standards

NF P18-470 “UHPFRC: specification, performance, production and conformity"

NF P18-451 “Execution of concrete structures – Specific Rules for UHPFRCs"[4]

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[4] In preparation

(4) The UHPFRCs covered by this standard are described as UHPFRC-S in standard NF P18-470.

NOTE These UHPFRCs therefore demonstrate the following mechanical properties:

- they are type M UHPFRCs, meaning that the fibres giving a strain-hardening behaviour on bending are metal fibres
- characteristic compression strength f_{ck} between 150 MPa and 250 MPa
- characteristic tensile strength $f_{ctk,el}$ greater than 6.0 MPa
- sufficiently ductile behaviour under tension in order that they satisfy the following inequality:

$$\frac{1}{w_{0.3}} \int_0^{w_{0.3}} \frac{\sigma(w)}{1.25} dw \geq \max(0.4 f_{ctm,el}; 3\,\text{MPa})$$

where $w_{0.3}$ = 0.3 mm

$f_{ctm,el}$ is the mean value of the tensile limit of elasticity

$\sigma(w)$ is the characteristic post-cracking stress as a function of the crack width w.

- density should be between 2300 and 2800 kg/m^3

1.2 Normative references

(1)P The following standards documents contain provisions which, when referred to in the present text, constitute provisions in this standard. For dated references, later amendments or revisions to one of these publications do not apply to this standard. The parties working on contracts based on the present standard are however encouraged to look at the possibility of applying the most recent editions of the standard documents given below. For undated references, the latest edition of the publication referred to applies.

(2) Reference standards:

Eurocode 2[5]

NF P18-470 "UHPFRC: Specifications, performance, production and conformity"

NF P 18-451 "Execution of concrete structures – Specific Rules for UHPFRCs"[6]

1.3 Assumptions

(1)P Besides the general assumptions from EN 1990, the following assumptions apply:

- The structures are designed and calculated by individuals with the required qualifications and experience.
- Adequate supervision and quality control is required in the plants, workshops and on the site.
- Construction is carried out by personnel with the required skills and experience.
- The construction materials and products are used in the manner specified in the present standard or according to the specifications specific to the materials or products employed. In particular, the UHPFRC is a UHPFRC of type S compliant with standard NF P 18-470.
- The structure shall be subject to adequate maintenance.
- The use of the structure shall comply with design specification.
- The execution and placement requirements given in standard NF P18-451 shall be complied with.

1.4 Distinction between Principles and Application Rules

(1)P Unchanged

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5 The precise references of the standards forming Eurocode 2 are given in part 1.1 of the present standard.

6 In preparation

1.5 Definitions

UHPFRC: the UHPFRCs covered by this standard are described as UHPFRC-S in standard NF P18-470. They are concretes with high compressive strength, and high post-cracking tensile strength giving it a ductile behaviour in tension, whose lack of brittleness makes it possible to design and produce structures and structural members without using reinforcing steel. To construct certain structures, the UHPFRC may nevertheless contain reinforcement (this is then called reinforced UHPFRC) or prestressing tendons (prestressed UHPFRC).

Thin members/thick members: a thin member is one where the thickness e is such that $e \leq 3\ L_f$ where L_f is the length of the longest fibres contributing to non-brittleness. Other members are considered to be thick members.

1.6 Symbols

The following notation is used in the present standard. Notation which is non-specific to UHPFRCs from the Eurocodes is in general omitted from the following list.

e: thickness of the member

c_{min}: minimum cover

$c_{min,p}$: minimum cover due to placement conditions of UHPFRC

e_h: horizontal clear spacing distance between bars

e_v: vertical clear spacing distance between bars

f_c: compressive strength

f_{ck}: characteristic value of compressive strength

f_{cm}: mean value of compressive strength

f_{cd}: design value of compressive strength

$f_{ct,el}$: tensile limit of elasticity

$f_{ctk,el}$: characteristic value of the tensile limit of elasticity

$f_{ctm,el}$: mean value of the tensile limit of elasticity

f_{ctf}: post-cracking tensile strength

f_{ctfk}: characteristic value of the post-cracking strength

f_{ctfm}: mean value of the post-cracking strength

$f_{ctf,1\%}$: post-cracking strength corresponding to a crack width of 0.01H where H is the height of the tested prism

$f_{ctf,1\%,k}$: characteristic post-cracking strength corresponding to a crack width of 0.01H where H is the height of the tested prism

$f_{ctf,1\%,m}$: mean post-cracking strength corresponding to a crack width of 0.01H where H is the height of the bending test prism

$f_{ct,fl}$: limit of elasticity directly derived from the 4-point bending test

h_{fs}: height of the part of a T-section operating in a shear mode

L_c: characteristic length which relates the crack width to an equivalent deformation

L_f: length of the longest fibres contributing to ensuring non-brittleness. If the UHPFRC contains a single type of fibre, L_f is the length of these fibres.

w: crack width

w_s: crack width at depth of passive reinforcement

w_t: crack width on the most tensile zone

x: neutral axis depth

x': height of non-cracked zone under tension

D_{sup}: maximum aggregate size in the UHPFRC (see 5.4.3 of standard NF P18-470)

E_{cm}: mean value of Young's modulus

$E_{c,eff}$: effective Young's modulus

K: orientation factor expressing the mechanical effect of the orientation of the fibres on the post-cracking behaviour under tension

K_{global}: orientation factor associated with global effects

K_{local}: orientation factor associated with local effects

δ**:** factor expressing the reduction in the reinforcement anchorage length due to the fibres

ε_{c0d}**:** maximum design elastic shortening strain at ULS

ε_{cud}**:** maximum design shortening strain at ULS

$\varepsilon_{u,el}$**:** elastic tensile strain at the ultimate limit state

$\varepsilon_{u,lim}$**:** tensile strain limit beyond which the participation of the fibres is no longer taken into account at the ultimate limit state

$\varepsilon_{u,pic}$ **:** equivalent ULS strain corresponding to the peak post-cracking stress or to a crack width equal to 0.3 mm if there is no post-cracking peak

$\varepsilon_{u,1\%}$**:** equivalent strain corresponding to a crack width equal to 0.01H where H is the height of the tested prism associated with the dimensions of the structure at the ULS

γ_{cf}**:** partial factor for UHPFRC under tension

θ**:** struts angle

$\sigma_f(w)$**:** stress law as a function of crack width

$\sigma_{Rd,f}$: mean value of the post-cracking strength along the shear crack at an angle θ

2 BASIS OF DESIGN

2.1 Requirements

2.1.1 Basic requirements

(1)P Unchanged

(2)P Unchanged

(3) Unchanged

2.1.2 Reliability management

(1) Unchanged

(2) Unchanged

2.1.3 Design working life, durability and quality management

(1) Unchanged

2.2 Principles of limit state design

(1) Unchanged

2.3 Basic variables

2.3.1 Actions and environmental influences

2.3.1.1 General

(1) Unchanged

2.3.1.2 Thermal effects

(1) Unchanged

(2) Unchanged

(3) Unchanged

2.3.1.3 Differential settlements/movements

(1) Unchanged

(2) Unchanged

(3) Unchanged

(4) Unchanged

2.3.1.4 Prestress

(1)P Unchanged

(2) Unchanged

(3) Unchanged

(4) Unchanged

2.3.2 Material and product properties

2.3.2.1 General

(1) Unchanged

(2) Unchanged

2.3.2.2 Shrinkage and creep

(1) Unchanged

(2) Unchanged

(3) Unchanged

2.3.3 Deformations of concrete

(1)P Unchanged

(2) Obeying the Application Rules in this Standard normally enables these effects to be taken into account. In addition:

— pay particular attention to the deformations and the cracking risks due to the change in the UHPFRC at early age and on creep and shrinkage;
— minimise restraining effects on deformations by the use of appropriate support devices or joints;
— ensure the influence of restrained deformations is included in the design.

NOTE: For UHPFRCs, early shrinkage phenomena (autogenous and possibly thermal) are greater than for traditional concretes and not taking them into account correctly may be detrimental. Special attention must therefore be paid in particular for slender parts, parts with significant variations in thickness, and for clamped parts.

(3) For buildings, the effects of temperature and shrinkage may be neglected in the overall analysis for precast members in UHPFRC subject to joints, spaced by d_{joint}, being incorporated in order to take up resulting deformations.

The supplement to the National Annex, which in particular gives the value of d_{joint} applicable to the French regions, applies.

2.3.4 Geometric data

2.3.4.1 General

(1) Unchanged

2.3.4.2 Supplementary requirements for cast in place piles

(1)P Does not apply

(2) Does not apply

2.4 Verification by the partial factor method

2.4.1 General

(1) Unchanged

2.4.2 Design values

2.4.2.1 Partial factor for shrinkage action

(1) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

2.4.2.2 Partial factors for prestress

(1) Unchanged

(2) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(3) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

2.4.2.3 Partial factor for fatigue loads

(1) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

2.4.2.4 Partial factors for materials

(1) The material-related partial factors γ_C and γ_{cf} should be used for the UHPFRC and γ_S for the reinforcement defined for the ultimate limit states.

The values of γ_C, γ_{cf} and γ_S are given in Table 2.201. They are not valid for fire design for which reference should be made to Annex R.

For fatigue verification, the partial factors for durable design situations given in Table 2.201 should be used for $\gamma_{C,fat}$, $\gamma_{cf,fat}$ and $\gamma_{S,fat}$.

Table 2.201 — Partial factors for materials at ultimate limit states

Design situations	γ_C (compressed UHPFRC)	γ_{cf} (tensioned UHPFRC)	γ_S (reinforcing steel)	γ_S (prestressing steel)
Durable Transient	1.5	1.3	1.15	1.15
Accidental	1.2	1.05	1.0	1.0

The partial factor γ_C for UHPFRCs produced from a premix of constituents may be reduced to 1.3 in a durable and transient design situation and to 1.05 in an accidental design situation.

This reduction may only be made for premixes defined in standard NF P18-470 and subject to an internal production check by the premix supplier. The detail of these checks is given in Annex G of standard NF P18-470. It is not possible for type TT1 or TT1+2 UHPFRCs, where the heat treatment applied to the UHPFRC before setting has a significant effect (according to 5.4.8 of standard NF P18-470), for which values of γ_C of 1.5 in a durable and transient situation and 1.2 in an accidental design situation must be maintained.

Furthermore for type TT1 or TT1+2 UHPFRCs, where the heat treatment applied to the UHPFRC before setting has a significant effect (according to 5.4.8 of standard NF P18-470), the partial tensile coefficient γ_{cf} must be increased for 1.4 in a durable and transient design situation and to 1.1 in an accidental design situation whether or not they are produced from a premix of constituents.

(2) With regard to partial factors for materials, the following values must be used for checks at serviceability limit states: $\gamma_C = \gamma_{cf} = \gamma_S = 1.0$

(3) Unchanged

2.4.2.5 Partial factors for materials for foundations

(1) Unchanged

(2) Does not apply

2.4.3 Combinations of actions

(1) Unchanged

(2) Unchanged

2.4.4 Verification of static equilibrium - EQU

(1) Unchanged

2.5 Design assisted by testing

(1) Unchanged

2.6 Supplementary requirements for foundations

(1)P Unchanged

(2) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(3) Does not apply

(4) Unchanged

2.7 Requirements for fastenings

(1) Both local and overall effects of fasteners must be considered taking account if necessary of any special features associated with the characteristics of the UHPFRCs.

3 MATERIALS

3.1 UHPFRC

3.1.1 General

(1)P The following sections give the principles applicable to UHPFRCs covered by the present standard (see 1.1 (4)).

(2) Does not apply

(3) Addition: The mechanical characteristics of the UHPFRC material, compression strength, tensile behaviour and Young's modulus along with the shrinkage and creep laws must be assessed in compliance with standard NF P18-470.

The execution design structure in UHPFRC must be produced from the characteristics of the UHPFRC used, validated in compliance with standard NF P18-470 at the suitability test stage. As explained in standard NF P18-470, this UHPFRC is subject to a full characterisation with regard to the properties required for the design, which may be determined by use of an identity card.

For preliminary or design studies, and in the absence of tests or an identity card, the values given in Annex T may be used.

3.1.2 Strength

(1)P The compressive strength of UHPFRC is designated by strength classes associated with the characteristic strength (5% fractile) measured on cylinder f_{ck}, in accordance with 5.5.2 of standard NF P18-470.

(2)P Does not apply

(3) Does not apply

(4) Does not apply

(5) The compressive strength of the UHPFRC at age t depends on any heat treatment it has undergone.

If the UHPFRC has not undergone heat treatment (STT) or is classified TT1 (see 4.3.3 of standard NF P18-470):

- if $t \geq 28$ days, the compressive strength to be used is f_{ck}
- if $t < 28$ days, the compressive strength must be specified and assessed in accordance with article 5.5.2 of standard NF P18-470.

If the UHPFRC is classified TT2 or TT1+2 (see 4.3.3 of standard NF P18-470):

- if date t is after the end of the heat treatment, the compressive strength to be used is f_{ck}
- if date t is prior to the end of heat treatment, the compressive strength must be specified and assessed in accordance with article 5.5.2 of standard NF P18-470

In an analogous way, the reference age for determining the design characteristics of the hardened UHPFRC is 28 days for UHPFRCs of type STT or TT1 and after the application of heat treatment for UHPFRCs of type TT2 or TT1+2.

(6) Does not apply

(7) The tensile behaviour characteristics of the UHPFRC are given in 3.1.7.3.

(8) Does not apply

(9) Does not apply

3.1.3 Elastic deformation

(1) The elastic deformations of UHPFRC mostly depend on its composition.

(2) The value of Young's modulus E_{cm} must be determined in accordance with 5.5.8 of standard NF P18-470.

The value of Young's modulus to be considered for the design of sections under tension is the same as that for compression.

NOTE For preliminary or design studies, and in the absence of tests or an identity card, a value for Young's modulus is proposed in Annex T.

(3) Does not apply

(4) Poisson's coefficient may be taken as 0.2 for UHPFRCs.

(5) The linear coefficient of thermal expansion for the UHPFRC must come from design tests or from the identity card for the material, in particular when the structure is sensitive to thermal deformations, whether the deformations may be prevented when the structure is new, or if it is subject to fire risk. Standard NF P18-470 gives the information necessary for determining the value of the coefficient of thermal expansion.

NOTE For preliminary or design studies, and in the absence of tests or an identity card, a value for the coefficient of linear thermal expansion is proposed in Annex T.

3.1.4 Creep and shrinkage

(1)P Unchanged

(2) Does not apply

(3) Does not apply

(4) Does not apply

(5) Addition: With regard to shrinkage, a final value or a complete evolution curve from the design tests or the identity card for the UHPFRC must be used, in particular when the structure is sensitive to instantaneous or delayed deformations and to their structural effects, or may be subject to restrained deformations at early age. Standard NF P18-470 gives the information necessary for determining the final value of the shrinkage and its evolution.

With regard to creep, a final value or a complete evolution curve of creep deformations from the design tests or the identity card for the UHPFRC must be used, in particular when the structure is sensitive to instantaneous or delayed deformations and to their structural effects, or may be subject to prestressing losses. The sensitivity of the structure may be evaluated by upper/lower bound assessments. Standard NF P18-470 gives the information necessary for the experimental determination of creep.

NOTE The evolution of shrinkage and creep may be described using the models in Annex B of standard NF EN 1992-2, through a calibration of the amplitudes and coefficients linked to the kinetics under section B.104 of NF EN 1992-2.

For preliminary or design studies, and in the absence of test results or an identity card, values for carrying out shrinkage and creep calculations are proposed in Annex T.

3.1.5 Stress-strain relation for non-linear structural analysis

(1) The stress-strain relation including the post-peak section for non-linear structural analysis (average law of compression behaviour) is defined by the following expressions, which take account of the confinement effect brought by the fibres through the post-cracking strength f_{ctfm} / K_{global} (K_{global} being normally associated to the radial transverse direction):

$$\sigma = f_{cm} \frac{\eta . \dfrac{\varepsilon}{\varepsilon_{c1,f}}}{\eta - 1 + \left(\dfrac{\varepsilon}{\varepsilon_{c1,f}} \right)^{\varphi . \eta}} \tag{3.201}$$

Expression in which:

$$\varepsilon_{c1,f} = \left[1+4\frac{f_{ctfm}}{K_{global}.f_{cm}}\right]\left[1+0,16\frac{k_0}{\left(f_{cm}^2+800\right)}\right]\frac{f_{cm}^{2/3}}{k_0} \tag{3.202}$$

f_{cm} being expressed in MPa and where:

$$k_0 = \frac{E_{cm}}{f_{cm}^{1/3}} \tag{3.203}$$

$$\eta = \frac{k}{k-1} \tag{3. 204}$$

where:

$$k = E_{cm}\frac{\varepsilon_{c1,f}}{f_{cm}} \tag{3. 205}$$

$$\varphi = \begin{cases} 1 \quad \text{si} \quad \varepsilon \le \varepsilon_{c1,f} \\ \dfrac{\ln\left(1-\eta+\dfrac{\eta}{0,7}\dfrac{\varepsilon_{cu1,f}}{\varepsilon_{c1,f}}\right)}{\eta.\ln\left(\dfrac{\varepsilon_{cu1,f}}{\varepsilon_{c1,f}}\right)} \quad \text{si} \;\; \varepsilon > \varepsilon_{c1,f} \end{cases} \tag{3. 206}$$

$$\varepsilon_{cu1,f} = \left[1+15\frac{f_{ctfm}}{K_{global}.f_{cm}}\right]\left[1+\frac{20}{f_{cm}}\right]\left[1+0,16\frac{k_0}{\left(f_{cm}^2+800\right)}\right]\frac{f_{cm}^{2/3}}{k_0} \tag{3.207}$$

In thin sections or parts of sections, the preferred orientation of the fibres parallel to the walls does not allow accounting for confinement effect. f_{ctfm} must be taken as 0 in the previous expressions.

(2) A schematic representation of the stress-strain relation under compression for non-linear structural analysis is given in Figure 3.201:

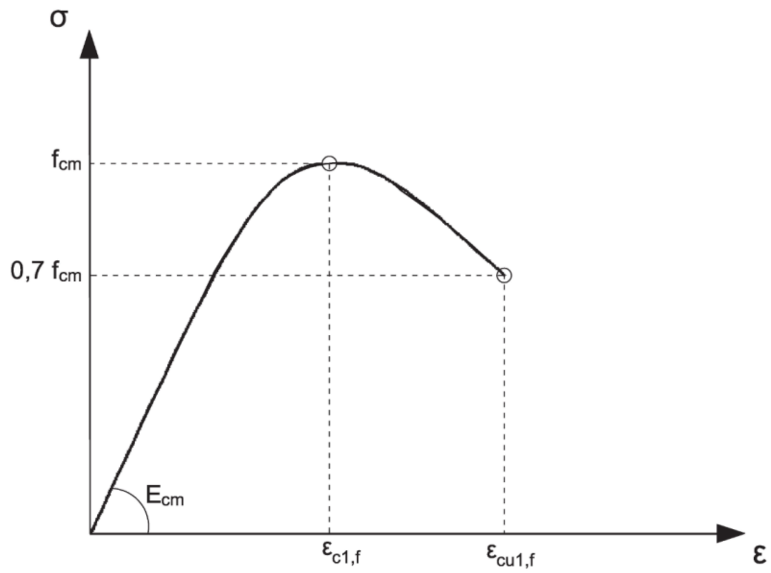


Figure 3.201 — Representation of the stress-strain relation of UHPFRC in compression for non-linear structural analysis

The deformation is limited to $\varepsilon_{cu1,f}$.

3.1.6 Design compressive and tensile strengths

(1)P The design compressive strength is defined as:

$$f_{cd} = \alpha_{cc} f_{ck} / \gamma_C \qquad (3.15)$$

where:

γ_C is the partial factor relating to compressed UHPFRC, see 2.4.2.4

α_{cc} is a coefficient which takes account of long-term effects on compressive strength and adverse effects resulting from the way the load is applied. The value to be selected for UHPFRCs is $\alpha_{cc} = 0.85$.

(2)P Does not apply

3.1.7 Stress-strain relations for the design of cross-sections

3.1.7.1 General

(1) Part 3.1.7 of the present standard fully replaces part 3.1.7 of standard NF EN 1992-1-1.

3.1.7.2 Constitutive law for UHPFRC in compression

(1) The constitutive law for UHPFRC in compression to be used for designing sections at ULS is as follows:

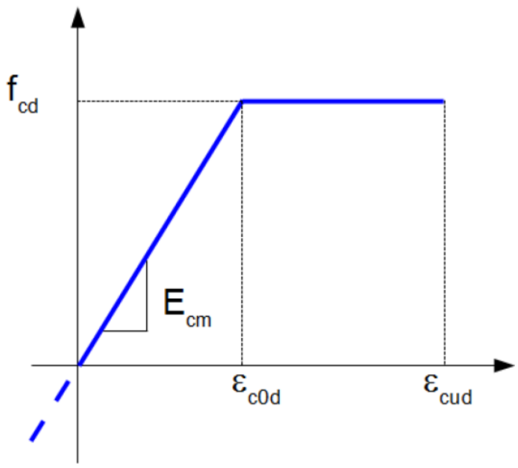


Figure 3.202 — Representation of the stress-strain relation of UHPFRC in compression for designs at ULS

The strain ε_{c0d} is defined by the following relationship:

$$\varepsilon_{c0d} = f_{cd}/E_{cm} \tag{3.9}$$

The ultimate strain to be taken into account at ULS is given by the following formula:

$$\varepsilon_{cud} = (1 + 14\frac{f_{ctm}}{K_{global}.f_{cm}}).\varepsilon_{c0d} \tag{3.208}$$

where:

f_{ctfm} is the mean value of the post-cracking tensile strength (see 5.5.4 of standard NF P18-470 for its determination)
K_{global} is the orientation factor associated with the global effects (see 4.4.3 of standard NF P18-470 for its determination)
f_{cm} is the mean value of the compression strength (see 5.5.2 of standard NF P18-470 for its determination).

3.1.7.3 Tensile strength

3.1.7.3.1 General

(1) The tensile behaviour is characterised by the tensile limit of elasticity and by the stress-crack width σ(w) or stress-strain σ(ε) post-cracking constitutive law. This law may be modulated according to the particular directions of the tensile force.

NOTE The various directions to be considered may be, for example, a longitudinal direction for the bending resistance, a direction perpendicular to that of the compression bars for shear resistance, a transverse radial direction for the confinement effect of compressed UHPFRC, a transverse direction around the circumference for the crack-bridging effect with regard to the splitting of the UHPFRC at anchorages or bar laps and concentrated force diffusion areas.

Inclusion in this way is possible for designing sections at ULS and SLS.

(2) Regarding tensile behaviour, reference should be made to article 5.5.4 of standard NF P18-470. There are two possible approaches:

- either a point-by-point law is chosen which comes directly from testing by possibly selecting a simplified description of this piecewise linear law

- or a conventional law described in the following sections is chosen, but for which the parameters are determined from test results

The execution specifications of the structure may use one or other of these approaches.

NOTE For preliminary or design studies, and in the absence of tests or an identity card, it is possible to use the conventional law given in Annex T.

(3) The orientation of the fibres is an important parameter of the tensile design law and is expressed by a set of orientation factors K. It must be determined in accordance with Annex F of standard NF P18-470.

(4) The characteristic value of the tensile limit of elasticity is denoted $f_{ctk,el}$ and its mean value $f_{ctm,el}$.

The characteristic value of the post-cracking strength is denoted f_{ctfk} and its mean value f_{ctfm}. This strength does not incorporate the orientation factor K.

The post-cracking strength is determined from experimental curves in accordance with standard NF P18-470.

It is equal to the maximum measured if it is greater than the limit of elasticity, otherwise there are two possible scenarios:

- if a local maximum is observed, f_{ctf} relates to the local maximum, as shown in Figure 3.203 below:

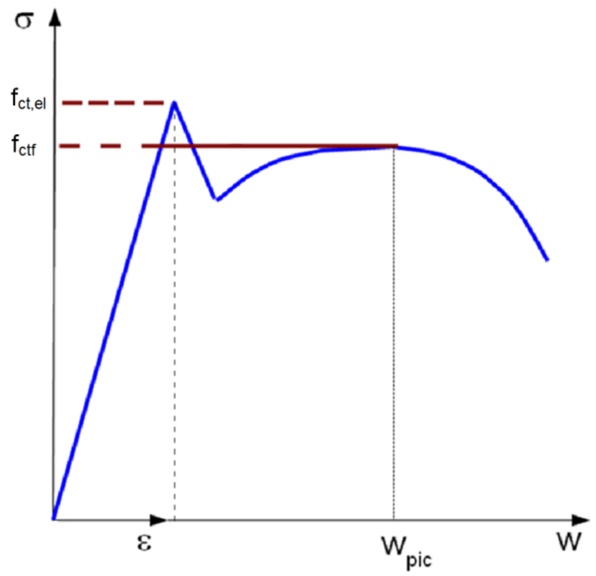


Figure 3.203 — Definition of f_{ctf} in the case of a local maximum

- if there is no local peak, f_{ctfk} is the stress associated to a crack width of 0.3 mm, as shown in the diagram below:

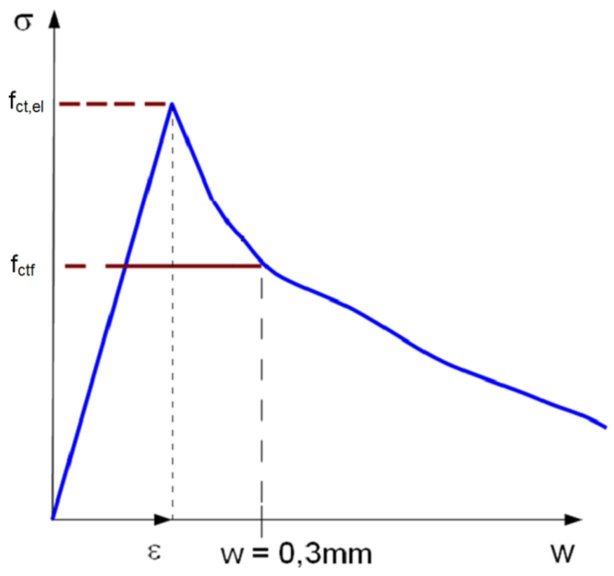


Figure 3.204 — Definition of f_{ctf} in the case where there is no local maximum

(5) The UHPFRCs covered by this standard have a tensile behaviour which must be specified according to section 5.5.4 of standard NF P18-470. For the calculation, the so-called "design" tensile behaviour classes are defined as follows:

- class T1* (softening under direct tension) when $f_{ctf}/K < f_{ct,el}$ both for the mean curve and for the characteristic curve, i.e. $f_{ctfm}/K < f_{ctm,el}$ and $f_{ctfk}/K < f_{ctk,el}$

- class T2* (exhibiting limited strain hardening) when $f_{ctf}/K \geq f_{ct,el}$ for the mean curve and $f_{ctf}/K < f_{ct,el}$ for the characteristic curve, i.e. $f_{ctfm}/K \geq f_{ctm,el}$ and $f_{ctfk}/K < f_{ctk,el}$

- class T3* (exhibiting significant strain hardening) when $f_{ctf}/K \geq f_{ct,el}$ both for the mean curve and the characteristic curve, i.e. $f_{ctfm}/K \geq f_{ctm,el}$ and $f_{ctfk}/K \geq f_{ctk,el}$

NOTE 1 If the orientation factor K is equal to 1.25, the tensile behaviour classes T1, T2 and T3 according to standard NF P 18-470 and the design tensile behaviour classes T1*, T2* and T3* coincide.

NOTE 2 As the orientation factor K may differ according to the directions of stress considered, the design tensile behaviour of a UHPFRC may come under different classes according to the directions of stress considered.

(6) When $f_{ctfk}/(K.\gamma_{cf}) < f_{ctk,el} / \gamma_{cf}$, the ULS law must be clipped meaning it must show a horizontal plateau equating to a stress of $f_{ctfk}/(K.\gamma_{cf})$.

When $f_{ctfk}/K < f_{ctk,el}$, the SLS law must also be clipped at a horizontal plateau equal to f_{ctfk}/K.

When $f_{ctfk}/(K.\gamma_{cf}) > f_{ctk,el} / \gamma_{cf}$, the ULS law is constructed by connecting the point corresponding to $f_{ctk,el}$ to that corresponding to $f_{ctfk}/(K.\gamma_{cf})$ by a line, and the stress is taken as null after the point corresponding to $f_{ctfk}/(K.\gamma_{cf})$.

When $f_{ctfk}/K > f_{ctk,el}$, the SLS law is constructed by connecting the point corresponding to $f_{ctk,el}$ to that corresponding to f_{ctfk}/K by a line, and the stress is taken as null after the point corresponding to f_{ctfk}/K.

(7) Regarding the design law at the SLSs, the tensile strain limit ε_{lim} is the strain beyond which the design tensile stress of the UHPFRC is null. The elastic limit strain is denoted ε_{el} and is equal to $f_{ctk,el}/E_{cm}$.

Regarding the design law at the ULSs, the tensile strain limit $\varepsilon_{u,lim}$ is the strain beyond which the design tensile stress of the UHPFRC is null. The elastic limit strain is denoted $\varepsilon_{u,el}$ and is equal to $f_{ctk,el}/(\gamma_{cf}.E_{cm})$.

(8) The tensile laws differ according to whether the member considered is thick or thin. A thin member is a member whose thickness e is such that:

$e \leq 3\ L_f$ where L_f = length of the longest fibres contributing to ensuring non-brittleness.

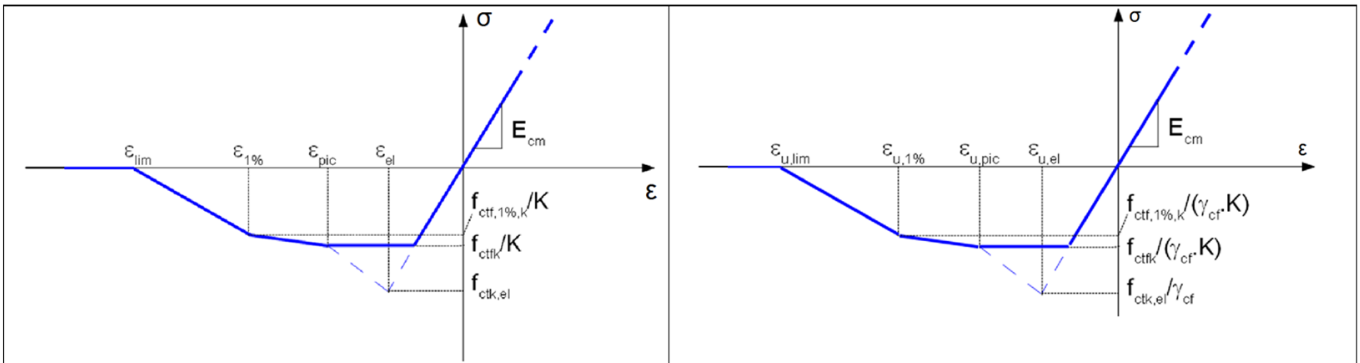
Other members are considered to be thick members.

The experimental determination of laws differs between thin and thick members (see 4.4.3 of standard NF P18-470).

(9) To apply certain specifications at the SLSs, design laws must be interpreted "using mean values", namely f_{ctfk} is to be replaced by f_{ctfm} and $f_{ctk,el}$ by $f_{ctm,el}$.

3.1.7.3.2 Conventional laws for thick members

(1) The conventional law which may be used for designing thick sections is as follows:



Design law at SLS	Design law at ULS

Figure 3.205 — Conventional law for UHPFRCs of class T1* or T2* (from σ(w) curve)

The parameters given in these design laws are:

At SLS:

$$\varepsilon_{pic} = \frac{w_{pic}}{L_c} + \frac{f_{ctk,el}}{E_{cm}} \tag{3.209}$$

At ULS:

$$\varepsilon_{u,pic} = \frac{w_{pic}}{L_c} + \frac{f_{ctk,el}}{\gamma_{cf} E_{cm}} \tag{3.210}$$

where w_{pic} is the crack width corresponding to the local peak in the curve from the tests carried out in accordance with Annex D of standard NF P18-470 or is equal to 0.3 mm if there is no peak.

At SLS:

$$\varepsilon_{1\%} = \frac{w_{1\%}}{L_c} + \frac{f_{ctk,el}}{E_{cm}} \tag{3.211}$$

At ULS:

$$\varepsilon_{u1\%} = \frac{w_{1\%}}{L_c} + \frac{f_{ctk,el}}{\gamma_{cf} E_{cm}} \tag{3.212}$$

where $w_{1\%} = 0.01H$, H being the height of the bending test prism (see Annex D of standard NF P18-470)

$\varepsilon_{u,lim} = \varepsilon_{lim} = \dfrac{L_f}{4L_c}$, L_f being the length of the longest fibres contributing to ensuring non-brittleness

$L_c = \dfrac{2}{3}h$ characteristic length (where h is the height of the section).

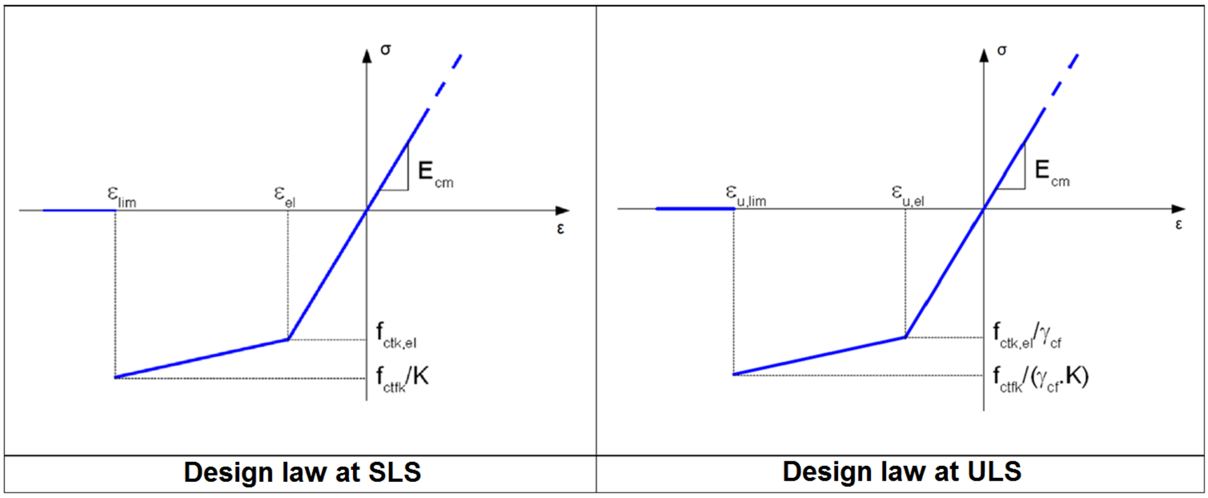


Figure 3.206 — Law for UHPFRCs of class T3* (obtained directly in σ(ε))

3.1.7.3.3 Conventional laws for thin members

(1) The conventional laws which may be used for designing thin sections are as follows:

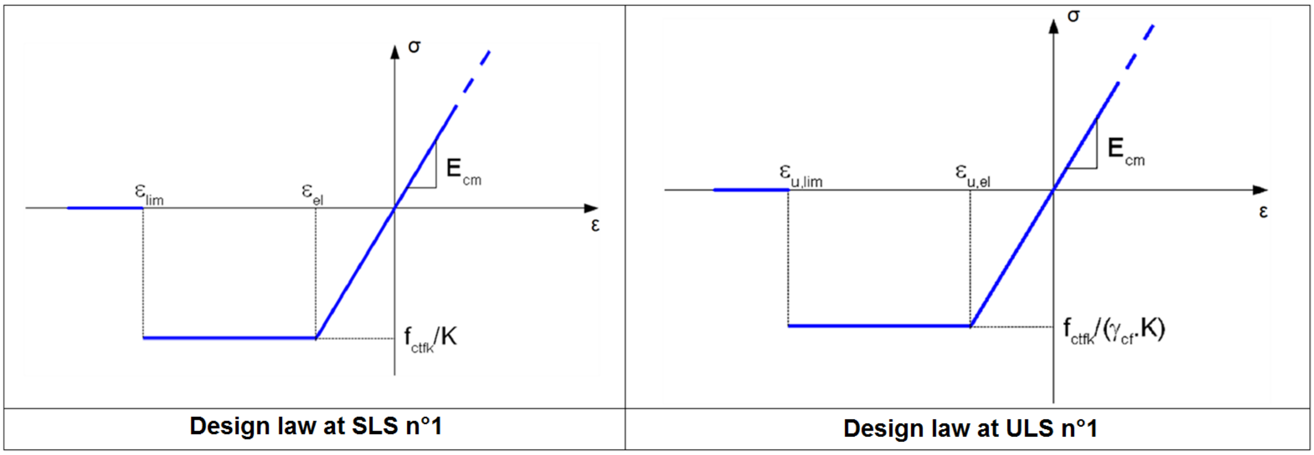


Figure 3.207 — Conventional law n°. 1 for thin members

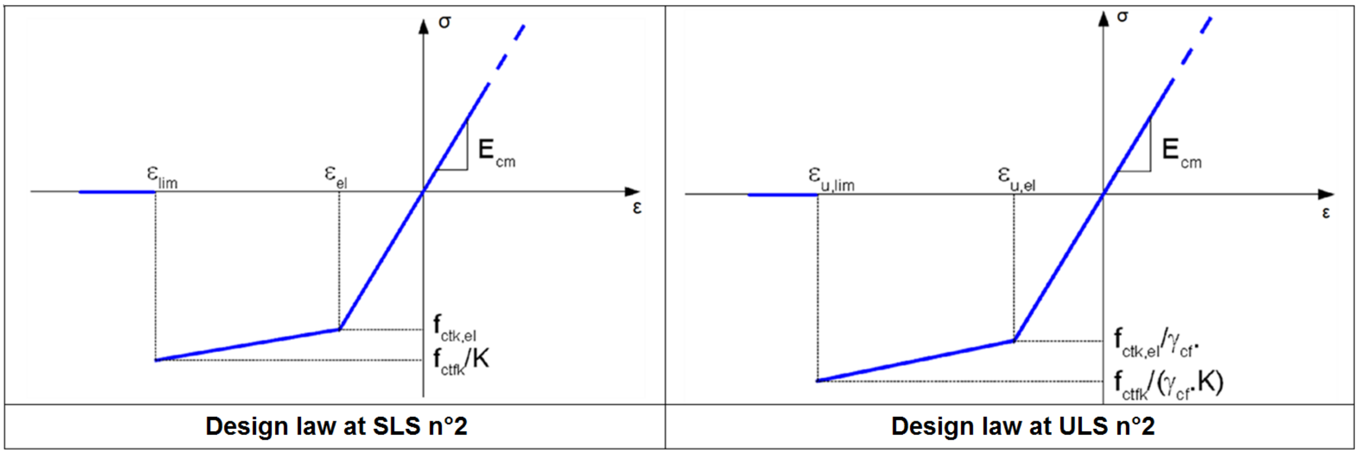


Figure 3.208 — Conventional law n°. 2 for thin members

Law 1 may only be used for members subject to simple bending or to bending-compression. Law 2 may be used whatever the type of stress applied, provided that the material is of class T3*.

In the laws 1 and 2 described above, $\varepsilon_{u,lim}$ (= ε_{lim}) is the maximum tensile strain resulting from the characterisation tests described in Annex E of standard NF P18-470.

3.1.7.3.4 Accounting for the fibre distribution

(1) To cover the difference in fibre orientation due to UHPFRC placement, the various justifications are based on a design law modulated considering an orientation factor K which takes the value K_{global} or K_{local}.

(2) K_{local} is dedicated to local effects corresponding to resistant mechanisms which require fibre contribution in very localised areas (for example the diffusion of prestressing forces).

K_{global} deals with the global effects corresponding to resistant mechanisms which require the fibres to act over wider areas and where a localised fault will not have significant consequences (for example shear, bending resistance of a slab).

The choice between K_{local} and K_{global} is specified in each check to be performed.

(3) The orientation factor K is established on the basis of tests on a representative model of the actual structure in accordance with Annex F of standard NF P18-470.

NOTE For preliminary or design studies, and in the absence of tests on a model representing the actual structure, values for the orientation factor K are proposed in Annex T.

(4) The orientation factor K may differ according to the directions of stress and parts of the structure considered. In the various checks involving this factor, its determination must relate to the direction perpendicular to the potential cracking plane in the resistant mechanism considered. Failing a direct experimental determination in this direction, the maximum of the K values determined in the two perpendicular directions will be used.

3.1.8 Flexural tensile strength

(1) Does not apply as dealt with in 3.1.7.

3.1.9 Confined concrete

(1) Does not apply

(2) Does not apply

3.2 Reinforcing steel

3.2.1 General

(1)P Unchanged

(2)P Unchanged

(3)P Unchanged

(4)P Unchanged

(5) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

3.2.2 Properties

(1)P Unchanged

(2)P Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(3)P Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(4)P Unchanged

(5) Unchanged

(6)P Unchanged

3.2.3 Strength

(1)P Unchanged

3.2.4 Ductility characteristics

(1)P Unchanged, including (101)P of standard NF EN 1992-2 and its National Annex

(2) Unchanged

3.2.5 Welding

(1)P Unchanged

(2)P Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(3)P Unchanged

(4) Unchanged

3.2.6 Fatigue

(1)P Unchanged

3.2.7 Design assumptions

(1) Unchanged

(2) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(3) Unchanged

(4) Unchanged

3.3 Prestressing steel

3.3.1 General

(1)P Unchanged

(2)P Unchanged

(3) Unchanged

(4) Unchanged

(5)P Unchanged

(6) Unchanged

(7)P Unchanged

(8)P Unchanged

(9)P Unchanged

(10)P Unchanged

(11)P Unchanged

3.3.2 Properties

(1)P Unchanged

(2)P Unchanged

(3)P Unchanged

(4)P Unchanged

(5) Unchanged

(6) Unchanged

(7) Unchanged

(8) Unchanged

(9) Unchanged

3.3.3 Strength

(1)P Unchanged

3.3.4 Ductility characteristics

(1)P Unchanged

(2) Unchanged

(3) Unchanged

(4) Unchanged

(5) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

3.3.5 Fatigue

(1)P Unchanged

(2)P Unchanged

3.3.6 Design assumptions

(1)P Unchanged

(2) Unchanged

(3) Unchanged

(4) Unchanged

(5) Unchanged

(6) Unchanged

(7) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

3.3.7 Prestressing tendons in sheaths

(1)P Unchanged

(2)P Unchanged

3.4 Prestressing devices

3.4.1 Anchorages and couplers

3.4.1.1 General

(1)P Unchanged

(2)P The anchorages and couplers in the prestressing system used are from a prestressing system which has a European technical assessment (ETA) in accordance with ETAG013 and may be adapted subject to an experimental validation in accordance with Annex S.

(3)P Unchanged

3.4.1.2 Mechanical properties

3.4.1.2.1 Anchored tendons

(1)P Unchanged

(2) Unchanged

3.4.1.2.2 Anchorage devices and anchorage zones

(1)P Unchanged

3.4.2 External non-bonded tendons

3.4.2.1 General

(1)P Unchanged

(2)P Unchanged

(3) Unchanged

3.4.2.2 Anchorages

(1) Unchanged

4 DURABILITY AND COVER TO REINFORCEMENT

4.1 General

(1)P Unchanged

(2)P Unchanged

(3)P Unchanged

(4) The prevention of corrosion of the fibres and of the active or passive reinforcement depends:

- on the compactness and the quality of the UHPFRC, in particular in the concrete cover area, obtained by compliance with the requirements associated with the exposure classes described in 5.3 of standard NF P 18-470;
- on the thickness of the concrete cover (see 4.4);
- on the crack control (see 7.3).

(5) Unchanged

(6) Unchanged

4.2 Environmental conditions

(1)P Unchanged

(2) The exposure classes of the UHPFRCs according to environmental conditions are explained in standard NF P18-470.

These are given in the following table as a reminder:

Table 4.1 — Exposure classes according to environmental conditions

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack. For concrete with reinforcement or embedded metal: very dry.	Concrete inside buildings with very low air humidity
2 Corrosion by carbonatation		
Where the reinforced concrete or concrete with embedded metallic parts is exposed to air and humidity, the exposure classes must be defined as follows:		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity; concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact; Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity; External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
Where the reinforced concrete or concrete with embedded metallic parts is subject to contact with water containing chlorides other than from marine sources, including ice clearance salts, the exposure classes must be defined as		

follows:		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools; Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides. Pavements; Car park slabs
4 Corrosion induced by chlorides from seawater		
Where the reinforced concrete or concrete with embedded metallic parts is subject to contact with seawater chlorides or the action of air carrying sea salt, the exposure classes must be defined as follows:		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5 Freeze/Thaw attack		
When the concrete is subject to a significant attack due to freezing-thawing cycles while it is damp, the exposure classes must be defined as follows:		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation with de-icing agent	Vertical concrete surfaces or road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents; Concrete surfaces exposed to direct spray containing de-icing agents and freezing; Splash zone of marine structures exposed to freezing
6 Chemical attack		
When the concrete is subject to chemical attack by soils and natural groundwater, the exposure classes must be defined as follows:		
XA1	Slightly aggressive chemical environment	Concrete exposed to natural soils and groundwater in accordance with Table 3 of NF P18-470
XA2	Moderately aggressive chemical environment	Concrete exposed to natural soils and groundwater in accordance with Table 3 of NF P18-470
XA3	Highly aggressive chemical environment	Concrete exposed to natural soils and groundwater in accordance with Table 3 of NF P18-470

Please note the following additional information:

- Type M UHPFRCs cannot come under exposure class X0.

- In the absence of special specifications, the following classifications must be made:

- XC1: parts of buildings sheltered from the rain, with the exception of parts classified as XC3;
- XC2: parts of buildings in contact with water over the long term;
- XC3: parts of buildings sheltered from the rain but not enclosed, or exposed to high levels of condensation both with regard to frequency and duration;
- XC4: superstructures of bridges, of civil engineering structures and of buildings not protected from the rain, including parts subjected to water routing and/or splashing.

- In the absence of special specifications, the following classifications must be made:

- XD1: moderately damp surfaces exposed to airborne chlorides;
- XD2: swimming pools or parts exposed to industrial water and containing chlorides;
- XD3: parts of structures subject to frequent and very frequent splashing and containing chlorides subject to the absence of a sealant coating protecting the UHPFRC.

- In the absence of special specifications, the following classifications must be made:

- XS1: structure members which are neither in contact with seawater nor exposed to spray, but which are directly exposed to saline air, either those located beyond the XS3 classification area and at least 1 km from the coast, sometimes more, up to 5 km depending on the specific topography;
- XS2: permanently immersed marine structures;
- XS3: marine structure members in tidal ranges and/or exposed to spray when they are located at least 100 m from the coast, sometimes more, up to 500 m depending on the specific topography.

- In the case of attack by freezing-thawing and unless specified otherwise particularly based on the state of saturation by long-term contact with liquid water (for example horizontal surface or otherwise), the exposure classes XF1, XF2, XF3 and XF4 are indicated in the map giving the freezing area (Figure NA2 of standard NF EN 206/CN) and in Table 4.201 below, with additional details from fascicle FD P 18-326. Salting is considered "infrequent" when the annual average of the number of days when salting takes place estimated over the last 10 years is less than 10, "very frequent" when it is greater than or equal to 30, and "frequent" between these two cases.

Table 4.201 — Exposure classes according to freezing and salting

Salting / **Freezing**	**None**	**Infrequent**	**Frequent**	**Very frequent**
Low or moderate	XF1	XF1	XF2	XF2*
Severe	XF3	XF3	XF4	XF4
*With the exception of concrete roadways and civil engineering structure members which are very exposed and which will be classified as XF4.				

Additional classes XH1, XH2 and XH3 from the "Recommendations for the prevention of damage due to delayed ettringite formation" published by the LCPC in August 2007 are introduced to describe the degree of water saturation in the immediate environment around the parts of the structure

NOTE The choice of exposure classes will be based on the DT003 guides prepared by the Ecole Française du Béton which may be downloaded from the site www.egfbtp.com:

"Guides for selecting exposure classes of cast in place or precast building structures"
"Guide for selecting exposure classes of concrete bridges"
"Guide for selecting exposure classes of maritime and fluvial structures"
"Guide for selecting exposure classes of road equipment structures"
"Guide for selecting exposure classes of excavated road tunnels"
"Guide for selecting exposure classes of covered trenches, galleries, caps and submerged caissons"
"Guide for selecting exposure classes of any civil engineering structures"

(3) In addition to the conditions in Table 4.1, particular forms of attack or indirect actions should be considered:

chemical attack due, for example, to:

- the use of the building or structure (storage of liquids etc.)
- acids or sulphates in solution
- chlorides contained in the concrete
- alkali-aggregate reactions

physical attack due, for example, to:

- temperature change
- abrasion
- water penetration

With regard to chemical and physical attacks, reference should be made to the appropriate parts of standard NF P18-470.

(104) of standard NF EN 1992-2: Unchanged

(105) of standard NF EN 1992-2: Unchanged, including what appears in the National Annex to standard NF EN 1992-2

(106) of standard NF EN 1992-2: Unchanged, including what appears in the National Annex to standard NF EN 1992-2

4.3 Requirements for durability

(1)P Unchanged

(2)P Unchanged

(103) of standard NF EN 1992-2: Unchanged

4.4 Methods of verification

4.4.1 Concrete cover

4.4.1.1 General

(1)P Unchanged

(2)P Unchanged

4.4.1.2 Minimum cover, c_{min}

(1)P Unchanged

NOTE As a reminder, the concrete cover is the distance between the axis of the reinforcement closest to the surface of the UHPFRC and the latter, reduced by half the nominal diameter of the reinforcement.

(2)P The value to be used is the largest c_{min} which satisfies the requirements both for bond and environmental conditions.

$$c_{min} = \max \{c_{min,b};\ c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add};\ c_{min,p}\ ;\ 10\ \text{mm}\} \qquad (4.2)$$

where:

$c_{min,b}$ minimum cover due to bond requirements, see 4.4.1.2 (3)

$c_{min,dur}$ minimum cover due to environmental conditions, see 4.4.1.2 (5)

$\Delta c_{dur,\gamma}$ additive safety margin, see 4.4.1.2 (6)

$\Delta c_{dur,st}$ reduction of minimum cover for use of stainless steel, see 4.4.1.2 (7)

$\Delta c_{dur,add}$ reduction of minimum cover for use of additional protection, see 4.4.1.2 (8)

$c_{min,p}$ minimum cover with regard to placement conditions of UHPFRC,

$c_{min,p} = \max\{1.5\ L_f;\ 1.5\ D_{sup};\ \phi\}$

where L_f is the length of the longest fibres contributing to ensuring non-brittleness, D_{sup} is the nominal upper dimension of the largest aggregate (see 5.4.3 of standard NF P18-470), ϕ is the diameter of the reinforcement, prestressing bars or ducts as applicable.

NOTE If specially demonstrated through a sufficiently uniform distribution of fibres in the control mockup during the suitability test, the value $c_{min,p}$ may be reduced but without going below L_f.

(3) Unchanged with regard to reinforcement.

The value of $c_{min,b}$ to be used is:

- For post-tensioned ducts:

- Ducts of circular cross-section: diameter of the duct
- Flat ducts: the smallest dimension or half of the largest dimension if this is greater

- For pre-tensioned prestressing bars: 2.0 times the diameter of the strand or wire, or the diameter of the largest aggregate if this is greater.

NOTE If using post-tensioned ducts and if specially demonstrated through a sufficiently uniform distribution of fibres in the control mockup during the suitability test, the value $c_{min,b}$ may be reduced but without going below L_f.

(4) For post-tensioning kits and anchorage, the minimum cover should be validated by transfer tests adapted from ETAG013 and conducted in accordance with Annex S.

(5) The structural class to be used for standard civil engineering buildings and structures is S4.

Table 4.3NF of standard NF EN 1992-1-1 does not apply; the only structural class changes possible are those associated with the design service life:

- when this is 100 years, the structural class is increased by 2,
- when this is 25 years or less, the structural class is decreased by 1.

The minimum structural class is S1.

The values of $c_{min,dur}$ to be used are those of Tables 4.202 (reinforcement steel) and 4.203 (prestressing steel) given below. When a structure member is affected by a number of exposure classes, the concrete cover with the strictest requirement is the one chosen.

Table 4.202 — Values of minimum cover $c_{min,dur}$ requirements with regard to durability for reinforcement steel compliant with EN 10080

Environmental requirement for $c_{min,dur}$ (mm)							
Structural class	Exposure class according to Table 4.1						
	X0	XC1	XC2/XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3
S1	-	5	5	10	10	15	15
S2		5	10	10	15	15	20
S3		5	10	15	15	20	20
S4		10	15	15	20	20	20
S5		10	15	20	20	20	25
S6		15	20	20	20	25	25

Table 4.203 — Values of minimum cover $c_{min,dur}$ requirements with regard to durability for prestressing steel

Environmental requirement for $c_{min,dur}$ (mm)							
Structural class	Exposure class according to Table 4.1						
	X0	XC1	XC2/XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3
S1	-	5	10	15	15	20	20

S2		10	15	15	20	20	20
S3		10	15	20	20	20	25
S4		15	20	20	20	25	25
S5		15	20	20	25	25	30
S6		20	20	25	25	30	30

(6) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(7) Unchanged

(8) Unchanged

(9) This paragraph also substitutes to 4.4.1.2. (109) of the bridges part and to the corresponding section of the National Annex.

In the case of a concrete or a UHPFRC cast in place in contact with other members in UHPFRC (precast or cast in place), the minimum concrete cover in UHPFRC relative to the interface may be reduced to the maximum of the values required for bonding ($c_{min,b}$ see (3) above) and for the compliance with the concreting conditions ($c_{min,p}$), provided that:

- the concrete belongs to at least strength class C25/30,
- the exposure of the concrete surface to an external environment is of a short duration (< 28 days),
- the interface has been treated (roughness, adhesive, resin, etc.)

(10) Unchanged

(11) Does not apply

(12) In the case of members in reinforced or prestressed UHPFRC exposed to freezing-thawing (classes XF), the concrete cover will be determined by reference to a class XC or XD by applying Table 4.204 below.

Table 4.204: Exposure classes to be adopted according to exposure class XF and the salting frequency

Salting type	**Exposure class**			
	XF1	**XF2**	**XF3**	**XF4**
Infrequent	XC4	Not applicable	XC4	Not applicable
Frequent	Not applicable	XD1, XD3 for very exposed members*	Not applicable	XD2, XD3 for very exposed members*
Very frequent	Not applicable	Not applicable	Not applicable	XD3
**For bridges: cornices, edge beams for anchoring safety barriers, expansion joint edges.*				

For reinforced or prestressed UHPFRCs exposed to chemical attack (class XA), concrete cover cannot be determined solely on the basis of exposure class XA without a special study and knowledge of the exposure classes chosen with regard to the corrosion risk of reinforcement or fibres.

(13) The severity of exposure to abrasion of a structure or part of a structure is described by classes XM1, XM2 and XM3 according to standard NF EN 1992-1-1 and NF EN 1992-3.

For structures subject to hydraulic flows at varying speeds and pressures, with possible impacting of sold particles carried along, the severity of the exposure is also described by these classes. The risk of abrasion must be assessed depending on the level of aggressiveness posed by the swell, currents, fluids circulating in and around the structure or the component considered, rubbing, and the presence of sediments or suspended abrasive materials.

NOTE Abrasion class XM1 relates to moderate abrasion, as on industrial sites subject to the movements of vehicles fitted with pneumatic tyres. Abrasion class XM2 relates to significant abrasion, as on industrial sites subject to the movements of fork-lift trucks fitted with pneumatic or solid rubber tyres. In particular, in accordance with standard NF EN 1992-2, raw road bridge decks (without waterproofing or pavement layers) normally come under class XM2. Abrasion class XM3 relates to extreme abrasion, as on industrial sites subject to the movements of fork-lift trucks fitted with elastomer or metal tyres, or track-laying equipment.

In the event of exposure to an abrasion risk associated with a hydraulic flow, compliance with the requirements described in 5.3.4 of standard NF P 18-470 makes it possible to dispense with a sacrificial thickness.

In other cases, except in case of an appropriate requirement defined by the specifier under 5.3.4 of NF P 18-470, article 4.4.1.2. (13) of NF EN 1992-1-1 supplemented by the National Annex applies.

(114) of standard NF EN 1992-2: Unchanged

(115) of standard NF EN 1992-2: Does not apply

4.4.1.3 Allowance in design for deviation

(1)P Unchanged

(2) The nominal cover c_{nom} should be used in designs and shown on drawings unless a value other than the nominal cover is specified (minimum value for example)

(3) The design margin for the execution tolerance Δc_{dev} may be reduced to 5 mm in the following case:

- when production is subject to a quality control system which includes measurements of the cover before casting the UHPFRC.

When the following conditions are satisfied, the execution tolerance Δc_{dev} may be reduced to 0 mm:

- when use of a very accurate measuring device for monitoring and rejecting non-conforming members (precast members for example) can be guaranteed.
- when the design and execution of structure members including their reinforcement are subject to a quality assurance system covering all the phases of the design on execution and including the following requirements for all exposure classes:
  - in the design and drawing phase: production of large scale detail drawings of sensitive reinforcement (cross section of strip, rail, parapet, etc.), specifying the concrete covers and forming,
  - in reinforcement construction phase: acceptance of shaped steel bars and checking their dimensions,
  - in the phase of steel frame installation within formwork: production of reinforcement support drawings (support type, frequency, fastening, etc.); acceptance of reinforcement and checking of covers before casting,
  - in UHPFRC placement phase: if applicable and relevant, fabrication of a control mock-up.

(4) Does not apply

5 STRUCTURAL ANALYSIS

5.1 General

5.1.1 General requirements

(1)P Unchanged

(2) Unchanged

(3) Does not apply

(4)P Unchanged

(5)P Unchanged

(6) Unchanged

(7) Unchanged

(108) of standard NF EN 1992-2: Unchanged

5.1.2 Specific requirements for foundations

(1)P Unchanged

(2) Unchanged

(3) Unchanged

(4) Unchanged

(5) Unchanged

5.1.3 Load case and combinations

(1)P Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1 and (101) of standard NF EN 1992-2 and its National Annex

5.1.4 Second order effects

(1)P Unchanged

(2)P Unchanged

(3) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

5.2 Geometric imperfections

(1)P The analysis of members and structures must take account of the unfavourable effects of any deviations in the geometry of the structure as well as in the positions of the loads.

NOTE Deviations in the dimensions of the sections are normally taken into account in the partial factors relating to the materials. There is therefore no need to include these imperfections in the structural analysis. The minimum eccentricity is given in 6.1(4) for the design of sections and does not relate to the stability design.

For the stability calculation, the geometrical imperfections given below shall not be less than 20 mm. This value may be reduced if specific provisions are taken into account on execution.

NOTE For example, for a column, the minimum eccentricity with regard to a geometrical imperfection may be taken as 15 mm where reduced tolerances are adopted relative to the class 1 tolerances of standard NF EN 13 670 and which will be defined by standard NF P18-451[7].

(2) Unchanged

(3) Unchanged

(4) Unchanged, including (104) of standard NF EN 1992-2

(5) The imperfections may be represented by an inclination θ_i:

$$\theta_i = \theta_0 \cdot \alpha_h \cdot \alpha_m \tag{5.1}$$

where

θ_0 is the basic value

α_h is a reduction factor for length or height: $\alpha_h = 2/\sqrt{l}$;

For buildings, $2/3 \leq \alpha_h \leq 1$

For bridges, $\alpha_h \leq 1$

α_m is a reduction factor for number of members:

For buildings, $\alpha_m = \sqrt{0{,}5(1+1/m)}$

For bridges, $\alpha_m = 1$

l is a length or a height [m], see (6)

m is the number of vertical members contributing to the overall effect

The value θ_0 to be used is 1/200. This value may be reduced if specific provisions are taken into account on execution.

NOTE For example, for a pier, θ_0 may be taken as 1/300 where reduced tolerances are adopted relative to the class 1 tolerances of standard NF EN 13 670 and which will be defined by standard NF P18-451[8].

(6) Unchanged, including (106) of standard NF EN 1992-2

(7) For isolated members (see 5.8.1), the effect of imperfections may be taken into account in two alternative ways a) and b) as desired:

a) As an eccentricity e_i given by

$$e_i = \theta_i \, l_0 / 2 \tag{5.2}$$

where l_0 is the effective length, see 5.8.3.2

For walls and isolated columns in braced systems, it is always possible, for simplicity, to adopt $e_i = l_0/400$, which equates to $\alpha_h = 1$. The value of e_i may be reduced if specific provisions are taken into account on execution.

NOTE For example, for a column, e_i may be taken as $l_0/600$ where reduced tolerances are adopted relative to the class 1 tolerances of standard NF EN 13 670 and which will be defined by standard NF P18-451[9].

b) As a transverse load H_i, in the position that gives maximum moment:

[7] In preparation

[8] In preparation

[9] In preparation

for unbraced members (see Figure 5.1 a1 of standard NF EN 1992-1-1):

$$H_i = \theta_i N \tag{5.3a}$$

for braced members (see Figure 5.1 a2 of standard NF EN 1992-1-1):

$$H_i = 2\theta_i N \tag{5.3b}$$

where N is the axial load

NOTE Eccentricity is suitable for statically determinate members, whereas transverse load can be used for both determinate and indeterminate members. The force H_i may be substituted by some other equivalent transverse action.

(8) Unchanged

(9) see 5.2 (7) a).

5.3 Idealisation of the structure

5.3.1 Structural models for overall analysis

(1)P The members of the structure are classified according to their type and function, in beams, piers, slabs, walls, plates, arcs, shells, etc. Rules are provided for the analysis of the most standard of these members and structures made up of assemblies of these.

Additional information is given in annex (V.2) of the present standard when a structure is modelled using finite members without necessarily being broken down into beams, piers, slabs, etc.

(2) Unchanged

(3) Unchanged

(4) Unchanged

(5) Unchanged

(6) Does not apply

(7) Unchanged

5.3.2 Geometric data

5.3.2.1 Effective width of flanges

(1)P Unchanged

(2) Unchanged

(3) Unchanged, however a note is added:

NOTE The effective width b_{eff} of the T beam flange is a maximum width. To calculate the cross section of flange/rib sliding reinforcement, it is possible to take into account the width of the flange which is just necessary to justify the strength of the beam under ultimate bending stresses.

(4) Unchanged

5.3.2.2 Effective span of beams and slabs in buildings

(1) Unchanged

(2) Unchanged

(3) Unchanged

(4) Unchanged, including (104) of standard NF EN 1992-2

5.4 Linear elastic analysis

(1) Unchanged

(2) Unchanged

(3) For the effects of heat-related deformations, settling and shrinkage at ultimate limit state (ULS), a reduced stiffness may be allowed corresponding to the cracked sections, taking into account the tensioned fibres and including the creep effect. For the serviceability limit state (SLS), a gradual change in cracking should be considered.

5.5 Linear analysis with limited redistribution

(1)P Does not apply

(2) Does not apply

(3) Does not apply

(4) Does not apply

(5) Does not apply

(6) Does not apply

5.6 Plastic analysis

5.6.1 General

(1) Unchanged, including (101)P of standard NF EN 1992-2

(2)P Unchanged

(3)P Unchanged

(4) Unchanged

(5) Addition: a plastic analysis according to the failure lines method for thin slabs along with an analysis with a strut and tie model is possible.

5.6.2 Plastic analysis for beams, frames and slabs

(1)P Unchanged

(2) Does not apply

(3) Does not apply

(4) Unchanged

(5) Does not apply

5.6.3 Rotation capacity

(1) Does not apply

(2) Does not apply

(3) Does not apply

(4) Does not apply

5.6.4 Analysis with strut and tie models

(1) Unchanged

(2) Does not apply

(3) Modelling by struts and ties consists of defining struts, which represent areas where compression stresses act, ties where tensile forces taken up by the UHPFRC act in the post-cracking area and by the reinforcement, if appropriate, and nodes which connect them. The forces in these members should be determined such that at the ultimate limit state, they continue to balance the loads applied. The members of the model should be sized according to the rules set out in 6.5.

(4) If reinforcement are installed, the position and orientation of these reinforcement should be made to coincide with those of the ties.

(5) Strut-and-tie models must be defined from stress isostatics and the distributions of the stresses obtained by application of linear elasticity theory.

5.7 Linear elastic analysis

(1) Unchanged

(2) Unchanged

(3) Unchanged

(4)P Unchanged

(5) For slender structures, it is possible to use the general method described in 5.8.6 which refers explicitly to the stability ULS. The verification of the stability ULS with the safety factors of the strength ULS dispenses with the need to check this under 5.7 (2).

(105) of standard NF EN 1992-2 and its National Annex: does not apply

(6) Addition : additional information is given in Annex V of this standard.

5.8 Analysis of second order effects with axial load

5.8.1 Definitions

Unchanged

5.8.2 General

(1)P Unchanged

(2)P When second order effects are taken into account, the balance and strength must be checked in deformed state. The deformations must be calculated taking account of the appropriate cracking effects and non-linear properties of the materials and the creep.

(3)P Unchanged

(4)P members bent in their main direction of smallest slenderness ratio (defined according to 5.8.3.2) must satisfy the rules of 5.8.9 (biaxial bending) or 5.9 (Lateral instability).

NOTE If the slenderness ratio is equal in both directions, it relates to members bent in their plane of greatest inertia.

(5)P Unchanged

(6) Does not apply

5.8.3 Simplified criteria for second order effects

5.8.3.1 Slenderness criterion for isolated members

(1) Does not apply

(2) Does not apply

5.8.3.2 Slenderness and effective length of isolated members

(1) Unchanged

(2) Unchanged

(3) Unchanged with the addition of the following note:

NOTE Attention is focused to the fact that the effective length of non-braced members given by the formula (5.16) is particularly affected with regard to lack of safety through an underestimation of flexibilities.

(4) Unchanged

(5) Unchanged

(6) In cases other than those mentioned in (2) and (3) above, in the case, for example, of members for which the axial force and/or section vary, the effective length may be established on the basis of the buckling load (calculated by a digital method, for example):

$$l_0 = \pi\sqrt{EI/N_B} \qquad (5.17)$$

where:

EI is a representative value of the bending stiffness

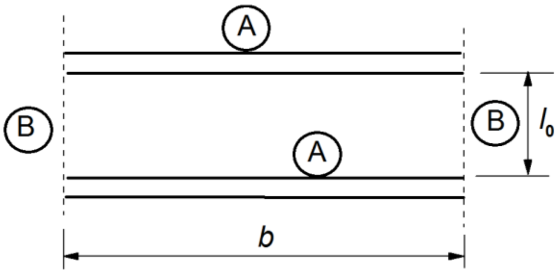
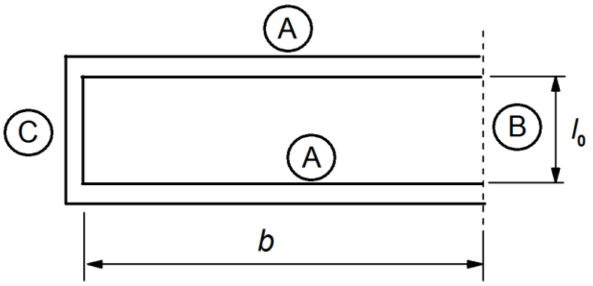
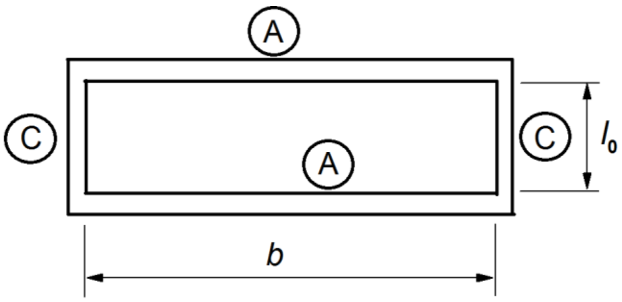
N_B is the buckling load expressed for this EI

(the i from Expression (5.4) should also relate to this same EI)

(7) The restraining effect of transverse walls may be allowed for in the calculation of the effective length of the walls by multiplying the effective length by a factor β obtained as follows:

- for walls free at one end β = 2;
- for other walls, use the values given by Table 5.201 below:

Table 5.201 — Values of β for various edge conditions

Edge embedding	Sketch	Expression	Factor β	
On two sides			β = 1.0 for all ratios l_0/b	
On three sides		$\beta = \frac{1}{1+\left(\frac{l_0}{3b}\right)^2}$	b/l_0	β
			0.2	0.26
			0.4	0.59
			0.6	0.76
			0.8	0.85
			1.0	0.90
			1.5	0.95
			2.0	0.97
			5.0	1.00
On four sides		If $\beta \geq l_0$ $\beta = \frac{1}{1+\left(\frac{l_0}{b}\right)^2}$ If $\beta < l_0$ $\beta = \frac{b}{2l_0}$	b/l_0	β
			0.2	0.10
			0.4	0.20
			0.6	0.30
			0.8	0.40
			1.0	0.50
			1.5	0.69
			2.0	0.80
			5.0	0.96

A — Floor slab

B — Free edge

C — Transverse wall

NOTE The information in Table 5.201 assumes that the wall has no openings with a height exceeding 1/3 of the wall height l_0 or with an area exceeding 1/10 of the wall area. In walls laterally restrained along 3 or 4 sides with openings exceeding these limits, the parts between the openings should be considered as laterally restrained along 2 sides only and be designed accordingly.

The values of β should be increased appropriately if the transverse bearing capacity is affected by chases or recesses.

A transverse wall may be considered as a bracing wall if:

- its total depth is not less than 0,5 h_w, where h_w is the overall depth of the braced wall;
- it has the same height l_w as the braced wall under consideration;
- its length l_{ht} is at least equal to l_0 / 5, where l0 denotes the clear height of the braced wall;
- within the length l_{ht} the transverse wall has no openings.

In the case of a wall connected along the top and bottom in flexurally rigid manner by cast in place UHPFRC and reinforcement, so that the edge moments can be fully resisted, the values for β given in Table 5.201 may be factored by 0,85.

The slenderness of walls in plain UHPFRC cast insitu should generally not exceed $\lambda = 86$ (i.e. $l_0/h_w = 25$).

5.8.3.3 Global second order effects in buildings

(1) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(2) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

5.8.4 Creep

(1)P Unchanged

(2) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(3) Unchanged

(4) Does not apply

(105) of standard NF EN 1992-2: Unchanged

5.8.5 Methods of analysis

(1)P In principle the general method of 5.8.6 based on a non-linear second order analysis is used.

NOTE As safety factors are introduced into the method, the demonstration of stability avoids the need to check the resistance of the sections with the exception of the method recommended in 5.8.8.2 (3).

(2) A simplified method based on a nominal stiffness is described in 5.8.7, along with a variant applicable to the members whose behaviour remains in the elastic domain. This method may also apply to complete structures.

(3) Simplifications or means of applying the previous methods are set out in 5.8.8.

5.8.6 General method

(1)P The general method is based on non-linear analysis including geometric non-linearity, i.e. second order effects.

(2)P Unchanged

(3) The stress-strain relationships of the compressed UHPFRC given in 3.1.5 of the present standard may be used. In the stress-strain relationship for UHPFRC, the compressive strength must be replaced by the design value f_{cd}. The value of Young's modulus to be used is E_{cd} defined by $E_{cd} = E_{cm}/\gamma_{CE}$.

γ_{CE} is equal to: 1.0 in the case defined in 5.8.7 (6)
1.2 in the other cases

For reinforcement steel, the relationship given in 3.2.7 shall be used.

The tensile strength of the UHPFRC may also be taken into account in accordance with 3.1.7, with the corresponding stress-strain relationships, with the fibre orientation factor K and the partial safety factor relating to tensioned UHPFRC γ_{cf}.

NOTE The general method can be used to calculate the ultimate shape stability ULS load by introducing the design values into the stress-strain relationships of the UHPFRC and reinforcement.

(4) Unchanged

(5) The favourable effect on average deformations the length of a structure section, therefore on displacements, of the participation by the tensioned and cracked UHPFRC may be taken into account. For UHPFRCs of class T3*, there is no distinction between average or local deformation. For UHPFRCs of class T1* and T2*, in the absence of bonded reinforcement, the mean stress-strain relationship shall be accepted in the same way as for local resistance. In the presence of bonded reinforcement, an average deformation mid-way between non-reinforced UHPFRC and non-fibre reinforced concrete shall be adopted.

NOTE This effect is favourable, and may always be ignored, for simplicity.

(6) Normally, conditions of equilibrium and strain compatibility are satisfied in a number of cross-sections. A simplified alternative is to consider only the critical cross section(s), and to assume a relevant variation of the curvature in between, e.g. similar to the first order moment or simplified in another appropriate way (see 5.8.8).

5.8.7 Simplified method based on nominal stiffness

(1) The method applies in the first instance to isolated members whose critical cross-section is known. It consists of demonstrating the existence of the stability ULS without explaining it. A value of the stiffness of the member is fixed a priori. The second order effects assuming pseudo-linear behaviour associated with this stiffness are then evaluated. It is then necessary to check that the deformations of the critical section under the total forces including first order (moment M_{0Ed}) and second order effects (moment M_{2Ed}) correspond to a rigidity at least equal to the fixed value.

(2) For a fixed stiffness (EI), the external forces being N_{Ed} and $M_{Ed} = M_{0Ed} + M_{2Ed}$, the state of equilibrium of the critical cross-section under these forces gives an internal curvature $1/r_i$, or a stiffness $(EI)_i = M_{Ed} / (1/r_i)$. The condition mentioned in (1) is then written:

$$1/r_i \leq M_{Ed} / (EI) \quad \text{or} \quad (EI)_i \geq (EI) \tag{5.201}$$

(3) The internal equilibrium must be calculated using the design stress-strain relationships (see 5.8.6 (3)).

(4) The method may also be applied to structures. The forces are evaluated by a second order linear analysis based on the stiffness values fixed in principle for each member. Condition (5.201) must be satisfied for each member.

(5) The linear analysis may be controlled using various approaches, for example for an isolated member, the amplification formula of the first order moment (see 5.8.8.3).

(6) In the special case where it can be assumed that the behaviour of the member remains within the linear elastic domain under the total ULS stresses, N_{Ed} and M_{Ed} (including the first order and second order effects), the stiffness is $(EI) = (EI)_0$ calculated with the raw cross-section of the UHPFRC and the effective Young's modulus $E_{cm} / (1 + \varphi_{ef})$.

This may be considered to be the case if the extreme stresses on the section under N_{Ed} and M_{Ed}, determined by an elastic calculation taking into account the raw cross section, fulfil:

$$\sigma_{min} \geq -\alpha \frac{f_{ctk,el}}{\gamma_{cf}} \tag{5.202}$$

$$\sigma_{max} \leq 0.7\ f_{cd} \tag{5.203}$$

where

$$\alpha = 0.7 \frac{f_{ctfk}}{K_{global}.f_{ctk,el}} \leq 1 \tag{5.204}$$

If the two inequalities below are satisfied, condition (5.201) is also satisfied.

5.8.8 Method based on nominal deformation shape

Section 5.8.8 of standard NF EN 1992-1-1 is fully replaced by the following.

5.8.8.1 General

(1) This method enables the calculations in the general method to be simplified. It is suitable, first and foremost, for isolated members of a given effective length and known critical cross-section subject to a constant axial force half-way along their length l_0. It gives a distribution of second order moments based on a nominal deformation shape, which enables the second order moment of the critical section to be calculated as a function of the curvature of this section. This nominal deformation shape assumption may also be used in the simplified method of 5.8.7.

5.8.8.2 Bending moments

(1) The design moment is equal to:

$$M_{Ed} = M_{0Ed} + M_2 \tag{5.205}$$

Where M_{0Ed} is the first order design moment taking account of the imperfections, according to 5.2
M_2 is the second order moment

The distribution of M_2 the length of l_0 may be considered sinusoidal, at least for the elements of a constant cross-section. The maximum moment M_{Ed} must be located in the median section of the effective length l_0.

(2) The second order moment M_2 in (5.205) is equal to:

$$M_2 = N_{Ed}\ e_2 \tag{5.206}$$

Where N_{Ed} is the design acting axial force
e_2 is the second order eccentricity, associated with the curvature 1/r of the critical section by $e_2 = (l_0^2/\pi^2)(1/r)$

(3) In the case of members not subject to a transverse load, the first order end moments M_{01} and M_{02}, when they differ, may be replaced by an equivalent constant first order moment M_{0e} defined by:

$$M_{0e} = 0.6\ M_{02} + 0.4\ M_{01} \tag{5.207}$$

M_{01} and M_{02} should be taken as having the same sign if they give rise to tension on the same side and opposing signs otherwise. In addition, it is assumed that $|M_{02}| \geq |M_{01}|$. Besides the verification of the stability ULS with the equivalent moment M_{0e}, the resistance ULS of section 2 must also be checked.

5.8.8.3 Amplification formula for the first order moment

(1) For a nominal stiffness (EI) which is assumed to be known, the critical elastic buckling load is expressed as:

$$N_B = (\pi^2 / l_0^2)\ (EI) \tag{5.208}$$

(2) The total moment can be calculated using the amplification formula for the first order moment:

$$M_{Ed} = \frac{M_{0Ed}}{1 - \frac{N_{Ed}}{N_B}} \quad (5.209)$$

This formula assumes a near-sinusoidal total moment distribution.

5.8.9 Biaxial bending

(1) The general method described in 5.8.6 may also be used for biaxial bending. The following provisions apply when simplified methods are used. Special care should be taken to identify the section along the member with the critical combination of moments.

For a member with a constant axial force over its length, the cross-sections with maximum moment in each of the planes y and z must coincide.

(2) An initial step may consist of carrying out a separate *plane* calculation in each of the main directions *y or z*. The corresponding geometric imperfections must be taken into account. Each of these calculations, with first order eccentricity e_0 fixed, gives a value M_{Ed} of the total acting moment including the second order moment and a value M_{Rud} of the resistance moment at stability ULS, such that:

$M_{Edy} \leq M_{Rudy}$ (bending in the zx plane)

and $M_{Edz} \leq M_{Rudz}$ (bending in the xy plane).

(3) The direction with the largest slenderness ratio $\lambda_y = l_{0y} / i_z$ (relative to the dimension h) is denoted as y and that of the smallest slenderness ratio $\lambda_z = l_{0z} / i_y$ (relative to dimension b) is denoted as z – see Figure 5.8.

No additional verification to those of (2) above is required if the slenderness ratios λ satisfy:

$$\lambda_y / \lambda_z \leq 2 \quad (5.210)$$

and if the relative eccentricities satisfy:

$$(e_y/h_{eq}) / (e_z/b_{eq}) \leq 0.2 \quad (5.211)$$

$$\text{or} \quad (e_y/h_{eq}) / (e_z/b_{eq}) \geq 5 \quad (5.212)$$

Where i_y, i_z are the radii of gyration of the sections in the z and y directions

$h_{eq} = i_z \sqrt{12}$; $b_{eq} = i_y \sqrt{12}$

$e_y = M_{Edz} / N_{Ed}$; $e_z = M_{Edy} / N_{Ed}$

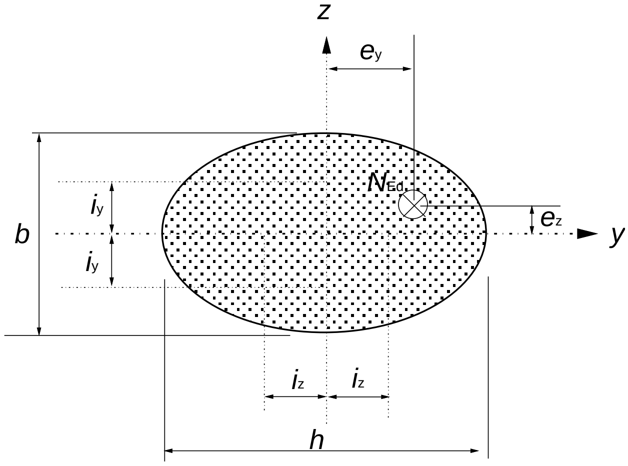


Figure 5.8 — Definition of eccentricities e_y and e_z

(4) If condition (5.210) is satisfied and (5.211) and (5.212) are not satisfied, or (5.210) is not satisfied but (5.212) is satisfied, the biaxial bending is taken into account by integrating the second order effects in both directions according to the following simplified criterion:

$$(M_{Edy} / M_{Rudy})^a + (M_{Edz} / M_{Rudz})^a \leq 1 \qquad (5.213)$$

Where M_{Edy} and M_{Edz} are the acting moments from the plane calculations mentioned in (2), second order inclusive

M_{Rudy} and M_{Rudz} are the stability ULS moment resistances from the plane calculations mentioned in (2).

Exponent a takes the following values:

for circular or elliptical sections: $a = 2$

for rectangular sections

N_{Ed}/N_{Rd}	0.1	0.7	1.0
a	1.0	1.5	2.0

with a linear interpolation for the intermediate values
N_{Ed} is the design axial force
$N_{Rd} = A_c f_{cd} + A_s f_{yd}$, design resistance in terms of axial force for the section
where:
A_c gross area of the UHPFRC section
A_s area of longitudinal reinforcement

(5) <u>Addition</u>: In cases not envisaged in (3) and (4) above, a biaxial calculation using the general method must be carried out explicitly by applying the imperfections in two directions simultaneously, even where the eccentricity e_y is null at the first order.

(6) <u>Addition</u>: Biaxial bending is always accompanied by torsion. The calculations mentioned in the above articles assume that the displacement term, due to torsion is negligible, which may be incorrect for sections with thin members or h/b ratio greater than 5. It will in general be necessary that the torsion stiffness is not significantly reduced by shear cracks by checking that the torsional moment at stability ULS does not exceed 40% of the torsional moment which leads cracking under pure torsion.

5.9 Lateral instability of slender beams

(1)P Unchanged

(2) When checking unbraced beams, a lateral deflection equal to l / 300 should be adopted, where l = total length of the beam, and treated as a geometric imperfection. In finished structures, the bracing provided by members fitted to the beam considered may be taken into account.

The lateral deflection of l/300 may be reduced if specific provisions are taken into account on execution.

NOTE For example, the lateral deflection may be taken as l/450 where reduced tolerances are adopted relative to the class 1 tolerances of standard NF EN 13 670 and which will be defined by standard NF P18-451[10].

(3) Does not apply

(4) Second order stresses due to the instability of the beam should be taken into account for calculating full fixed end moment in bearing structures, including torsion embedment.

NOTE For beams with thin sections, the displacement term due to torsion may not be negligible.

[10] In preparation

5.10 Prestressed members and structures

5.10.1 General

(1)P Unchanged

(2) Unchanged

(3) Unchanged

(4) Unchanged

(5)P Unchanged

(6) Brittle failure should be avoided by one or more of the following methods:

- comply with the non-brittleness condition (see 9.1 (3))
- providing pretensioned bonded tendons
- ensure that if failure were to occur due to either an increase of load or a reduction of prestress under the frequent combination of actions, cracking would occur before the ultimate capacity would be exceeded

(106) of standard NF EN 1992-2: unchanged

(7) Addition: when reference is made to the European Technical Approval (or European Technical Assessment) in the following sections, reference should also be made to Annex S.

5.10.2 Prestressing force during tensioning

5.10.2.1 Maximum stressing force

(1)P Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(2) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

5.10.2.2 Limitation of concrete stress

(1)P Unchanged

(2) Unchanged

(3) A lower limit should be applied to the strength of concrete at application or transfer of prestress. The corresponding values for post-tensioning are given in Annex S.

(4) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(5) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

5.10.2.3 Measurements

(1)P Unchanged

5.10.3 Prestress force

(1)P Unchanged

(2) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(3) Unchanged

(4) Unchanged

5.10.4 Immediate losses of prestress for pre-tensioning

(1) Unchanged

5.10.5 Immediate losses of prestress for post-tensioning

5.10.5.1 Losses due to the instantaneous deformation of concrete

(1) Unchanged

(2) Unchanged

5.10.5.2 Losses due to friction

(1) Unchanged

(2) Unchanged

(3) Unchanged

(4) Unchanged

5.10.5.3 Losses at anchorage

(1) Unchanged

(2) Unchanged

5.10.6 Time dependent losses of prestress for pre- and post-tensioning

(1) Unchanged

(2) Unchanged

(3) Unchanged

5.10.7 Consideration of prestress in analysis

(1) Unchanged

(2) Unchanged

(3) Does not apply

(4) Does not apply

(5) Unchanged

(6) Unchanged

5.10.8 Effects of prestressing at ultimate limit state

(1) Unchanged

(2) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(3) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1 and (103) of standard NF EN 1992-2

5.10.9 Effects of prestressing at serviceability limit state and limit state of fatigue

(1)P Unchanged, including the additions of the National Annexes to standards NF EN 1992-1-1 and NF EN 1992-2

5.11 Analysis for some particular structural members

(1)P Unchanged

(2)P Bracing walls are walls which contribute to the lateral stability of the structure.

6 ULTIMATE LIMIT STATES

6.1 Bending with or without axial force

(1)P Unchanged

(2)P When determining the ultimate moment resistance of UHPFRC cross-sections (non-reinforced, reinforced or prestressed), the following assumptions are made:

- plane sections remain plane
- for a given deformation state, the stress in compression or tensioned UHPFRC are inferred from the diagrams given in 3.1.7
- the strain in bonded reinforcement or bonded prestressing tendons, whether in tension or in compression, is the same as that in the surrounding UHPFRC
- the stresses in the reinforcing or prestressing steel are derived from the design curves in 3.2.7 and 3.3.6.
- the initial strain in prestressing tendons is taken into account when assessing the stresses in the tendons

(3)P The compressive strain in the UHPFRC shall be limited to ε_{cud} according to the diagram defined in 3.1.7. The strains in the reinforcing steel and the prestressing steel shall be limited to ε_{ud} (where applicable); see 3.2.7 (2) and 3.3.6 (7) respectively.

(4) For cross-sections loaded by the compression force (excluding the prestressing force) it is necessary to assume the minimum eccentricity, e_0 = h/30 but not less than 20 mm where h is the depth of the section.

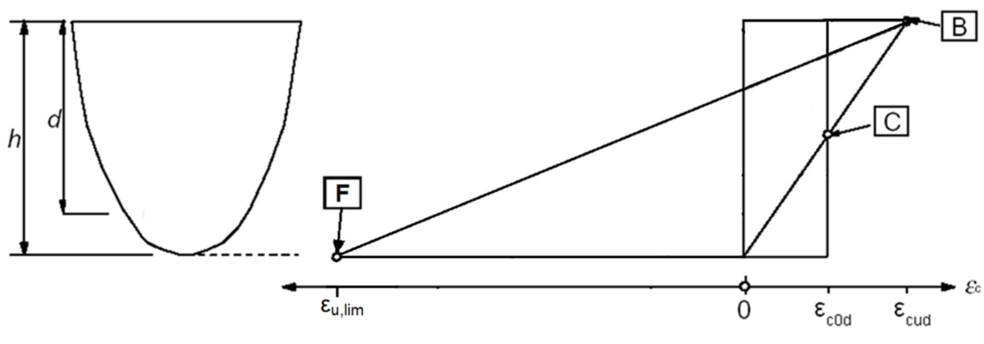
This minimal eccentricity value may be reduced if specific provisions are taken into account on execution.

NOTE For example, the minimum eccentricity value may be taken as the maximum of h/45 and 10 mm where reduced tolerances are adopted relative to the class 1 tolerances of standard NF EN 13 670 and which will be defined by standard NF P18-451[11].

(5) In parts of cross-sections which are subjected to approximately concentric loading ($e_d/h \leq 0.1$), such as compression flanges of box girders, the mean compressive strain in that part of the section should be limited to ε_{c0d}.

(6) The resisting forces of the sections are calculated from a linear deformation diagram included in a domain defined by the limiting deformations known as pivots defined below.

a) In the case of non-reinforced UHPFRC, the pivot limits are as follows:



B - UHPFRC compression strain limit

C - UHPFRC pure compression strain limit

F - UHPFRC tension strain limit

Figure 6.201 — Diagram of relative deformations admissible in ultimate limit state for non-reinforced UHPFRC

[11] In preparation

Pivot B: pivot B relates to the limit shortening strain ε_{cud} of the UHPFRC on the most compressed fibre of the section.

Pivot C: pivot C relates to the limit shortening strain ε_{c0d} of the UHPFRC under compression for the part due only to the axial force.

Pivot F: when the tensile law chosen for the calculation does not have descending branch (for example the conventional laws for class T3* UHPFRCs or conventional laws for thin members), pivot F relates to the limit lengthening $\varepsilon_{u,lim}$ of the UHPFRC on the most tensioned fibre of the section.

NOTE For other cases, the resistance moment is obtained for a deformation of the most tensioned fibre less than $\varepsilon_{u,lim}$ (pivot F is then never reached).

b) In the case of reinforced and/or prestressed UHPFRC, the pivot limits are as follows:

Pivot A: pivot A relates to the ultimate deformation of the reinforcement, if applicable, in accordance with 3.2.7.

NOTE In the case of a diagram with a horizontal plateau, pivot A is undefined.

Pivot B: pivot B relates to the limit shortening strain ε_{cud} of the UHPFRC on the most compressed fibre of the section.

Pivot C: pivot C relates to the limit shortening strain ε_{c0d} of the UHPFRC under compression for the part due only to the axial force.

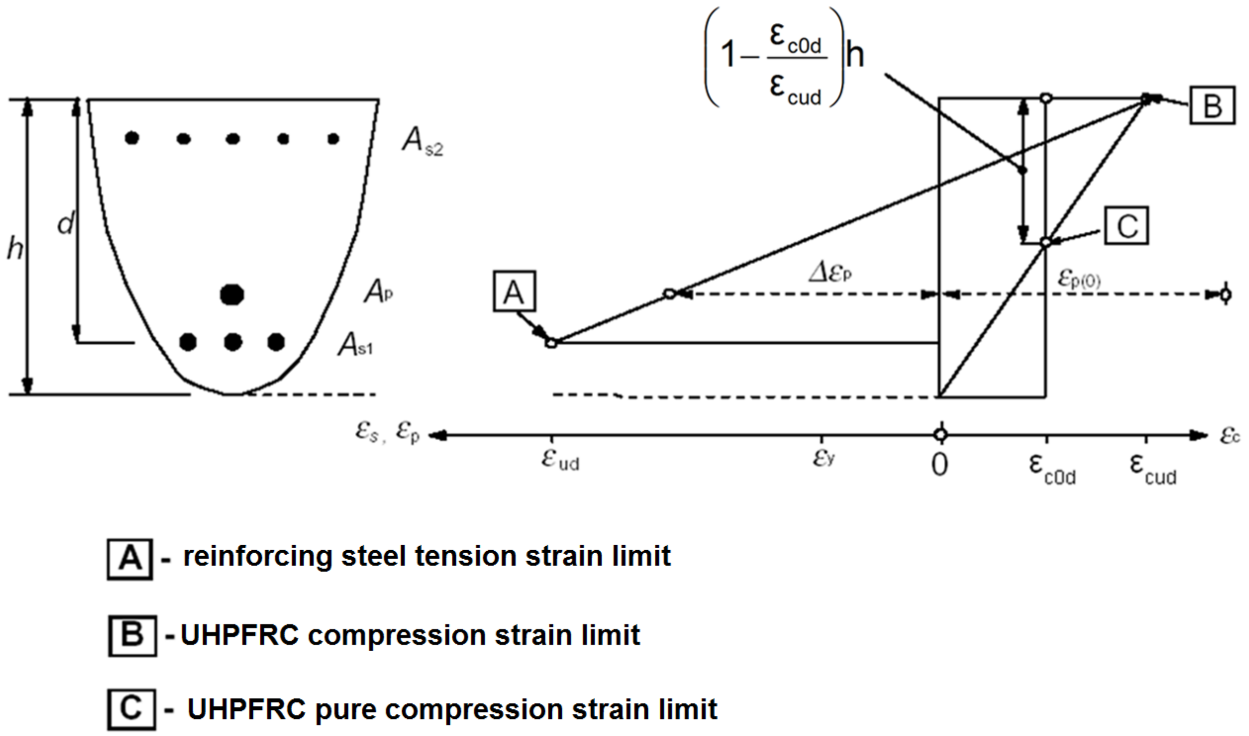


Figure 6.202 — Diagram of relative deformations admissible in ultimate limit state for reinforced and/or prestressed UHPFRC

In addition, for thin member sections of non-reinforced, reinforced or prestressed UHPFRC elements under bending with axial force, the contribution of the tensioned UHPFRC can only be taken into account if the tensile deformation over the mean fibre is not greater than $\varepsilon_{u,lim}$ / 2 unless special justification.

(7) Unchanged

(8) Unchanged, including (108) of standard NF EN 1992-2

(109) of standard NF EN 1992-2:

To satisfy the requirement of non-brittleness at ULS from 5.10.1 (5) of standard NF EN 1992-1-1 with a reduced prestress cross-section, the following method should be applied:

i) Calculate the applied bending moment due to the frequent combination of actions

ii) Determine the reduced area of prestress that results in the tensile stress reaching $f_{ctm,el}$ at the extreme tension fibre when the section is subject to the bending moment calculated in i) above.

iii) Using this reduced area of prestress, calculate the ultimate flexural capacity. It should be ensured that this exceeds the bending moment due to the frequent combination. Redistribution of internal actions within the structure may be taken into account for this verification and the ultimate bending resistance should be calculated using the material partial safety factors for accidental design situations given in Table 2.101 of 2.4.2.4.

6.2 Shear

6.2.1 General verification procedure

6.2.1.1 General

(1) Part 6.2.1 of the present standard substitutes for parts 6.2.1, 6.2.2 and 6.2.3 of standard NF EN 1992-1-1 and standard NF EN 1992-2.

(2) The acting design shear force V_{Ed} must be less than the resisting shear force $V_{Rd,total}$. The resisting shear force $V_{Rd,total}$ is equal to the smallest of V_{Rd} and $V_{Rd,max}$.

- $V_{Rd,max}$ is the limit force for the compressive strength of the concrete compression struts in the truss diagram
- V_{Rd} is the superposition of the three resistance terms $V_{Rd,c} + V_{Rd,s} + V_{Rd,f}$
- $V_{Rd,c}$ is the UHPFRC contribution term
- $V_{Rd,s}$ is the contribution term for the transverse reinforcement in the truss diagram
- $V_{Rd,f}$ if the fibre contribution term

NOTE As terms V_{ccd} and V_{td} of standard NF EN 1992-1-1 respectively designate the design values of the shear component contributed by an inclined compression chord and tensile chord, they are not used in this standard as they are not specific to UHPFRCs. However, they are used in the same way in the calculations for UHPFRC structures.

(3) The following provisions relate to members loaded on their upper surface. If the loads are applied from below, vertical suspension reinforcement must be provided in addition.

(4) The tensioned longitudinal reinforcement should be able to resist the additional tensile force generated by the shear force (see 6.2.1.6).

(5) In the case of members mainly subject to uniformly distributed loads, there is no need to carry out a shear force verification at a distance to the reference surface of the support less than h for members in UHPFRC with bonded flexure reinforcement or d if there are bonded reinforcement. The justified design in this section remains valid right up to the support. A check should also be made that the shear force acting on the support does not exceed the value $V_{Rd,max}$ defined in 6.2.1.5

6.2.1.2 Term $V_{Rd,c}$

(1) For a reinforced section, the design resisting shear force $V_{Rd,c}$ provided by the UHPFRC is given by the formula:

$$V_{Rd,c} = \frac{0.21}{\gamma_{cf}\gamma_E} k\, f_{ck}^{1/2}\, b_w\, d \tag{6.201}$$

where:

$$k = 1 + 3 \cdot \frac{\sigma_{cp}}{f_{ck}} \tag{6.202}$$

Where

$$\sigma_{cp} = N_{Ed} / A_c \tag{6.203}$$

N_{Ed} is the axial force in the cross-section, due to the external loads ($N_{Ed} > 0$ for compression). The influence of imposed deformations on N_{Ed} may be ignored.

A_c is the cross-sectional area of the UHPFRC.

Under Expression (6.202), the value of σ_{cp} is limited as follows:

$$0 \leq \sigma_{cp} \leq 0.4\ f_{ck} \tag{6.204}$$

The term γ_E is a safety factor taken such that $\gamma_{cf}\ \gamma_E$ is equal to 1.5.

(2) For a reinforced or non-reinforced prestressed section, the design resisting shear force $V_{Rd,c}$ is given by the formula:

$$V_{Rd,c} = \frac{0.24}{\gamma_{cf}\gamma_E} k\, f_{ck}^{1/2}\, b_w\, z \tag{6.205}$$

(3) For a non-prestressed and non-reinforced section:

$$V_{Rd,c} = \frac{0.18}{\gamma_{cf}\gamma_E}\ k\, f_{ck}^{1/2}\, b_w\, h \tag{6.206}$$

In all cases:

f_{ck} is expressed in MPa

b_w is the smallest width of the cross-section in tensile area [m]. In the case of a circular section of diameter Φ, 0.55 Φ should be selected for b_w.

z is the lever arm of the internal forces for a member of constant height corresponding to the bending moment in the member considered.

d is the distance between the most compressed fibre and the longitudinal reinforcement.

NOTE The formulae are consistent if z = 0.9 d and d=7/8 h in the non-reinforced case

6.2.1.3 Term $V_{Rd,s}$

The resistance to the shearing force from the vertical reinforcement is given by the following expression:

$$V_{Rd,s} = \frac{A_{sw}}{s} z\ f_{ywd} \cot\theta \tag{6.207}$$

In the case where the member includes inclined reinforcement, the term for the resistance to the shearing force provided by the reinforcement is:

$$V_{Rd,s} = \frac{A_{sw}}{s} z\ f_{ywd} \left(\cot\theta + \cot\alpha\right)\sin\alpha \tag{6.208}$$

A_{sw} is the cross-sectional area of the shear reinforcement

s is the spacing of frames or stirrups

f_{ywd} is the design elastic limit of the shear reinforcement

θ is the inclination of the main compression stress on the longitudinal axis. The stress tensor is evaluated adjacent to the centre of gravity of the section by an elastic calculation from the force torsor determined at the ultimate limit states.

θ = 30° should be chosen as the minimum value.

α is the inclination of the reinforcement on the longitudinal axis.

In circular sections reinforced with hoops or circular frames, $V_{Rd,s}$ should be reduced by 30% to take account of the fact that the reinforcement do not work directly in the direction of the tie parallel to the shear force, unlike with frames.

6.2.1.4 Term $V_{Rd,f}$

(1) The design resisting shear force $V_{Rd,f}$ contributed by the fibres is given by the following expressions:

$$V_{Rd,f} = A_{fv} \sigma_{Rd,f} \cot\theta \tag{6.209}$$

In the case of class T1* or T2* UHPFRCs:

$$\sigma_{Rd,f} = \frac{1}{K\ \gamma_{cf}} \frac{1}{w^*} \int_0^{w^*} \sigma_f(w)\, dw \tag{6.210}$$

$$w^* = \max(w_u\ ; 0.3\ \text{mm}) \tag{6.211}$$

NOTE $\sigma_{Rd,f}$ is the mean value of the post-cracking strength along the shear crack of inclination θ, and perpendicular to it. The term $A_{fv}\, \sigma_{Rd,f} \cot\theta$ is the projection of the resultant force parallel to the shear force, the area A_{fv} being itself the projection in the cross section of the inclined area on which the fibres act.

- for a rectangular section or a T section,

$$A_{fv} = b_w z \tag{6.212}$$

z is the lever arm of the internal forces for a member of constant height corresponding to the bending moment at the same time as the shear force in the member considered. For shear force resistance calculations on a reinforced section without normal force, it is possible to adopt the best-estimate value z = 0.9 d.

- for a circular section of diameter Φ,

$$A_{fv} = 0.58\Phi^2 \tag{6.213}$$

w_u is the ultimate opening of cracks reached at ULS under bending with axial force on the end fibre, under the moment acting in the section.

In the case of UHPFRCs of class T3*, the expression for $\sigma_{Rd,f}$ becomes:

$$\sigma_{Rd,f} = \frac{1}{K.\gamma_{cf}} \times \frac{1}{\varepsilon^* - \varepsilon_{el}} \int_{\varepsilon_{el}}^{\varepsilon^*} \sigma_f(\varepsilon)\, d\varepsilon \tag{6.214}$$

Where:

$\varepsilon^* = \max(\varepsilon_u\ ;\ \varepsilon_{u,lim})$

ε_u being the maximum elongation in the ULS calculation of bending with axial force

ε_{el} et $\varepsilon_{u,lim}$ being defined in 3.1.7.3.1 (7)

Except for members of very small dimensions, in Expressions (6.210) and (6.214), the value of K to use is K_{global}. For calculating the shear force, a member is of very small dimensions if the width b_w and the height h are both less than $5.L_f$, where L_f is the length of the longest fibres contributing to ensuring non-brittleness.

6.2.1.5 Term $V_{Rd,max}$

(1) For members in UHPFRC without shear reinforcement, the resistance limit of the compression struts is:

$$V_{Rd,max} = 2.3\frac{\alpha_{cc}}{\gamma_C} b_w z\ f_{ck}^{2/3}.\tan\theta \tag{6.215}$$

θ designates the inclination of the main compression stress on the longitudinal axis.

For members in UHPFRC with shear reinforcement inclined at α :

$$V_{Rd,max} = 2.3\frac{\alpha_{cc}}{\gamma_C} b_w z\ f_{ck}^{\ 2/3}\left[\frac{V_{Rd,s}(\cot\theta+\cot\alpha)}{(1+\cot^2\theta)} + V_{Rd,f}\tan\theta\right].\left[\frac{1}{V_{Rd,s}+V_{Rd,f}}\right] \tag{6.216}$$

(2) When the web comprises ducts, whether or not they are grout-filled, the diameter of the duct is deducted from the width of the beam b_w. This width must be replaced by the nominal width $b_{w,nom}$ defined as follows:

$$b_{w,nom} = b_w - \sum\varphi \tag{6.217}$$

6.2.1.6 Complementary tensile force due to the shear force

(1) For members in UHPFRC without flexure reinforcement or shear force, the balance of the sections on bending must be demonstrated taking account of the offset a_l of the moments curve defined in 9.2.1.3.

(2) For members in flexure-reinforced UHPFRC, but without shear reinforcement, the longitudinal flexure reinforcement must be able to resist the additional tensile force ΔF_{td}, due to the shear force V_{Ed}, calculated using the Expression:

$$\Delta F_{td} = 0.5\ V_{Ed}\ (\cot\theta - \tan\theta) \geq 0 \tag{6.218}$$

(3) For flexure- and shear-reinforced UHPFRC members (transverse reinforcement at angle α), the longitudinal flexure reinforcement must be able to resist the additional tensile force:

$$\Delta F_{td} = 0.5\ V_{Ed}\ [V_{Rd,f}\ (\cot\theta - \tan\theta) + V_{Rd,s}\ (\cot\theta - \cot\alpha)]\ /\ (V_{Rd,s} + V_{Rd,f}) \geq 0 \tag{6.219}$$

(4) In all cases, $(M_{Ed}/z) + \Delta F_{td}$ should not be greater than $M_{Ed,max}/z$, where $M_{Ed,max}$ is the maximum moment along the beam.

6.2.1.7 Concentrated loads near supports

(1) When loads are applied to the upper surface of the member at a distance a_v from the reference surface of the support such that $a_v \leq 2.0d$, the acting design shear V_{Ed} must be less than the resisting shear which is equal to the smallest of the values V'_{Rd} and $V'_{Rd,max}$:

$$V_{Ed} \leq \min\ (V'_{Rd}\ ;\ V'_{Rd,max}) \tag{6.220}$$

$$V'_{Rd} = V'_{Rd,c} + V'_{Rd,s} + V'_{Rd,f} \tag{6.221}$$

V'_{Rd} is the superimposition of the three resisting terms which must be calculated according to the following Expressions in (2), (3) and (4).

(2) The UHPFRC contribution term $V'_{Rd,c}$ is given by the following Expression:

$$V'_{Rd,c} = \beta'\ V_{Rd,c} \tag{6.222}$$

Where $V_{Rd,c}$ is given in 6.2.1.2

$$\beta' = 3 - a_v/d \tag{6.223}$$

In the absence of a longitudinal reinforcement, d = 0.9 h should be adopted.

(3) The transverse reinforcement contribution term $V'_{Rd,s}$ is given by the following Expressions:

- For $0.5 < a_v/d < 2$:

$$V'_{Rd,s} = \frac{1}{1{,}50}\left[(\frac{a_v}{d} - 0{,}5)\cot\theta \frac{A_{sw,t}}{s_t} l_t f_{ywd,t} + (2 - \frac{a_v}{d})\frac{A_{sw,l}}{s_l} h_l f_{ywd,l}\right] \tag{6.224}$$

where:

$A_{sw,t}$ is the cross-sectional area of a course of transverse reinforcement

s_t is the spacing of the transverse reinforcement courses

The total cross-sectional area of the transverse reinforcement $(A_{sw,t}/s_t).l_t$ is counted over a length l_t centred on the span a_v, defined by:

$$l_t / d = 0.6 + 0.15\ (a_v/d) \tag{6.225}$$

$A_{sw,l}$ is the cross-sectional area of a course of longitudinal reinforcement distributed over the height of the section

s_{tl} is the spacing of the distributed longitudinal reinforcement courses

NOTE For further explanation about the distance a_v, refer to FD P18-717 6.2.1 - (II)

The total cross-sectional area of the longitudinal reinforcement $(A_{sw,l} / s_l).h_l$ is aggregated over a height h_l, reckoned from the flexure reinforcement or from the tensioned surface if there is no reinforcement, defined by:

$$h_l / d = 0.6 + 0.15\ (a_v/d) \tag{6.226}$$

θ should be bounded by:

$$\cot\theta \le a_v/d \tag{6.227}$$

If cotθ is greater than a_v/d, cotθ must be replaced by a_v/d in $V'_{Rd,s}$ (Expression (6.224)).

It is possible to simplify the calculation by adopting a distribution in principle between the two reinforcement families:

$$\frac{\frac{A_{sw,l}}{s_l} h_l}{\frac{A_{sw,l}}{s_l} + \frac{A_{sw,t}}{s_t} l_t \cot\theta} = 1 - \frac{1}{1{,}15}(\frac{a_v}{d} - 0{,}5) \tag{6.228}$$

In the case of circular sections, the $A_{sw,t}$ areas should be reduced by 30%.

- For $0 < a_v/d \le 0.5$:

$$V'_{Rd,s} = (A_{sw,l} / s_l)\ h_l\ f_{ywd,l} \tag{6.229}$$

Where:

$$h_l / d = 1 - 0.65\ (a_v/d) \tag{6.230}$$

(4) The fibre contribution term $V'_{Rd,f}$, under flexure with or without axial force with a axial compressive force N_{Ed} is given by the following expressions:

- for $1.0 < a_v/d \le 2.0$:

$$V'_{Rd,f} = A'_{fv}\ \sigma_{Rd,f} \cot\theta \tag{6.231}$$

Where $\sigma_{Rd,f}$ is calculated by formula (6.210) or (6.214) with the limit w* or ε* relating to the ULS flexure calculation in the section at a distance 2 d from the support.

Where:

$$A'_{fv} = b_w h_f \tag{6.232}$$

The height h_f is defined by the following expression:

$$h_f / d = 0.6 + 0.15 (a_v/d) \tag{6.233}$$

θ should be bounded by:

$$\cot\theta \leq a_v/d$$

- for $0.5 < a_v/d \leq 1.0$:

$$V'_{Rd,f} = A'_{fv} \sigma_{Rd,f} (a_v/d) \tag{6.234}$$

Where A'_{fv} is defined according to (6.232) and therefore h_f according to (6.233).

- for $0 < a_v/d \leq 0.5$:

$$V'_{Rd,f} = 0.8 (1 - 0.75 a_v/d) A'_{fv} \sigma_{Rd,f} \tag{6.235}$$

Where A'_{fv} is defined according to (6.232) and h_f is defined by $h_f / d = 1 - 0.65 (a_v / d)$

For circular sections, take $b_w = 0.7 \Phi$.

(5) The limit force $V'_{Rd,max}$ is expressed as follows:

$$V'_{Rd,max} = 1.14 \Psi (\alpha_{cc} / \gamma_c) b_w z f_{ck}^{2/3} \tag{6.236}$$

- For $1 < a_v / d < 2$:

$$\psi = \frac{2}{1,5}.\left[1 - 0,5\frac{a_v}{d} + (\frac{a_v}{d} - 0,5).\left[\frac{\frac{V'_{Rd,s}}{(\cot\theta + \tan\theta)} + V'_{Rd,ft}\tan\theta}{V'_{Rd,s} + V'_{Rd,f}}\right]\right] \tag{6.237}$$

θ should be bounded by:

$$\cot\theta < a_v/d$$

- For $0 < a_v / d < 1$: $\Psi = 1$

6.2.2 Members not requiring design shear reinforcement

Does not apply, see 6.2.1.

6.2.3 Members requiring design shear reinforcement

Does not apply, see 6.2.1.

6.2.4 Shear between web and flanges

(1) The shear strength of the flange may be calculated by considering the flange as a system of compressive struts combined with ties relating to the tensioned UHPFRC with or without tensile reinforcement.

(2) Does not apply

(3) Unchanged, including (103) of standard NF EN 1992-2

(4) To justify the shear between the web and flanges, the following inequality must be satisfied:

$$v_{Ed} \leq v_{Rd,f} + v_{Rd,s} \tag{6.238}$$

Where:

$$v_{Rd,f} = \frac{h_{fs}}{h_f} \sigma_{Rd,f} \cot\theta_f \tag{6.239}$$

and

$$v_{Rd,s} = \frac{A_{sfs}.f_{yd}}{h_f.s_f} \cot\theta_f \tag{6.240}$$

θ_f is the angle of the compression struts to the web of the beam. The value of cot θ_f =1 should be adopted except for transverse prestress.

$\sigma_{Rd,f}$ is the mean value of the post-cracking strength defined in 6.2.1.4. For UHPFRCs of class T1* and T2*, $\sigma_{Rd,f}$ is calculated using w* = 0.3 mm

h_f is the thickness of the flange at the joint

h_{fs} is the height of the flange acting in shear mode, see Figure 6.203

A_{sfs} cross-section of transverse reinforcement used for shear

s_f spacing between transverse reinforcement used for shear

As indicated in Expression (6.238), any reinforcing steel perpendicular to the web/flange joint located in the area used to resist the longitudinal shear v_{Ed} (area defined by the height h_{fs}) may be used to resist all or part of this shear via term A_{sfs}.

It is also necessary to check the compression struts with Expression (6.241) below:

$$v_{Ed} \leq v_{Rd,max} \tag{6.241}$$

In the case of non-reinforced UHPFRC:

$$v_{Rd,max} = 2.3 \times \alpha_{CC} \frac{f_{ck}^{2/3}}{\gamma_C} . \frac{\tan\theta_f}{h_f/h_{fs}} \tag{6.242}$$

For UHPFRC with shear reinforcement, Expression (6.242) must be changed in the same way as (6.215) is changed to move on to (6.216).

(5) When the shear between flanges and web is combined with transverse bending, the tensioned UHPFRC located in the area used to resist the longitudinal shear v_{Ed} (area defined by the height h_{fs}) cannot be used to resist the transverse bending. Reinforcing steel located in the same area used to resist shear cannot be used to withstand transverse bending.

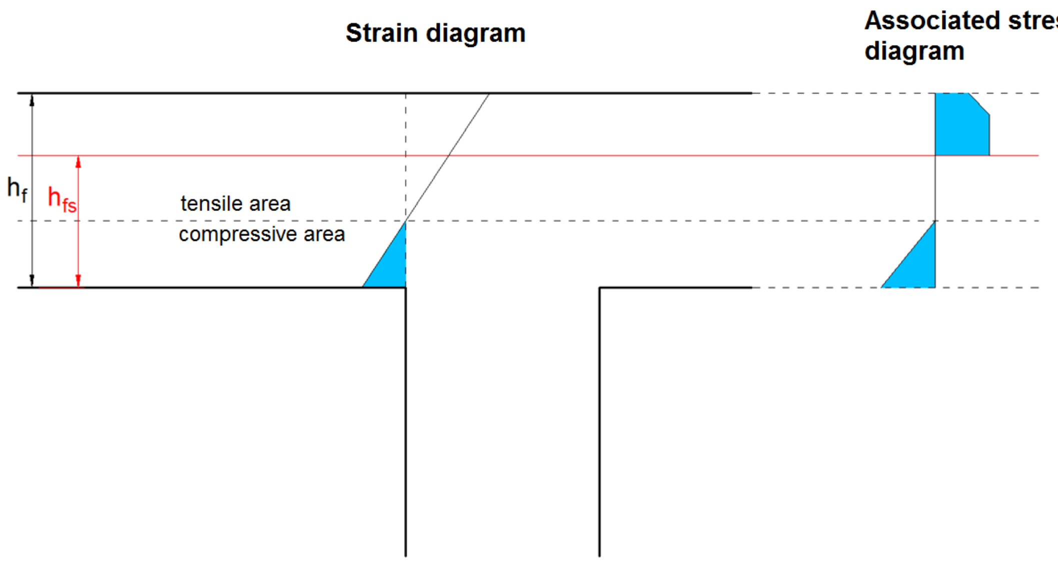


Figure 6.203 — Diagram of the web/flange joint

(6) Does not apply

(7) Unchanged

6.2.5 Shear at the interface between concrete cast at different times

(1) At the interface between two different UHPFRCs, not including the requirements of 6.2.1 to 6.2.4, the following inequality must also be verified:

$$v_{Edi} \leq v_{Rdi} \tag{6.23}$$

v_{Edi} is the design value of the shear stress at the interface; it is given by:

$$v_{Edi} = \beta\, V_{Ed} / (z\, b_i) \tag{6.24}$$

where:

β is the ratio of the axial force (longitudinal) in the new UHPFRC area and the total longitudinal force either in the compression or tension zone, both calculated for the section considered

V_{Ed} is the transverse shear force

z is the lever arm of the internal forces in the composite section

b_i is the width of the interface (see Figures 6.204 and 6.205)

v_{Edi} is the design shear resistance at the interface

If there are no indentations:

$$v_{Rdi} = c\, f_{ctk,el}/\gamma_C + \mu\, \sigma_n + \rho\, f_{yd}\, (\mu \sin\alpha + \cos\alpha) \leq 1.15.\alpha_{cc}.f_{ck}^{2/3}/\gamma_C \tag{6.243}$$

where:

c and μ are factors which depend on the roughness of the interface (see (2))

σ_n is the stress caused by the minimum external axial force across the interface that can act simultaneously with the shear force, positive for compression, such that $\sigma_n < 0.6\, f_{cd}$ and negative for tension. When σ_n is tensile, $c\, f_{ctk,el}/\gamma_C$ should be taken as 0.

$\rho = A_s / A_i$

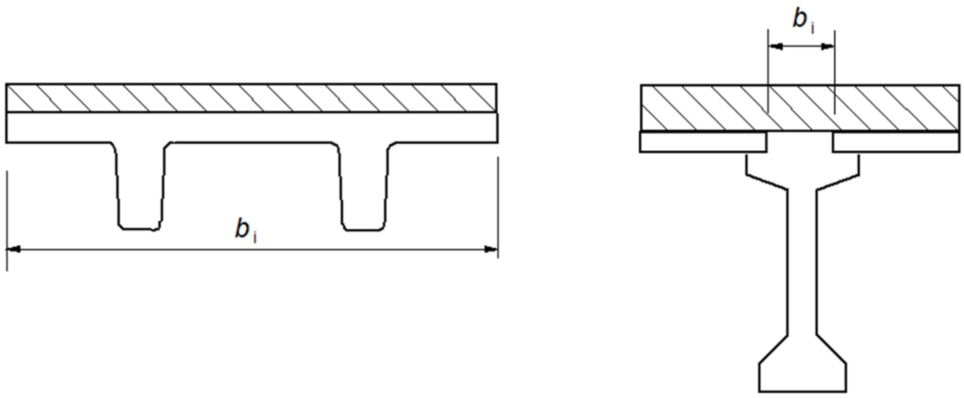


Figure 6.204 — Examples of joint area in composite section

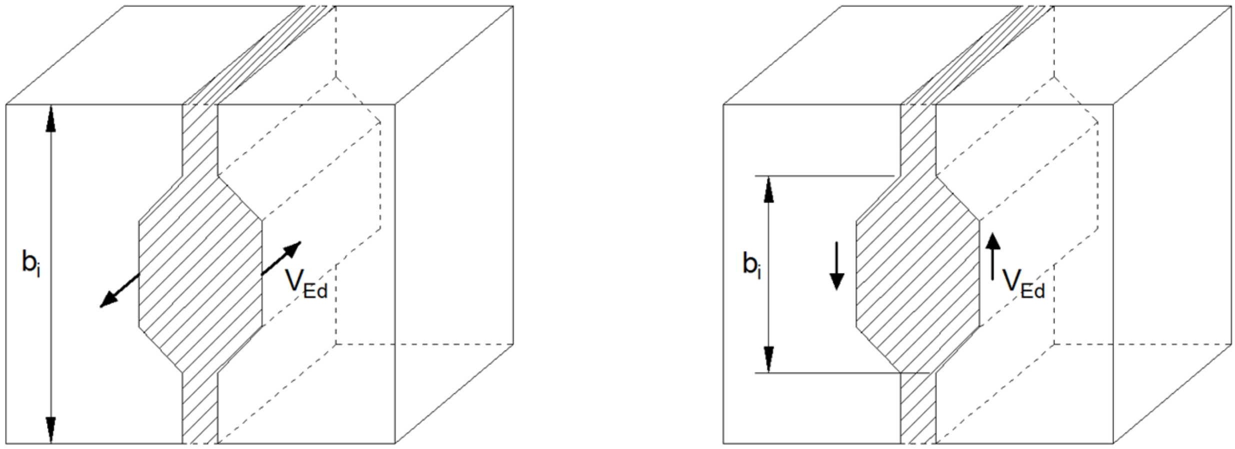


Figure 6.205 — Example de concreting joint areas

A_s is the area of reinforcement crossing the interlace, including ordinary shear reinforcement (if any), with adequate anchorage at both sides of the interface

A_i is the area of the joint

α defined in Figure 6.9 of standard NF EN 1992-1-1; α must be limited such that $45° \le \alpha \le 90°$

NOTE If there are no indentations, if the UHPFRCs either side of the interface have different properties, v_{Rdi} is the smallest value of the shear resistance.

For indentations:

$$v_{Rdi} = c\frac{f_{ctkel}}{\gamma_C} + \mu.\sigma_n + \rho.f_{yd}(\mu\sin\alpha + \cos\alpha) + (0.35\mu + 0.3)\frac{f_{ctfk}}{K.\gamma_{cf}} \le 1.15.\alpha_{cc}.\frac{f_{ck}^{2/3}}{\gamma_C} \tag{6.244}$$

K is the orientation factor of the fibres in the indented area in the direction perpendicular to the joint area. This orientation factor is not necessarily that of the overall structure.

The height of the indentations must satisfy:

$$d \ge L_f/2 \tag{6.245}$$

The lengths and width of the indentations must fulfil the following conditions:

$$h_1 \geq 2.L_f \tag{6.246}$$

$$h_2 \geq 2.L_f \tag{6.247}$$

$$b \geq 2.L_f \tag{6.248}$$

$$h_1 \leq 10.d \tag{6.249}$$

$$h_2 \leq 10.d \tag{6.250}$$

where h_1, h_2 are defined in Figure (6.204) below.

b is the width of the indentation

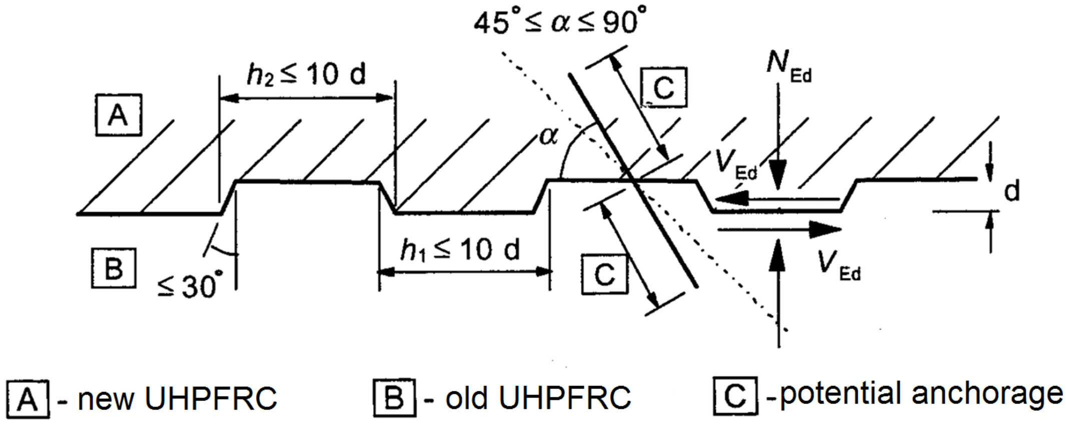


Figure 6.206 — Indented construction joint

NOTE Attention is focused to the flow conditions when casting the UHPFRC which must enable the indentations to be filled satisfactorily and the fibres to be distributed and orientated correctly.

(2) If no detailed information is available, surfaces in UHPFRC are classified as very smooth where c = 0.025 to 0.10 and $\mu = 0.5$;

For UHPFRC surfaces with indentations fulfilling the above conditions: c = 0.5 and $\mu = 1.4$

(3) Unchanged

(4) Unchanged

(5) Unchanged, including (105) of standard NF EN 1992-2

(6) Addition: In the case of a joint between a UHPFRC and an ordinary concrete, the design shear resistance at the interface is given by Expression (6.25) and standard NF EN 1992-1-1, i.e. the term relating to ordinary concrete.

If a construction joint has indentations, the shear resistance between a member in UHPFRC (member no. 1) and a member in ordinary cement (member no. 2) is expressed as:

$$V_{Rd} = \mathrm{Min}(\ v_{Rd,1}\ \Sigma_i\ b_{1,i}\ h_{1,i}\ ;\ v_{Rd,2}\ \Sigma_i\ b_{2,i}\ h_{2,i}\ ) \tag{6.251}$$

In this expression:

$v_{Rd,1}$ is the value of the shear resistance for member 1. $v_{Rd,1}$ is bounded by the value $1.15.\alpha_{cc}.f_{ck}^{2/3}/\gamma_C$
$v_{Rd,2}$ is the value of the shear resistance for member 2. $v_{Rd,2}$ is bounded by the value $0.5\ \upsilon\ f_{cd}$
$b_{1,i}$ is the width of indentation i of member 1
$h_{1,i}$ is the height of indentation i of member 1
$b_{2,i}$ is the width of indentation i of member 2

$h_{2,i}$ is the height of indentation i of member 2

6.3 Torsion

6.3.1 General

(1)P Where the static equilibrium of a structure depends on the torsional resistance of elements of the structure, a full torsional design at the ultimate limit state shall be made.

(2) Where, in statically indeterminate structures, torsion arises from consideration of compatibility only, and the structure is not dependent on the torsional resistance for its stability, then it will normally be unnecessary to consider torsion at the ultimate limit state. Excessive cracking is avoided by fulfilling the non-brittleness condition given in 9.1 (3).

(3) The torsional resistance of a section may be calculated on the basis of one or more thin-wall closed sections for which a dummy thickness t_{ef} is defined and in which equilibrium is satisfied by a closed shear flow. For hollow sections, the thickness of the dummy wall should be limited to the actual thickness.

Solid convex sections may be modelled directly by equivalent thin-walled closed sections. Solid non-convex sections, such as T sections, may be first of all broken down into sub-sections, each modelled using an equivalent thin-wall section, the torsional resistance of the assembly being taken equal to the sum of the resistances of the sub-sections.

(4) The distribution of the acting torsional moments over the sub-sections should be in proportion to their uncracked torsional stiffnesses.

(5) Each sub-section may be calculated separately by superimposing the shear forces due to torsion with those due to the shearing action. To carry out superimposition of hollow and convex solid sections, the wall may need to be broken down into a number of members. A distinction must also be made between compressed and tensioned sections at ULS for bending with axial force.

6.3.2 Design procedure

(1) For hollow and convex solid sections subject to torsional moment T_{Ed}, the torsional shear flow at all points on the wall is calculated using the following expression:

$$\tau_t t_{ef} = T_{Ed} / 2 A_k$$

Where τ_t is the tangential stress in the wall

t_{ef} is the thickness of the dummy wall equal a sixth of the diameter of the largest circle able to be inscribed within the outer contour of the section

A_k is the area limited by the centre line of the dummy tubular section (including hollow section)

In the case of non-convex solid sections broken down into sub-sections, the torsional shear flow in the wall of sub-section i, subject to fraction $T_{Ed,i}$ of the design torsional moment, is calculated by the expression:

$$\tau_{ti} t_{ef,i} = T_{Ed,i} / 2 A_{ki}$$

Where τ_{ti} is the shear in member i

$t_{ef,i}$ is the dummy thickness of the wall equal to a sixth of the total thickness of the member

A_{ki} is the area limited by the centre line of the dummy tubular section (including hollow section).

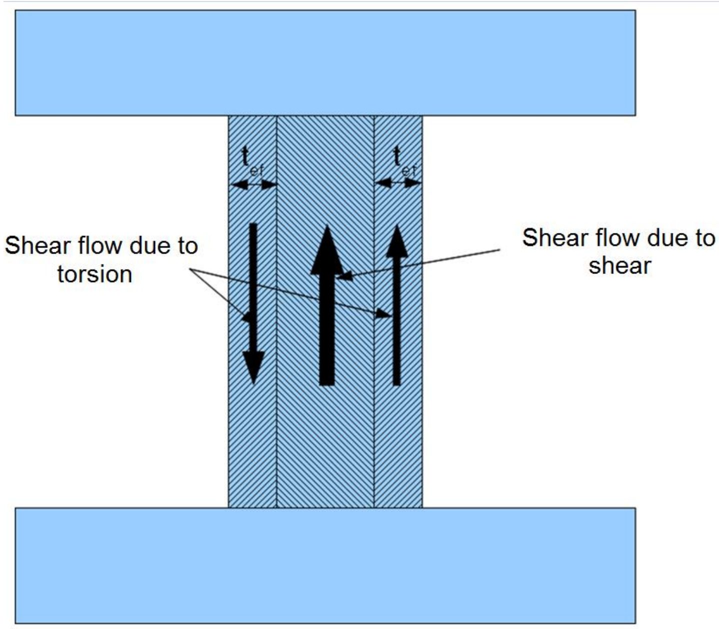


Figure 6.207 — Shear flow due to torsion and shear

(2) The torsion effects may be superimposed with those of the shear force taking the same value for the compression strut inclination θ. This is evaluated as shown in 6.2.1.3 from the elastic stress calculation.

Addition: The area A_{sw} of the cross-section of the transverse reinforcement which possibly supplement the contribution of the fibres to torsional resistant moment only, may be calculated by means of the expression:

$$t_{ef}\sigma_{Rd,f} + \frac{A_{sw}f_{yd}}{s} = \frac{T_{Ed}}{2A_k \cot\theta} \tag{6.252}$$

$\sigma_{Rd,f}$ is the mean value of the post-cracking strength defined in 6.2.1.4 (1). In this expression the contribution of the UHPFRC is ignored (other than that of the fibres), in an analogous way to the resistance term $V_{Rd,c}$ for shear, which is done for safety reasons.

In a member with a non-convex solid cross-section, such as the web of a T-beam (Figure 6.207), a thickness t_{ef} may be defined for the section where the shear stresses due to torsion and the shear force are of the same sign and on which it is conventionally accepted that the fibres contribute to resist only torsional moment, according to Expression (6.252). It is conventionally accepted that the fibres contribute to resist shear force in the central part and in the thickness t_{ef} where the stresses have opposite signs, according to Expressions (6.209) to (6.214) in which the total width b_w is replaced by a reduced width $b'_w = b_w - t_{ef}$. Condition $V_{Ed} \leq V_{Rd}$ given 6.2 must be satisfied.

For hollow and convex solid sections, each member of wall i should be designed to be of length z_i for the combined shear and torsion forces. The following formula may be applied to check each wall:

$$V_{Rd,c} + V_{Rd,f} + V_{Rd,s} \geq V_{Ed,i} + \frac{T_{Ed}}{2A_k} z_i \tag{6.253}$$

The term $V_{Ed,i}$ is the force acting parallel to wall member i, resulting from shear forces.

In the resistance term $V_{Rd,f}$ and for UHPFRCs of class T1* or T2*, the mean value of the post-cracking strength $\sigma_{Rd,f}$ is defined by the expression:

$$\sigma_{Rd,f} = \frac{1}{K_{global}\gamma_{cf}\left(w_2^* - w_1^*\right)} \int_{w_1^*}^{w_2^*} \sigma_f(w)dw \tag{6.254}$$

w^*_1, w^*_2: crack widths at ends 1 and 2 of the length z_i of the wall or wall member considered, which must satisfy:

$$w^*_j = \max(w_{uj}\ ;\ w_{max}) \tag{6.255}$$

w_{uj} being the design opening under ULS bending stresses comprising at ends j = 1 or 2 of the length z_i et w_{max} the maximum permissible opening of the cracks according to 7.3.1.

K_{global}: orientation factor for the direction perpendicular to compression struts of inclination θ_i.

In the case of UHPFRCs of class T3*, the expression for $\sigma_{Rd,f}$ becomes:

$$\sigma_{Rd,f} = \frac{1}{K_{global}\gamma_{cf}\left(\varepsilon_2^* - \varepsilon_1^*\right)}\int_{\varepsilon_1^*}^{\varepsilon_2^*}\sigma_f(\varepsilon)d\varepsilon \tag{6.256}$$

(3) The cross-sectional area of the longitudinal torsion reinforcement $\sum A_{sl}$ may be calculated using the expression:

$$\frac{A_k\,\sigma_{Rd,f} + \sum A_{sl}\,f_{yd}}{u_k} = \frac{T_{Ed}}{2A_k}\cot\theta \tag{6.257}$$

A_k is the area of the mean layer defined in 6.3.2(1)

$\sigma_{Rd,f}$ is the mean value of the post-cracking strength defined by the formulae of 6.3.2 (2) above.

In compressed flanges, longitudinal reinforcement if applicable may be reduced in proportion to the compression force available. In tensioned flanges, the longitudinal torsion reinforcement should be added to the other reinforcement. Longitudinal reinforcement should in general be distributed over the length z_i, but for small cross-sections, they may be concentrated at the ends of the sides.

Likewise in tensioned flanges, the term $A_k\ \sigma_{Rd,f}$ in (6.257) must be reduced to take account of the longitudinal tensile force.

The same goes for the contribution of the fibres: it is possible to reduce their contribution if the part is tensioned. The term $\sigma_{Rd,f}$ may thus be used both for the longitudinal tension and torsion.

Bonded prestressing tendons may be allowed for by limiting the increase in their stress to $\Delta\sigma_p$ < 500 MPa. In this case, $\sum A_{sl}\,f_{yd}$ in Expression (6.257) is replaced by $\sum A_{sl}\,f_{yd} + A_p\,\Delta\sigma_p$.

(4) The strength of a member subject to shear and torsion is limited by the strength of the concrete compression struts. The following condition must be satisfied in order not to exceed this strength for solid sections:

$$\frac{T_{Ed}}{T_{Rd,max}} + \frac{V_{Ed}}{V_{Rd,max}} \le 1 \tag{6.258}$$

Where:

T_{Ed} is the acting design torsional moment

V_{Ed} is the acting design shear

$T_{Rd,max}$ is the limit resisting torsional moment. In the case of UHPFRC without reinforcement, it is given by:

$$T_{Rd,max} = 2.3\frac{\alpha_{CC}}{\gamma_C} \times 2A_k t_{ef\,i} f_{ck}^{2/3}\tan\theta \tag{6.259}$$

$V_{Rd,max}$ is the maximum value of the limit resisting shear according to Expression (6.215).

For hollow and convex solid sections, each wall should be designed separately for the combined shear and torsion forces. The compression ultimate limit state of the UHPFRC compression struts should be checked against the limit strength $V_{Rd,max}$.

(5) Does not apply

6.3.3 Warping torsion

(1) Unchanged

(2) Unchanged

6.4 Punching

(1) Part 6.4 of the present standard fully replaces part 6.4 of standard NF EN 1992-1-1.

(2) Considering a reference contour located at a distance h/2 from the loaded area, the mean shear stress τ in the UHPFRC must be less than:

$$\tau_{max} = \frac{0.8}{\gamma_{cf}} Min\left(\frac{f_{ctfk}}{K_{local}} ; f_{ctk,el} \right) \tag{6.260}$$

NOTE This verification is valid whether the UHPFRC is non-reinforced, reinforced or prestressed.

6.5 Design with strut and tie models

6.5.1 General

(1) Part 6.5 of the present standard fully replaces part 6.5 of standard NF EN 1992-1-1.

Where a non-linear strain distribution exists (e.g. supports, near concentrated loads or planar stress) strut-and-tie models may be used. The strut and tie method can be used if it is demonstrated that the forces path in the strut and tie model relates to forces path in an elastic analysis.

6.5.2 Struts

(1) The maximum stress in a compressed strut is set to $f_{cd}=f_{ck}/\gamma_C$ where the strut is subject to a positive or null transverse stress and to $2.3.\alpha_{cc}.f_{ck}^{2/3}/\gamma_c$ when the strut is subject to a negative transverse stress (tensile) for consistency with the term $V_{Rd,max}$ using in verifying shear.

6.5.3 Ties

(1) Ties may be comprised of reinforcing steel, but may also be implemented through the friction force contributed by the fibres. In the latter case, the force on the tie is $A_t.\sigma_{Rd,f}$ where A_t is the area of the tie considered.

In calculations on ties, $\sigma_{Rd,f}$ is the mean value of the post-cracking strength defined in 6.2.1.4 (1) calculated using K_{local}. In addition, for UHPFRCs of class T1* and T2*, the stress $\sigma_{Rd,f}$ is calculated using w* = 0.3 mm.

NOTE In cases where UHPFRC is used over extensive areas where a localised fault would not have an effect, it is possible to calculate $\sigma_{Rd,f}$ using K_{global}.

6.5.4 Nodes

(1) When the node is subject to compression only, the maximum stress is taken as $f_{cd}=f_{ck}/\gamma_C$.

When the node is subject to compression and traction, the maximum stress is set to $2.3.\alpha_{cc}.f_{ck}^{2/3}/\gamma_C$.

6.6 Anchorages and laps

(1)P Unchanged

(2) Unchanged

(3) Unchanged

6.7 Partially loaded areas

(1)P Unchanged

(2) For a uniform distribution of load on an area A_{c0} (see Figure 6.29 of standard NF EN 1992-1-1), the limit compression force may be determined as follows:

$$F_{Rdu} = A_{c0} \cdot \frac{0.46.f_{ck}^{2/3}}{1+0.1.f_{ck}}.f_{cd} \cdot \sqrt{A_{c1}/A_{c0}} \leq 3,0 \cdot \frac{0.46.f_{ck}^{2/3}}{1+0.1.f_{ck}}.f_{cd} \cdot A_{c0} \qquad (6.261)$$

where:

A_{c0} is the loaded area,

A_{c1} is the maximum design distribution area with a similar shape to A_{c0}

(3) Unchanged

(4) The transverse tensile forces due to the load effect must be absorbed by tensioned UHPFRC and distributed reinforcement if applicable.

(105) Support areas for civil engineering structures should be designed using recognised methods.

NOTE Additional information maybe found in Annex J.

6.8 Fatigue

6.8.1 Verification conditions

(1)P Unchanged

(2) A fatigue verification should be carried out for structures and structural components which are subjected to regular load cycles.

The following structures are exempt from fatigue verification:

Standard buildings, except those with thin members on the façade;
Foundations, retaining walls and embedded walls;
Buried structures with a minimum covering of 1 m of earth;
Piers and columns not rigidly connected to superstructures, bridge piers not rigidly connected to the deck;
Arch and bridge abutments with the exception of hollow abutments.

(3) The fatigue strength is verified by satisfying stress criteria based on:

- the range of and/or maximum compressive strength of the compressed UHPFRC
- the maximum tensile stress of the UHPFRC
- the maximum shear stress of the UHPFRC
- the range of and/or maximum reinforcing steel stresses, if applicable
- the range of variations in the stress of prestressing tendons, if applicable.

The following arrangements are deemed to ensure the fatigue strength of UHPFRCs or reinforcement independently of the number of cycles:

- UHPFRC compressed when $\sigma_c < 0.6\ f_{ck}$ under characteristic SLS combination of actions
- reinforcement tensioned in reinforced UHPFRC sections, when σ_s < 300 MPa under characteristic SLS combination of actions
- prestressing tendons and reinforcing steel, in areas where the end fibres of the UHPFRC section remain compressed under a frequent combination of actions with P_m

- shear reinforcement, where the reinforcement has been designed at ULS using a diagram of compression struts at inclination θ such that $1.0 \leq \cot\theta \leq 1.5$

6.8.2 Internal forces and stresses for fatigue verification

(1)P The stress calculation shall be based on the assumption of cracked cross sections. The contribution of tensioned UHPFRC must be allowed for by using the SLS stress-strain laws described in 3.1.7.3. Stresses in the UHPFRC and any reinforcement if applicable must be calculated by assuming compatibility of strains.

(2)P The effect of different bond behaviour of prestressing and reinforcing steel shall be taken into account by increasing the stress range in the reinforcing steel calculated under the assumption of perfect bond by the factor η:

$$\eta = \frac{A_S + A_P}{A_S + A_P\sqrt{\xi(\phi_S/\phi_P)}} \qquad (6.64)$$

where:

A_s is the area of reinforcing steel

A_p is the area of prestressing tendon or tendons

ϕ_S is the actual or equivalent diameter of reinforcing steel

ϕ_P is the actual or equivalent diameter of prestressing tendons

$\phi_P = 1.6\,\sqrt{A_P}$ for bundles

$\phi_P = 1.75\,\phi_{wire}$ for single 7 wire strands

$\phi_P = 1.20\,\phi_{wire}$ for single 3 wire strands

where ϕ_{wire} is the wire diameter

ξ is the ratio of the bond strength of the bonded prestressing tendons to that of the high strength reinforcement in the UHPFRC. The value is subject to the relevant European Technical Approval. In the absence of this the values given in Table 6.201 may be used.

Tableau 6.201 — Ratio of bond strength ξ between tendons and reinforcing steel

Prestressing steel	ξ	
	pre-tensioned	bonded, post-tensioned
Smooth bars and wires	0.35	0.11
Strands	0.6	0.18
Indented wires	0.7	0.22
Ribbed bars	0.8	0.24

The equivalent diameter $Ø_{éq}$ of a number of reinforcing steel bars may be obtained by applying the following expression:

$$Ø_{éq} = \sum Ø_i^2 / \sum Ø_i \qquad (6.262)$$

$Ø_i$ diameter of reinforcement i in the group

(3) Unchanged

6.8.3 Combination of actions

(1)P Unchanged

(2)P Unchanged

(3)P Unchanged

6.8.4 Verification procedure for reinforcing and prestressing steel

(1) Unchanged, including the National Annex

(2) Unchanged

(3)P Unchanged

(4) Unchanged

(5) Unchanged, including the National Annex

(6)P Unchanged

(107) Unchanged

6.8.5 Verification using damage equivalent stress range

(1) Instead of an explicit verification of the damage strength according to 6.8.4, the fatigue verification of reinforcement in standard cases with known loads (railway and road bridges) may also be performed using the damage equivalent stress ranges as described in 6.8.5 (3)

(2) Unchanged

(3) Unchanged

6.8.6 Other verifications

(1) Unchanged, including the National Annex

(2) Unchanged

(3) Unchanged, including the National Annex

6.8.7 UHPFRC verifications

(1) Does not apply

(2) The fatigue verification for UHPFRC under compression may be assumed, if the following condition is satisfied:

$$\frac{\sigma_{c,max}}{f_{ck}} \leq 0.4 + 0.4\frac{\sigma_{c,min}}{f_{ck}} \tag{6.263}$$

where:

$\sigma_{c,max}$ is the maximum compressive stress under a frequent load combination (compression measured positive)

$\sigma_{c,min}$ is the minimum compressive stress in the same direction. If the minimum stress is tensile, $\sigma_{c,min}$ should be taken as 0.

(3) Does not apply

(4) Does not apply

(5) Addition: It may be assumed that the fatigue strength of tensioned UHPFRC is satisfactory if the normal tensile stress at SLS is limited to 0.95 min($f_{ctk,el}$; f_{ctfk} /K) both regarding the frequent combination of actions for designs sustainable over the long term and the characteristic combination of actions associated with the construction phases.

(6) Addition: It may be assumed that the fatigue strength of UHPFRC under shear is satisfactory if the sum of the shear stresses due to shear and torsion at SLS is limited to 0.95 min($f_{ctk,el}$; f_{ctfk} /K) both regarding the frequent combination of actions for designs sustainable over the long term and the characteristic combination of actions associated with the construction phases.

7 SERVICEABILITY LIMIT STATES

7.1 General

(1)P Unchanged

(2) In the calculation of stresses and deflections, cross-sections should be assumed to be uncracked provided that the flexural tensile stress does not exceed $f_{ctk,el}$.

(3) Addition: The calculation laws given in 3.1.7 (defined with the characteristic stress values) are used for stress verifications described in 7.2 (mainly in reinforcement).

For the verification of crack widths under 7.3, calculation laws using "mean values" defined in 3.1.7.3.1 (9) are used with the exception of UHPFRCs of class T2* where the characteristic values are retained.

(4) Addition: stress calculations shall be performed by including the contribution of tensioned UHPFRC in accordance with the stress laws - SLS strains described in 3.1.7.3. Stresses in the UHPFRC and any reinforcement if applicable must be calculated by assuming compatibility of strains. For reinforced and prestressed UHPFRC, the stress calculation shall also take into account bonding factors between the UHPFRC and reinforcement given in 6.8.2.

7.2 Stress limitation

(1)P Unchanged

(2) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1 and (102) of standard NF EN 1992-2

(3) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(4) Unchanged

(5) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

NOTE Clearly relates only to the case where the UHPFRC is coupled with reinforcing steel or prestressing tendons. Where the UHPFRC is coupled with bonded reinforcement, the stress limitation of these reinforcement does not dispense with the need to verify crack widths and deflections.

(6) Addition: for non-reinforced and non-prestressed UHPFRCs, it is not necessary to consider another tensile stress limitation for fibre-reinforced UHPFRC other than that which results from the crack width limitation given in 7.3.

(7) Addition: In the case of bending with axial force with a axial tensile force on a thin member, the tensile strain adjacent to the centreline must not be greater than ε_{lim} / 2 unless demonstrated otherwise.

7.3 Crack control

7.3.1 General considerations

(1)P Unchanged

(2) Unchanged

(3) Unchanged

(4) Unchanged

(5) The limiting calculated crack width w_{max} is given in Table 7.201 below.

Table 7.201 — Recommended values of w_{max} (mm)

Exposure class	Members in reinforced UHPFRC and members in prestressed UHPFRC with unbonded tendons	Members in prestressed UHPFRC with bonded tendons	Members in non-reinforced and non-prestressed UHPFRC	
	Quasi-permanent load combination	Frequent load combination	Characteristic load combination	Frequent load combination
X0, XC1	0.3	0.2	0.3	0.3
XC2, XC3, XC4	0.2	0.1	0.2	0.1
XD1, XD2, XD3 XS1, XS2, XS3	0.1	Tensile limitation to 2/3.min($f_{ctm,el}$, f_{ctfm}/K_{global})	0.1	0.05

(6) Unchanged

(7) Unchanged

(8) When using strut-and-tie models with the struts oriented according to the compressive stress trajectories in the uncracked state, it is possible to use the forces in the ties to obtain the corresponding steel stresses, if applicable, to estimate the crack width.

(9) Does not apply

(110) For members in UHPFRC, shear cracking must be controlled by application of Annex QQ. This verification relates to civil engineering structures but is not generally necessary for buildings.

7.3.2 Minimum reinforcement areas

(1)P Members in UHPFRC do not require a minimum quantity of steel reinforcement to control cracking. This is assumed to be provided by the checks in 7.3.4 and the sufficiently ductile character under tension of the UHPFRCs covered by the present standard (see 1.1 (4)).

(2) Does not apply

(3) Does not apply

7.3.3 Control of cracking without direct calculation

(1) Does not apply

(2) Does not apply

(3) Does not apply

(4) Does not apply

(5) Does not apply

7.3.4 Calculation of crack widths

(1) Part 7.3.4 of the present standard fully replaces part 7.3.4 of standard NF EN 1992-1-1. The present part 7.3.4 is only to be applied to thick members in UHPFRC of class T1* and T2*. The verification of crack widths is not necessary for thin members and UHPFRCs of class T3*.

(2) For non-reinforced UHPFRCs, the following inequality must be satisfied:

$$w_{t,a} = \left(\varepsilon_{t,a} - \frac{f_{ctm,el}}{K.E_{cm}} \right) . L_c \leq w_{max} \qquad (7.201)$$

where

$\varepsilon_{t,a}$ is the maximum strain value (as an absolute value) resulting from a section equilibrium calculation in which the UHPFRC laws are so-called mean SLS calculation laws (see 3.1.7.3.1 (9)).

For UHPFRCs of class T2*, the strain $\varepsilon_{t,a}$ shall however result from the equilibrium of the section using calculation laws with characteristic stress values and $f_{ctm,el}$ shall be replaced by $f_{ctk,el}$ in Expression (7.201).

The orientation factor K_{global} relates to the longitudinal direction.

w_{max} is the limiting calculated crack width given by Table 7.201.
L_c is the characteristic length: L_c = 2/3.h where h is the height of the section

(3) For reinforced or prestressed UHPFRCs, the following inequality shall be satisfied:

$$w_{t,b} \leq w_{max} \qquad (7.202)$$

where

w_{max} is the limiting calculated crack width given by Table 7.201.
$w_{t,b}$ is the crack width calculated at the most tensioned fibre. $w_{t,b}$ is deduced from w_s which is the crack width calculated at the most tensioned reinforcement by the following expression:

$$w_{t,b} = w_s\ (h - x - x') / (d - x - x') \qquad (7.203)$$

where h is the total height of the section
d is the useful height of the section
x is the compressed height
x' is the uncracked tensioned height (between stresses 0 and $f_{ctm,el}$)

(4) The crack width at the most tensioned reinforcement w_s is given by the following expression:

$$w_s = s_{r,max,f}\ .\ (\varepsilon_{sm,f} - \varepsilon_{cm,f}) \qquad (7.204)$$

where $s_{r,max,f}$ is the maximum crack spacing, calculated according to Expression (7.211)

$\varepsilon_{sm,f}$ is the average strain on the reinforcement under the combination of loads considered, including the effect of imposed strains and taking into account the contribution of the tensioned UHPFRC between the cracks. In the case of a prestressing tendon, only the relative elongation beyond the state equating to the absence of strain in the concrete at the same level is taken into account

$\varepsilon_{cm,f}$ is the average strain in the UHPFRC between cracks

($\varepsilon_{sm,f} - \varepsilon_{cm,f}$) is calculated according to Expression (7.205) below:

$$\varepsilon_{sm,f} - \varepsilon_{cm,f} = \frac{\sigma_s}{E_s} - \frac{f_{ctfm}}{K_{global}.E_{cm}} - \frac{1}{E_s} \left[k_t \left(f_{ctm,el} - \frac{f_{ctfm}}{K} \right) . \left(\frac{1}{\rho_{eff}} + \frac{E_s}{E_{cm}} \right) \right] \qquad (7.205)$$

σ_s is the stress in the tensioned reinforcing steel, calculated for UHPFRCs by using the so-called mean SLS calculation laws. (see 3.1.7.3.1 (9)). For members in UHPFRC prestressed by pre-tension, σ_s may be replaced by

$\Delta\sigma_p$, stress variation in prestressing tendons from the state equating to the absence of strain in the UHPFRC at the same level up to the limit of elasticity $f_{p,0.1k}$

For UHPFRCs of class T2*, the stress σ_s must however result from the equilibrium of the section using calculation laws with characteristic stress values and $f_{ctm,el}$ must be replaced by $f_{ctk,el}$ in Expression (7.205).

$\rho_{eff} = A_s / A_{c,eff}$ for reinforcing steel

$\rho_{eff} = A_p / A_{c,eff}$ for prestressing tendons

$A_{c,eff}$ is the effective cross-sectional area of UHPFRC around the tensioned reinforcement, of height $h_{c,ef}$ equal to the smallest of the following two values: 2.5 (h –d) or h/2 (see Figure 7.201)

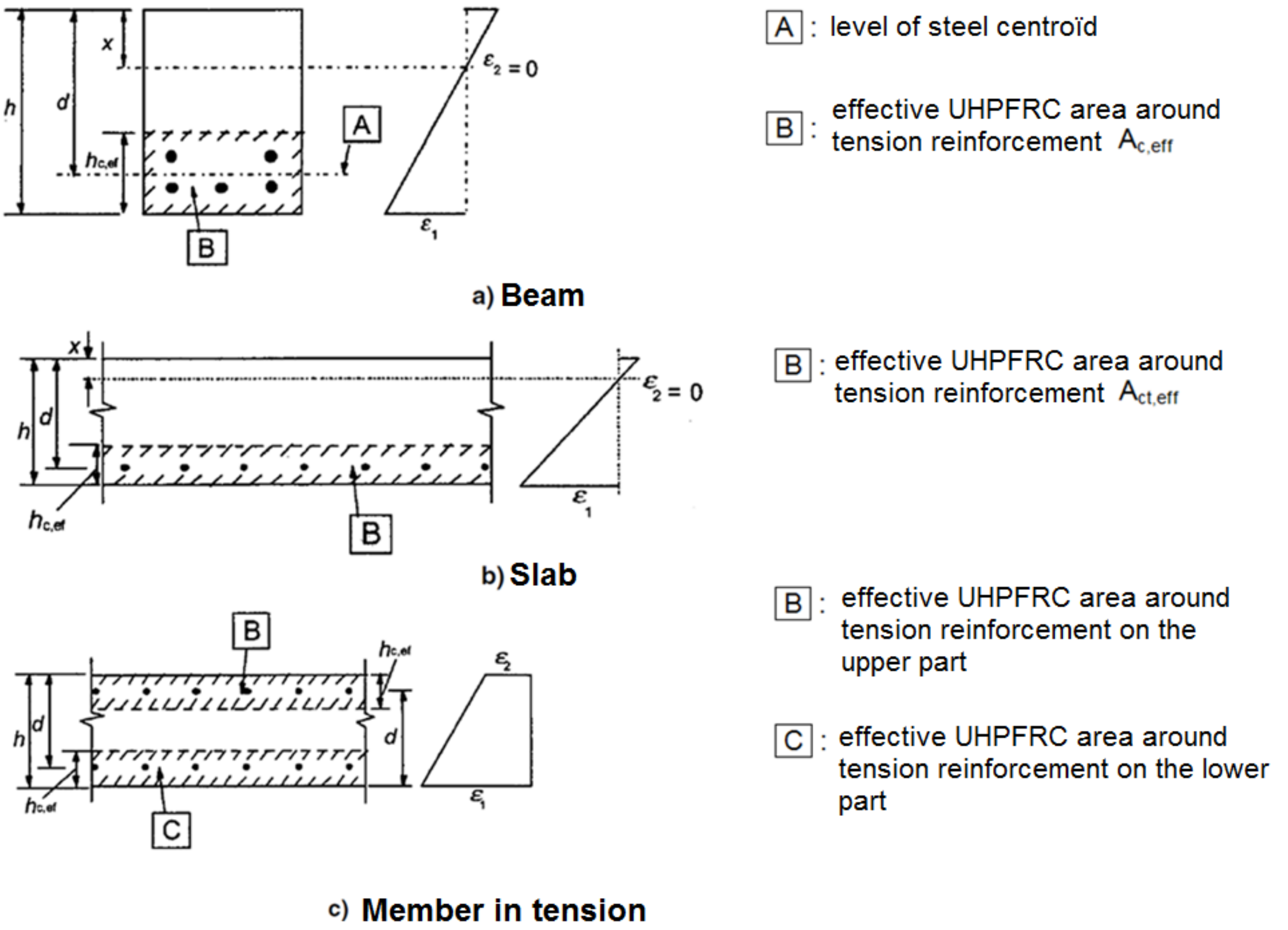


Figure 7.201 — Effective UHPFRC cross-sections around tensioned reinforcement (standard cases)

A_s or A_p are the total areas of reinforcement steel or prestressing tendons located in UHPFRC area $A_{c,eff}$.

k_t is a factor dependent on the duration of the load or its repetition: $k_t = 0.6$ in the case of a short duration load; $k_t = 0.4$ in the case of a long duration load applied while the concrete is still new, or in the case of repeated high-amplitude loads.

For a layer of tensioned reinforcement at spacing s parallel to the concrete face, the calculation according to Expressions (7.204) and (7.205) is sufficient if the spacing satisfies the condition:

$$s \leq 5 (c + Ø / 2)$$

where c is the concrete cover and Ø the diameter of the reinforcement.

(5) If the spacing s is between 5 (c + Ø/2) and 10 (c + Ø/2), the maximum crack width shall be calculated half-way between the reinforcement from the crack width calculated adjacent to the reinforcement $w_{t,b}$ using the following expression:

$$w_1 = \left| 1 + 0.015\,\alpha \cdot \beta \cdot \left[\frac{s}{c + \frac{\phi}{2}} \right]^2 \right| w_{t,b} \tag{7.206}$$

Where:

$$\alpha = 1 - 0.5\, f_{ctfm} / (K\, f_{ctm,el}) > 0 \tag{7.207}$$

And

$$\beta = \frac{100\,\rho_{eff}}{100\,\rho_{eff} + f_{ctfm}/(K.f_{ctm,el})} \tag{7.208}$$

The corrected crack width w_1 shall satisfy:

$$w_1 \leq w_{max} \tag{7.209}$$

If the spacing of the reinforcement is greater than 10 (c + Ø / 2), the corrected crack opening w_1 shall be calculated with s = 10 (c + Ø / 2).

The orientation factor K_{global} relates to the longitudinal direction. For UHPFRCs of class T2*, $f_{ctm,el}$ and f_{ctfm} shall be replaced by $f_{ctk,el}$ and f_{ctfk} in formulae (7.207) and (7.208).

(6) When prestressing tendons and steel reinforcement are combined, the expression of ρ_{eff} to be used is given by the following expression:

$$\rho_{eff} = \frac{A_s + \xi \frac{\phi_s}{\phi_s} A_s}{A_{c,eff}} \tag{7.210}$$

ϕ_s is the diameter of the steel reinforcement

ϕ_p is the diameter of the prestressing tendon

ξ is defined in Table 6.201 in 6.8.2

When a number of steel reinforcement or prestressing tendons are used, it is possible to bring them back to equivalent diameters in accordance with the rules given in 6.8.2.

(7) The maximum spacing between cracks may be calculated using Expression (7.211) which adds a concrete coating term l_0 and a transmission length term l_t, in the case where all reinforcement has the same diameter and same bonding:

$$s_{r,max,f} = 2.55\,(l_0 + l_t) \tag{7.211}$$

where $l_0 = 1.33\ .\ c / \delta$

$$l_t = 2 \times \left| 0.3 k_2 \left(1 - \frac{f_{ctfm}}{K_{global} f_{ctm,el}} \right) \frac{1}{\delta \cdot \eta} \right| \frac{\phi}{\rho_{eff}} \geq \frac{L_f}{2} \tag{7.212}$$

$$\delta = 1 + 0.4\left(\frac{f_{ctfm}}{K'_{global}\, f_{ctm,el}}\right) \leq 1.5 \qquad (7.213)$$

where:

c is the concrete coating for reinforcement of diameter Ø

η is equal to 2.25 in the case of a steel reinforcement and 2.25 ξ for a prestressing tendon (ξ being defined in Table 6.201).

δ is a parameter which expresses the improvement contributed by the fibres in the behaviour of the concrete cover area and to the bonding of the reinforcement

The factor K_{global} corresponds to the longitudinal direction of the reinforcement and K'_{global} to the transverse direction.

k_2 is a factor which takes account of the distribution of strains ε in the cracked section; k_2 = 1 for pure tension and 0.5 for bending with or without axial force with a partially compressed section; $k_2 = (\varepsilon_1 + \varepsilon_2) / 2\, \varepsilon_1$ for bending with axial force with the section fully tensioned, ε_1 and ε_2 being respectively the longest and shortest elongation of the end fibres in the section.

For UHPFRCs of class T2*, $f_{ctm,el}$ and f_{ctfm} must be replaced by $f_{ctk,el}$ and f_{ctfk} in formulae (7.212) and (7.213).

When prestressing tendons (actual or equivalent diameter $Ø_p$, bonding factor η_p) and reinforcing steel (actual or equivalent diameter $Ø_s$, bonding factor η_s) are combined, the term l_t of formula (7.211) is replaced by:

$$l_t = \frac{0.3\,\dfrac{k_2}{\delta}\left(1 - \dfrac{f_{ctfm}}{K \cdot f_{ctm,el}}\right)}{\xi_1 \eta_s \dfrac{\rho_{s,eff}}{\phi_s} + \eta_p \dfrac{\rho_{p,eff}}{\phi_p}} \geq \frac{L_f}{2} \qquad (7.214)$$

where:

$\xi_1 \leq \sqrt{\xi \dfrac{\phi_s}{\phi_p}}$ and ξ defined in 6.8.2.

$$\rho_{s,eff} = A_s / A_{c,eff} \qquad (7.215)$$

$$\rho_{p,eff} = A_p / A_{c,eff} \qquad (7.216)$$

For UHPFRCs of class T2*, $f_{ctm,el}$ and f_{ctfm} must be replaced by $f_{ctk,el}$ and f_{ctfk} in formula (7.214).

7.4 Deflection control

7.4.1 General considerations

(1)P Unchanged

(2) Unchanged

(3) Unchanged, including what appears in standard NF EN 1992-2

(4) Unchanged, including what appears in standard NF EN 1992-2

(5) Unchanged, including what appears in standard NF EN 1992-2

(6) Unchanged, including what appears in standard NF EN 1992-2

7.4.2 Cases where calculations may be omitted

(1)P Unchanged, including what appears in standard NF EN 1992-2

(2) Does not apply

7.4.3 Checking deflections by calculation

(1)P Unchanged

(2)P The calculation method adopted shall represent the true behaviour of the structure under relevant actions to an accuracy appropriate to the objectives of the calculation.

(3) Deflections are calculated by integrating curvatures.

In non-cracked sections (maximum tension less than $f_{ctm,el}$), the curvature is χ_I = M/EI, I being the raw inertia of the section.

For UHPFRCs of class T3* or thin members, characterised under post-cracking by their stress σ_f - average strain ε behaviour law, the equilibrium calculation in planar section gives the average curvature $\chi_{II,moy}$ directly from the strains ε.

For structures in non-reinforced class T1* and T2* UHPFRC, the equilibrium calculation in a cracked planar section from the σ_f-w law converted to the σ_f-ε law gives the maximum strain of the compressed UHPFRC ε_c at compressed height x, hence the maximum curvature $\chi_{II} = \varepsilon_c$ / x. The curvature variation is considered parabolic as a function of the distance y from the crack:

$$\chi(y) = \chi_I + (\chi_{II} - \chi_I)\left(1 - \frac{y}{h-x}\right)^2 \qquad (7.217)$$

In areas where several cracks co-exist, the average curvature is given by the following formula:

$$\chi_{II,moy} = 2\,\chi_I / 3 + \chi_{II} / 3 \qquad (7.218)$$

For structures in reinforced class T1* and T2* UHPFRC, the equilibrium calculation in a cracked planar section gives the stress σ_s of the reinforcement and the maximum curvature $\chi_{II} = \varepsilon_c$ / x. The mean elongation of the reinforcement ε_{sm} is given by the formula (7.217) and its ratio to the maximum strain $\varepsilon_{II} = \sigma_s / E_s$:

$$\varphi = \varepsilon_{sm} / \varepsilon_{II} = \varepsilon_{sm} / (\sigma_s/E_s) \qquad (7.219)$$

The mean curvature is then given by the following formula:

$$\chi_{II,moy} = \varphi\,\chi_{II} \qquad (7.220)$$

This value is valid over a beam length of the order of $s_{r,moy,f}$ / 2 either side of the section, $s_{r,moy,f}$ being the mean spacing of the cracks, equal to $s_{r,max,f}$ calculated by the formula (7.211) divided by 1.7. If reinforcement with different bonding are combined, the value of σ_s is corrected if necessary with regard to the planar section calculation.

(4) Deformations due to loading may be assessed using the tensile limit of elasticity $f_{ctm,el}$ and the effective Young's modulus for UHPFRC (see (5)).

(5) For loads with a duration causing creep in the UHPFRC, the total deformation including creep may be calculated by using the effective Young's modulus for the UHPFRC according to the following Expression:

$$E_{c,eff} = \frac{E_{cm}}{1 + \varphi(\infty, t_0)} \qquad (7.221)$$

where:

$\varphi(\infty, t_0)$ is the creep coefficient relevant for the load and time interval (see 3.1.4)

(6) To calculate curvatures due to a differential shrinkage between the two opposing faces of a beam, reference should be made to 7.4.3 (3) to (5) for the evaluation of the inertias in the cracked sections.

(7) Does not apply directly as it stands, see 7.4.3 (3).

8 DETAILING OF REINFORCEMENT AND PRESTRESSING TENDONS - GENERAL

8.1 General

(1)P Unchanged

(2) Unchanged

(3) Does not apply

(4) Unchanged

(5) Addition: this standard deals with reinforcing steel bars of diameter greater than or equal to 8 mm and less than or equal to 32 mm.

8.2 Spacing of bars

(1)P Unchanged

(2) A clear distance (e_h horizontal and e_v vertical) should be adopted between parallel bars in accordance with Figure 8.201 below:

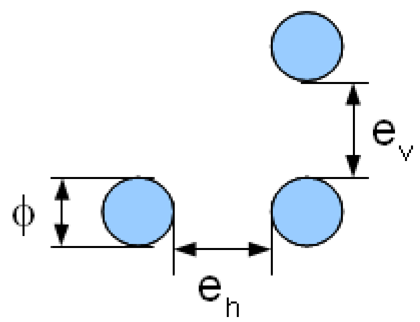


Figure 8.201 — Clear distances between reinforcement

The clear distances e_v and e_h must satisfy:

$$e_v \geq e_{mini} = \max\{\ \phi\ ;\ (D_{sup} + 5\text{ mm})\ ;\ 1.5\ L_f\ ;\ 20\text{ mm}\} \quad (8.201)$$

$$e_h \geq e_{mini} = \max\{\ \phi\ ;\ (D_{sup} + 5\text{ mm})\ ;\ 1.5\ L_f\ ;\ 20\text{ mm}\} \quad (8.202)$$

Where

D_{sup} is the nominal upper dimension of the largest aggregate (see 5.4.3 of standard NF P18-470)
ϕ is the nominal diameter of the steel reinforcement
L_f is the length of the longest fibres contributing to ensuring non-brittleness

The distance between reinforcement must be greater than 1.5 L_f to enable the UHPFRC to flow correctly.

NOTE The minimum value 1.5 L_f may be reduced without going below 1.0 L_f, subject to approval in the suitability test (visual inspection after cutting).

(3) Does not apply

(4) Unchanged

8.3 Permissible mandrel diameters for bent bars

(1)P Unchanged

(2) Unchanged

(3) The mandrel diameter need not be checked to avoid failure of the UHPFRC if the following conditions exist:

- the anchorage of the bar does not exceed max ($5\ \phi/\ \delta$; $2.5\ \phi$).
- the bar is not positioned near the surface (plane of bend close to concrete face) and there is a cross bar of diameter ≥ max($\phi/\ \delta$; $2.5\ \phi$) inside the bend.
- the mandrel diameter is at least equal to the recommended values given in Table 8.1N of standard NF EN 1992-1-1.

δ is defined in 7.3.4 (7)

Otherwise the mandrel diameter ϕ_m should be increased in accordance with Expression (8.203):

$$\phi_m \geq F_{bt} \frac{\frac{1}{a_b} + \frac{a}{2\phi}}{2.7 \cdot \delta \cdot f_{ck}^{2/3}} \qquad (8.203)$$

Where:

- F_{bt} is the tensile force from ultimate loads in a bar or group of bars in contact at the start of a bend
- a_b, for a given bar (or group of bars in contact), is half of the centre-to-centre distance between bars (or groups of bars) perpendicular to the plane of the bend. For a bar or group of bars adjacent to the face of the member, a_b should be taken as the cover plus $\phi/2$.

8.4 Anchorage of longitudinal reinforcement

8.4.1 General

(1)P Unchanged

(2) Unchanged with the following addition: on Figure 8.1 of standard NF EN 1992-1-1, the length of $5\ \phi$ must be replaced by $5\ \phi/\ \delta$, δ being defined in 7.3.4 (7).

(3) Unchanged

(4) Unchanged

(5) Unchanged

(6) Unchanged

8.4.2 Ultimate bond stress

(1)P Unchanged

(2) The design value of the ultimate bond stress, f_{bd}, for ribbed bars may be taken as:

$$f_{bd} = \eta\ \delta\ f_{ctk,el}/\gamma_c \qquad (8.204)$$

η is equal to 2.25 in the case of a steel reinforcement and 2.25 ξ for a prestressing tendon (ξ being defined in Table 6.201)

δ is defined in 7.3.4 (7) by Expression (7.213)

8.4.3 Basic anchorage length

(1)P Unchanged

(2) Unchanged

(3) Unchanged

(4) Unchanged

8.4.4 Design anchorage length

(1) The expression for l_{bd} is changed as follows:

$$l_{bd} = \alpha_1.\alpha_2.\alpha_3.\alpha_4.\alpha_5\ l_{b,rqd} + l_{tol} \geq l_{b,min} + l_{tol} \qquad (8.205)$$

l_{tol} takes into account possible positioning imperfections, its value is given by the following expression:

$$l_{tol} = \max(\phi, 10\ \text{mm}) \qquad (8.206)$$

The coefficients α_1, α_3, α_4, α_5 are defined in the same way as in standard NF EN 1992-1-1.

Coefficient α_2 is defined by the following expression:

$$0.80 \leq \alpha_2 = 1.6 - 0.4\left(\frac{c}{\phi} - 1\right) \leq 1.6 \qquad (8.207)$$

Where c is the concrete cover and Φ the nominal diameter of the bar.

The minimum anchorage length is given by the following expressions:

$l_{b,min} = \max(0.3l_{b,rqd}\ ;\ (1/\delta - 0.15).10\ \phi\ ;\ (1/\delta - 0.15).100\ \text{mm})$ for anchorages in tension
$l_{b,min} = \max(0.7l_{b,rqd}\ ;\ (1/\delta - 0.15).10\ \phi\ ;\ (1/\delta - 0.15).100\ \text{mm})$ for anchorages in compression

where δ is defined in 7.3.4 (7) by Expression (7.213)

(2) Unchanged

8.5 Anchorage of links and shear reinforcement

(1) Unchanged

(2) The anchorage should either comply with Figure 8.202 below, or in the case where rectilinear bars are placed, the inner lever arm z must be reduced by the anchorage lengths at each end of the bar. Welding must be carried out in accordance with EN ISO 17660 and have a welding capacity in accordance with 8.6 (2).

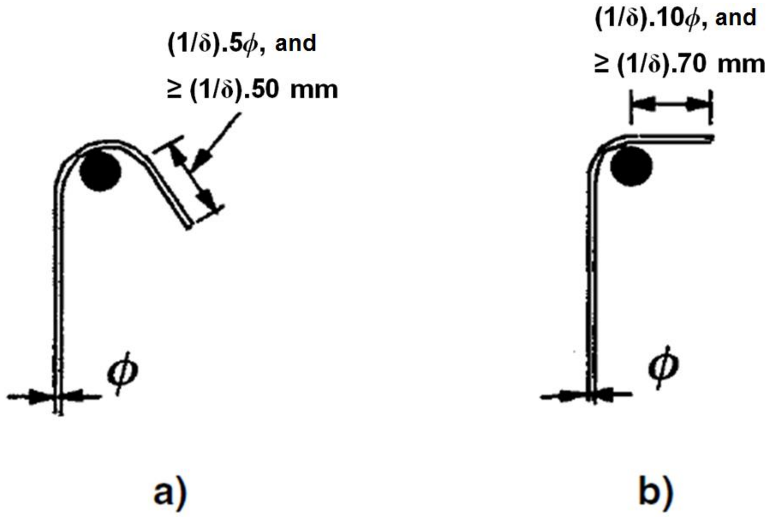


Figure 8.202 — Anchorage of links

- in diagram a), the minimum lengths 5ϕ and 50 mm are multiplied by 1/ δ, where δ is defined in 7.3.4 (7) by Expression (7.213).

- in diagram b), the minimum lengths 10ϕ and 70 mm are multiplied by 1/ δ, where δ is defined in 7.3.4 (7) by Expression (7.213).

NOTE Diagrams c) and d) of Figure 8.5 in standard NF EN 1992-1-1 do not apply.

8.6 Anchorage by welded bars

(1) Does not apply

(2) Does not apply

(3) Does not apply

(4) Does not apply

(5) Does not apply

8.7 Laps and mechanical couplers

8.7.1 General

(1)P Unchanged

8.7.2 Laps

(1)P Unchanged

(2) Unchanged

(3) Unchanged

8.7.3 Lap length

(1) Regarding section 8.7.3 of standard NF EN 1992-1-1, only the expression for l_{0min} is changed, it becomes:

$$l_{0,min} \geq \max\{0.3\ \alpha_6\ l_{b,rqd}\ ;\ 15\ \phi/\ \delta\ ;\ (1/\ \delta).200\ \text{mm}\} \qquad (8.208)$$

Where δ is defined in 7.3.4 (7) by Expression (7.213).

NOTE The lap length may be increased in order to respect the conditions for transverse reinforcement (or fibres) in a lap zone (see 8.7.4).

8.7.4 Transverse reinforcement in the lap zone

8.7.4.1 Transverse reinforcement for bars in tension

(1) Unchanged

NOTE As the fibres can contribute to resist to transverse tensile forces, the placement of transverse reinforcements may be dispensed in lap zones if it is specifically justified.

(2) Unchanged

(3) Unchanged

(4) Unchanged

NOTE Another way of arranging the transverse reinforcement consists of distributing them evenly over the lap length.

8.7.4.2 Transverse reinforcement for bars permanently in compression

(1) Unchanged

NOTE Another way of arranging the transverse reinforcement consists of distributing them evenly over the lap length.

8.7.5 Laps for welded mesh fabrics made of ribbed wires

8.7.5.1 Laps of the main reinforcement

(1) Unchanged

(2) Unchanged

(3) When lapping panels are the same plane, the lapping arrangements for the main longitudinal bars should be in accordance with 8.7.2 and any favourable effects of the transverse bars should be ignored. A value of α_1 = 0.9 shall be adopted to take into account the favourable effect of the fibres.

(4) Unchanged

(5) When condition (4) above is not fulfilled, the effective depth of the steel for the calculation of bending resistance in accordance with 6.1 should apply to the layer furthest from the tension face.

(6) Unchanged

(7) Unchanged

8.7.5.2 Laps of secondary or distribution reinforcement

(1) Part 8.7.5.2 of standard NF EN 1992-1-1 applies with the following addition: the lap lengths l_0 are multiplied by δ defined in 7.3.4 (7) by Expression (7.213).

8.8 Additional rules for large diameter bars

(1) Part 8.8 of standard NF EN 1992-1-1 does not apply to UHPFRCs. Regarding bar diameters, see 8.1 (5) of this standard.

8.9 Bundled bars

8.9.1 General

(1) Unchanged, including (101) of standard NF EN 1992-2 and its National Annex

(2) Unchanged

(3) Unchanged

(4) Unchanged

(5) Bar bundles with an equivalent diameter greater than or equal to 32 mm are not recommended.

8.9.2 Anchorage of bundles of bars

(1) Bundles of bars in tension may be curtailed over end and intermediate supports. Bundles with an equivalent diameter < 32 mm may be curtailed near a support without the need for staggering bars. Bundles with an equivalent diameter greater than or equal to 32 mm are not recommended.

(2) Unchanged

(3) For compression anchorages bundled bars need not be staggered.

8.9.3 Lapping bundles of bars

(1) Unchanged

(2) Unchanged

(3) Does not apply

8.10 Prestressing tendons

8.10.1 Arrangement of prestressing tendons and ducts

8.10.1.1 General

(1)P Unchanged

8.10.1.2 Pre-tensioned tendons

(1) The requirements given in Figure 8.14 of standard NF EN 1992-1-1 in terms of spacing between reinforcement applies to UHPFRCs. Nevertheless, the following condition is added: the spacing between reinforcement must be greater than 1.5 L_f for the UHPFRC to flow satisfactorily.

(2) Unchanged

8.10.1.3 Prestressing ducts

(1)P Unchanged

(2) Unchanged

(3) The requirements given in Figure 8.15 of standard NF EN 1992-1-1 in terms of spacing between reinforcement applies to UHPFRCs. Nevertheless, the following condition is added: the spacing between reinforcement must be greater than 1.5 L_f for the UHPFRC to flow satisfactorily.

8.10.2 Anchorage of pre-tensioned tendons

8.10.2.1 General

(1) Unchanged

8.10.2.2 Transfer of prestress

(1) At release of tendons, the prestress may be assumed to be transferred to the concrete by a constant bond stress f_{bd}, where:

$$f_{bd} = \eta\ \delta\ f_{ctk,el}(t)/\gamma_C \tag{8.209}$$

where:

$f_{ctk,el}$: characteristic tensile limit of elasticity
t: time of tendon
η is defined by Expression (6.64) of this standard
δ is defined in 7.3.4 (7) by Expression (7.213)

(2) Unchanged

(3) The design value of the transmission length should be taken as the less favourable of two values, depending on the design situation:

$$l_{pt1} = 0.4\ l_{pt} \tag{8.210}$$

or

$$l_{pt2} = 1.2\ l_{pt} \tag{8.211}$$

NOTE Normally the lower value is used for verifications of local stresses at release, the higher value for ultimate limit states (shear, anchorage etc.).

(4) Unchanged

(5) Unchanged

8.10.2.3 Anchorage of tendons for the ultimate limit state

(1) The anchorage of tendons should be checked in sections where the concrete tensile stress exceeds $f_{ctk,el}$. The tendon force should be calculated using the law adopted in 3.1.7 for UHPFRCs including the effect of shear. Where the UHPFRC tensile stress is less than $f_{ctk,el}$, no anchorage check is necessary.

(2) The bond strength for anchorage in the ultimate limit state is:

$$f_{bpd} = 0.5.\ \eta\ \delta\ f_{ctk,el}/\gamma_c \tag{8.212}$$

where

η is defined by Expression (6.64) of this standard
δ is defined in 7.3.4 (7) by Expression (7.213)

(3) Does not apply

(4) Unchanged

(5) Unchanged

(6) Unchanged

8.10.3 Anchorage zones of post-tensioned members

(1) Unchanged

(2) Unchanged

(3) Unchanged

(4) Tensile forces due to concentrated forces should be assessed by a strut and tie model, or other appropriate representation. Reinforcement should be detailed assuming that it acts at its design strength. If the stress in this reinforcement is limited to 250 MPa no check of crack widths is necessary.

(5) Unchanged

(106) of standard NF EN 1992-2: Unchanged

8.10.4 Anchorages and couplers for prestressing tendons

(1)P Unchanged

(2)P Unchanged

(3) Unchanged

(4) Unchanged

(5) Unchanged including (105) of standard NF EN 1992-2

(106) of standard NF EN 1992-2: Unchanged

(107) of standard NF EN 1992-2: Unchanged, including what appears in the National Annex to standard NF EN 1992-2

(108) of standard NF EN 1992-2: Unchanged

8.10.5 Deviators

(1)P Unchanged

(2)P Unchanged

(3)P Unchanged

(4) Unchanged

9 DETAILING OF MEMBERS AND PARTICULAR RULES

9.1 General

(1)P Unchanged

(2) Unchanged

(3) Non-brittleness condition must be verified in order to prevent a brittle failure, wide cracks and also to resist forces arising from restrained actions. The non-brittleness condition is assumed to be fulfilled if two conditions are complied with, one condition on the material (see (4)) and one on the verification of sections (see (5)).

(103) of standard NF EN 1992-2: see (3) above

(4) Addition: the behaviour of the UHPFRC material is sufficiently ductile under tension if the following inequality is satisfied:

$$\frac{1}{w_{0.3}}\int_0^{w_{0.3}}\frac{\sigma(w)}{1.25}dw \geq \max(0.4\ f_{ctm,el}\ ;3MPa) \qquad (9.201)$$

where $w_{0.3}$ = 0.3 mm

$f_{ctm,el}$ is the mean value of the tensile limit of elasticity

$\sigma(w)$ is the characteristic post-cracking stress according to the crack width w.

NOTE This condition is satisfied for UHPFRCs covered by the present standard (see 1.1 (4)).

(5) Addition: sections of the structure which are not fully compressed following a linear elastic calculation using Young's modulus E_{cm} and the non-cracked inertia under the SLS combinations characterised by the SLS torsors (N_i ; M_i), must satisfy the following condition:

- if $M_i \neq 0$: the moment resistance $M_{Rd,i}$ calculated in a cracked section using the ULS behaviour law for the material defined in 3.1.7.3 under concomitant N_i must be greater than or equal to Min ($M_{lin,i}$; 1.2 M_i) where the moment $M_{lin,i}$ corresponds to the moment to reach the characteristic tensile limit of elasticity $f_{ctk,el}$ with an elastic behaviour under N_i.

- if M_i = 0, the resisting normal force $N_{Rd,i}$ calculated in the cracked section using the ULS behaviour law for the material defined in 3.1.7.3 must be greater than or equal to Min ($N_{lin,i}$; 1.2 N_i) where $N_{lin,i}$ corresponds to the normal force to reach the characteristic tensile limit of elasticity $f_{ctk,el}$ with an elastic behaviour.

(6) Addition: when the non-brittleness condition is fulfilled and the sections are verified at ULS and SLS including the only contribution of the fibres, it is possible to dispense with the placement of reinforcements. When the word "unchanged" is shown in this section regarding a detailing arrangement relating to a reinforcement, it should be considered that this detailing arrangement must be respect in case in which reinforcements are placed.

9.2 Beams

9.2.1 Longitudinal reinforcement

9.2.1.1 Minimum and maximum reinforcement areas

(1) Does not apply

(2) Does not apply

(3) Unchanged

(4) Does not apply

9.2.1.2 Other detailing arrangements

(1) In monolithic construction, even when simple supports have been assumed in design, the section at supports should be designed for a bending moment arising from partial fixity of at least 0.15 of the maximum bending moment in the span, including when simple supports have been adopted in the design.

(2) When reinforcement is laid at intermediate supports of continuous beams, the total area of tension reinforcement A_s of a flange cross-section should be spread over the effective width of the upper flange (see 5.3.2). Part of it may be concentrated over the web width (See Figure 9.1).

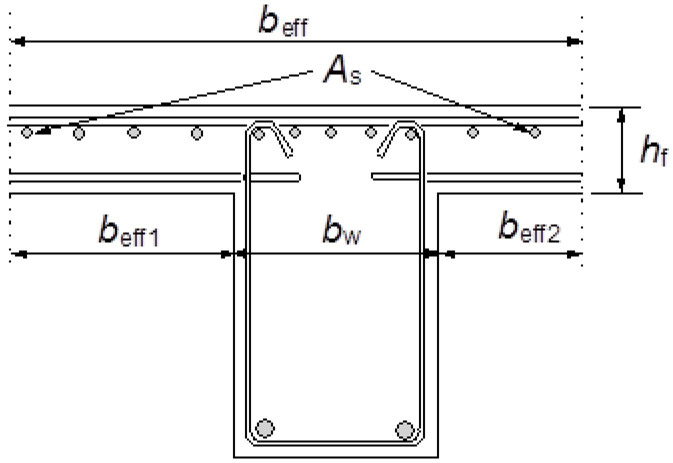


Figure 9.1 — Placing of tension reinforcement in flanged cross-section

(3) Unchanged

9.2.1.3 Staggering of moments curve and refining of curtailment of tension reinforcement

(1) In all sections, the design of the tension area should be justified to take account of the interaction between M_{Ed} and V_{Ed} due to the inclined cracks in webs and flanges.

(2) The additional tensile force ΔF_{td} must be calculated in accordance with 6.2.1.6.

It may also be estimated by shifting the envelope curve of the moments a distance a_l, as shown in Figure 9.2.

For UHPFRC members with or without flexural reinforcement and without shear reinforcement:

$$a_l = (z/2)\,(\cot\theta - \tan\theta) \tag{9.202}$$

For UHPFRC members with flexural and shear reinforcement:

$$a_l = \frac{z}{2}\,\frac{V_{Rd,s}(\cot\theta - \tan\alpha) + V_{Rd,f}(\cot\theta - \tan\theta)}{V_{Rd,s} + V_{Rd,f}} \tag{9.203}$$

(3) Unchanged

(4) Unchanged

9.2.1.4 Anchorage of bottom reinforcement at end supports

(1) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(2) Unchanged

(3) Unchanged

(4) Addition: In the absence of reinforcement, detailing arrangements and special execution arrangements must be shown to provide a good transmission of forces between the horizontal member and the vertical member taking account of the effect of any clamping.

NOTE If reinforcement are present, it may be worthwhile reading the additional information given in leaflet FD P18-717 with regard to this section.

9.2.1.5 Anchorage of bottom reinforcement at intermediate supports

(1) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(2) Unchanged

(3) Unchanged

(4) Addition: In the absence of reinforcement, detailing arrangements and special execution arrangements must be justified to provide a good transmission of forces between the horizontal member and the vertical member taking account of the effect of any clamping.

NOTE If reinforcement are present, it may be useful reading the additional information given in booklet FD P18-717 with regard to this section.

9.2.2 Shear reinforcement

(1) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1 and (101) of standard NF EN 1992-2 and its National Annex

(2) Unchanged

(3) Unchanged

(4) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(5) Does not apply

(6) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(7) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(8) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

9.2.3 Torsion reinforcement

(1) Unchanged

(2) Does not apply

(3) Unchanged

(4) Unchanged

9.2.4 Surface reinforcement

(1) Does not apply

9.2.5 Indirect supports

(1) Unchanged

(2) Unchanged

9.3 Solid slabs

(1) Unchanged

9.3.1 Flexural reinforcement

9.3.1.1 General

(1) Does not apply

(2) Does not apply

(3) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(4) Unchanged

9.3.1.2 Reinforcement in slabs near supports

(1) Unchanged

(2) Unchanged

9.3.1.3 Corner reinforcement

(1) Unchanged

9.3.1.4 Reinforcement at the free edges

(1) Unchanged

(2) Unchanged

9.3.2 Shear reinforcement

(1) Does not apply

(2) Does not apply

(3) Does not apply

(4) Unchanged

(5) Unchanged

9.4 Flat slabs

9.4.1 Slab at internal columns

(1) Unchanged

(2) Does not apply

(3) Does not apply

9.4.2 Slab at edge and corner columns

(1) Unchanged

9.4.3 Punching shear reinforcement

(1) Unchanged

(2) Does not apply

(3) Unchanged

(4) Unchanged

9.5 Columns

9.5.1 General

(1) Unchanged

9.5.2 Longitudinal reinforcement

(1) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(2) Does not apply

(3) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(4) Unchanged

9.5.3 Transverse reinforcement

(1) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(2) Unchanged

(3) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(4) Unchanged

(5) Unchanged

(6) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

9.6 Walls

9.6.1 General

(1) Unchanged

9.6.2 Vertical reinforcement

(1) Does not apply

(2) Does not apply

(3) Unchanged

9.6.3 Horizontal reinforcement

(1) Does not apply

(2) Unchanged

9.6.4 Transverse reinforcement

(1) Does not apply

(2) Unchanged

9.7 Deep beams

(1) Does not apply

(2) Unchanged, including (102) of standard NF EN 1992-2 and its National Annex

(3) Unchanged

9.8 Foundations

9.8.1 Pile caps

(1) Unchanged

(2) Unchanged

(3) If reinforcement are placed in the pile caps, the main tensile reinforcement resisting the effects of the actions should be concentrated in the tension areas located between the piles. A minimum bar diameter of Φ_{min} should be provided for any reinforcement placed.

Φ_{min} = 8 mm for buildings
Φ_{min} = 12 mm for bridges

(4) Unchanged

(5) Unchanged

9.8.2 Column and wall footings

9.8.2.1 General

(1) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(2) Unchanged

(3) Unchanged

9.8.2.2 Anchorage of bars

(1) Unchanged

(2) Unchanged

(3) Unchanged

(4) Unchanged

(5) Unchanged

9.8.3 Tie beams

(1) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(2) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

9.8.4 Column footing on rock

(1) Does not apply

(2) Does not apply

9.8.5 Bored piles

(1) Does not apply

(2) Does not apply

(3) Does not apply

(4) Does not apply

9.9 Regions with discontinuity in geometry or action

(1) Unchanged

(2)P Unchanged

9.10 Tying systems

9.10.1 General

(1)P Unchanged

(2) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(3) Unchanged

(4) In the design of the ties the reinforcement may be assumed to be acting at its characteristic strength. For class T2* and T3* UHPFRCs, it may be assumed that the UHPFRC acts at $\sigma_{Rd,f}$ defined in 6.2.1.4 (1) with a partial factor γ_{cf} = 1.05 and orientation factor K_{local}. For $\sigma_{Rd,f}$ used in tying system designs, the value $\varepsilon^* = \varepsilon_{u,lim}$ must be adopted for class T3* UHPFRCs and w^* = 0.3 mm for class T2* UHPFRCs. For tying systems with class T1* UHPFRCs, only reinforcement is taken into account.

Note In cases where UHPFRC is used over extensive areas where a localised lack would not have an effect, it is possible to calculate $\sigma_{Rd,f}$ using K_{global}. The contribution of the UHPFRC to the tying systems is given by $A_t \cdot \sigma_{Rd,f}$, A_t being the transverse area of the tie in continuous UHPFRC. This area is chosen in accordance with the information set out in 9.10.2.

The reinforcement or the UHPFRC are capable of carrying tensile forces defined in the following clauses.

(5) Unchanged

9.10.2 Proportioning of ties

9.10.2.1 General

(1) Ties are assumed to be made up of continuous UHPFRC members or minimal reinforcement, and not by additional UHPFRC members or reinforcement added to those required by the structural analysis.

9.10.2.2 Peripheral ties

(1) Unchanged

(2) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(3) Unchanged

9.10.2.3 Internal ties

(1) Unchanged

(2) Unchanged

(3) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(4) Unchanged

(5) Unchanged

9.10.2.4 Horizontal ties to columns and/or walls

(1) Unchanged

(2) Unchanged, including what appears in the National Annex to standard NF EN 1992-1-1

(3) Unchanged

9.10.2.5 Vertical ties

(1) Unchanged

(2) Unchanged

(3) Unchanged

9.10.2.6 Continuity and anchorage of ties

(1)P Ties in two horizontal directions shall be effectively continuous and anchored at the perimeter of the structure. For UHPFRC ties, provisions must be made at construction joints and joints between the various members in order to ensure continuity of the forces calculated in the previous sections.

(2) Unchanged

(3) Unchanged

10 ADDITIONAL RULES FOR PRECAST CONCRETE ELEMENTS AND STRUCTURES

This section does not apply.

11 LIGHTWEIGHT AGGREGATE CONCRETE STRUCTURES

This section does not apply.

12 PLAIN AND LIGHTLY REINFORCED CONCRETE STRUCTURES

This section does not apply.

Annex A
(informative)
Modification of partial factors for materials

This annex does not apply.

Annex B
(informative)
Creep and shrinkage strain

This annex does not apply.

Annex C
(normative)
Properties of reinforcement suitable for use with this Eurocode

This annex applies.

Annex D
(informative)
Detailed calculation method for prestressing steel relaxation losses

This annex applies.

Annex E
(informative)
Indicative strength classes for durability

This annex does not apply.

Annex F
(informative)
Tension reinforcement expressions for in-plane stress conditions

This annex does not apply.

Annex G
(informative)
Soil structure interaction

This annex applies.

Annex H
(informative)
Global second order effects in structures

This annex applies.

Annex I
(informative)
Analysis of flat slabs and shear walls

This annex does not apply.

Annex J
(informative)
Detailing rules for particular situations

Annex J of standard NF EN 1992-1-1 does not apply.

Only part J.104 of standard NF EN 1992-2 applies and is adapted to the case of UHPFRCs as follows:

J.104 Partially loaded areas

J.104.1 Bearing zones of civil engineering structures

(101) The design of bearing zones of bridges should be in accordance with the rules given in this clause in addition to those in 6.5 and 6.7.

(102) The distance from the edge of the loaded area to the free edge of the concrete section should not be less than 1/6 of the corresponding dimension of the loaded area measured in the same direction. In no case should the distance to the free edge be taken as less than 50 mm.

(103) The maximum force F_{Rdu} is given in 6.7 (2).

(104) In order to avoid edge sliding, the strength of the tensioned UHPFRC should be verified, and in addition if necessary, uniformly distributed reinforcement parallel to the loaded face should be provided to the point at which local compressive stresses are dispersed. This point is determined as follows: a line inclined at an angle θ (30°) to the direction of load application is drawn from the edge of the section to intersect with the opposite edge of the loaded surface, as shown in Figure J.101. Any reinforcement provided to avoid edge sliding shall be adequately anchored.

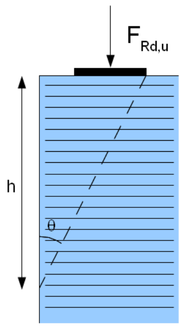


Figure J.101 — Edge sliding mechanism

(105) The tensile strength of the UHPFRC in tension and any reinforcements designed to avoid edge sliding (A_r) should be calculated in accordance with the expression:

$$A_r\, f_{yd} + h.b.\sigma_{Rd,f} \geq F_{Rdu}/2 \qquad \text{(J.101)}$$

b is the width of the specimen, h is the height defined in Figure J.101.

$\sigma_{Rd,f}$ is the mean value of the post-cracking strength defined in 6.2.1.4 (1) calculated using K_{local}. In addition, for UHPFRCs of class T1* and T2*, $\sigma_{Rd,f}$ is calculated using w* = 0.3 mm.

J.104.2 Anchorage zones of post-tensioned members

(101) The following rules apply in addition to those in 8.10.3 for the design of anchorage zones where two or more tendons are anchored. The reinforcement is usually calculated under ULS load conditions, with a prestressing force taken equal to $\gamma_{p,unfav}$ x P_{max} ($\gamma_{p,unfav}$ = 1.2 in general) and the stress is verified under characteristic SLS conditions with the prestressing force taken as equal to P_{max}.

(102) The minimum distance between the centreline of the anchorage and the edge of the UHPFRC should not be less than the distances determined through the load transfer test described in appendix S. The compressive strength f_c of the UHPFRC determined by the compression tests or by maturometry should satisfy:

$$f_c \geq f_{cm,0} + 3 \text{ MPa} \quad (J.102)$$

$$f_c \geq f_{ck} + 6 \text{ MPa} \quad (J.103)$$

$f_{cm,0}$ is the minimum UHPFRC strength required for full prestressing as determined in accordance with Annex S.

f_{ck} is the characteristic compressive strength required for the design.

The tensile strength of the UHPFRC should satisfy:

$$f_{ct,el} \geq f_{ctm,el,0} + 0.5 \text{ MPa} \quad (J.104)$$

This tensile strength should be determinated by direct tensile tests or from 4-point bending tests, or calculated from the compressive strength using a method validated by experimental results.

Any reinforcement required to prevent bursting and spalling in anchorage zones is determined in relation to a rectangular prism of concrete, known as the primary regularisation prism, located behind each anchorage. The cross section of the prism associated with each anchorage is known as the associate rectangle. The associate rectangle has the same centre and the same axes of symmetry as the anchorage plate (which should have two axes of symmetry) and should satisfy:

$$\frac{P_{max}}{c.c'} \leq 0,6.f_{ck}(t) \quad (J.105)$$

where:

P_{max} is the maximum force applied to the prestressing tendon according to 5.10.2.1.

c, c' are the dimensions of the associated rectangle

NOTE Dimensions c and c' must remain at least equal to the dimensions of the test block defined in accordance with the provisions of Annex S.

$f_{ck}(t)$ is the compressive strength of the UHPFRC at the time tensioning

The associate rectangle should have approximately the same aspect ratio as the anchor plate. For a member in UHPFRC, this requirement is satisfied if:

$$0,8\frac{a}{a'} \leq \frac{c}{c'} \leq 1,25\frac{a}{a'}$$

In parallel, the ETA for the anchorage or the tests carried out under annex S, may possibly stipulate more restrictive conditions of homothety between the associated rectangle and the anchor plate than the above condition.

Rectangles associated with anchorages located in the same cross section should remain inside the UHPFRC and should not overlap.

The “primary regularisation prism” represents very approximately the volume of UHPFRC in which the stresses change from very high values just behind the anchorage plate to a reasonable value for UHPFRC under uniaxial compression. The axis of the prism is taken as the axis of the tendon, its base is the associate rectangle and its depth behind the anchorage is taken as δ = 1.2 max(c,c').

The prisms associated with different anchorages may overlap (this can occur when the tendons are not parallel) but should remain inside the UHPFRC.

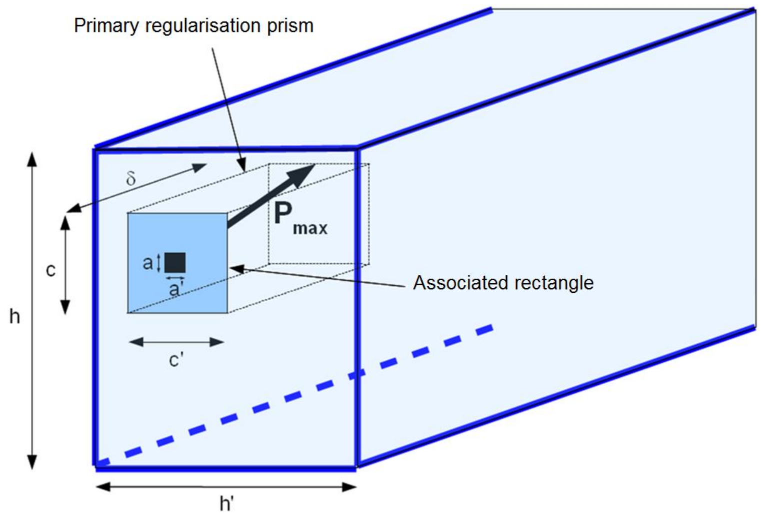


Figure J.102 — Primary regularisation prism

(103) A check should be made that the cross-section of the reinforcement and the contribution of the fibres are sufficient to prevent any bursting or spalling in each regulation prism.

$$A_s.f_{yd} + S_{fe}.\sigma_{Rd,f} \geq P_{max}\gamma_{p,unfav}\left[0.15\xi + \sin\alpha.(\xi - 1)\right] \tag{J.106}$$

where

A_s, is the cross-sectional area of the reinforcement crossing the regulation prism in a given direction (verification to be carried out in the two perpendicular directions)

S_{fe}, is the action area of the fibres, equal to c x c' for post-tensioning and to e'.l_{pt1}/4 for pre-tensioning.

P_{max} is the maximum force applied to the tendon

f_{yd} is the design yield strength of the reinforcement.

$\sigma_{Rd,f}$ is the mean value of the post-cracking strength defined in 6.2.1.4 (1) calculated using K_{local}. In addition, for UHPFRCs of class T1* and T2*, $\sigma_{Rd,f}$ is calculated using w* = 0.3 mm.

α is the inclination of the tendon at the anchorage, counted as positive if the tangential component of the cable is orientated towards the nearest free edge and negatively if it is orientated towards the inside of the block.

ξ is a factor taking into account the distance d (see Figure J.103 below) of the anchor to the concrete face and its eccentricity relative to the local height h of the part in UHPFRC adjacent to the anchor.

$$\xi = \sqrt{\frac{2}{d \times \left(\frac{3}{h} + \frac{1}{c}\right)}} \geq 1 \tag{J.107}$$

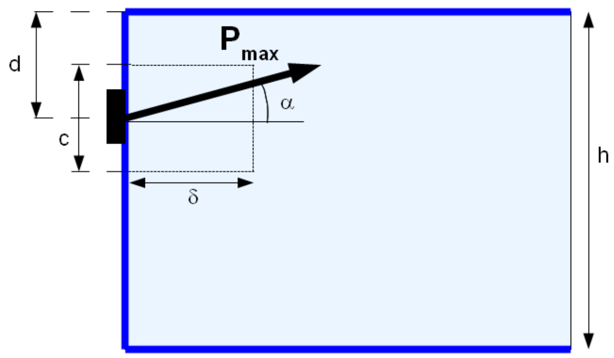


Figure J.103 — Primary regularisation prism – longitudinal view

Any reinforcement should be distributed in each direction over the whole length of the prism.

A check should also be made that the reinforcement cross sectional area and the contribution of the fibres near to the surface around the loaded face are sufficient by satisfying the following inequality:

$$A_s.f_{yd} + S_{fs}.\sigma_{Rd,f} \geq 0.03\ P_{max}\gamma_{p,unfav} \tag{J.108}$$

A_s, is the reinforcement cross sectional area close to the loaded face in a given direction (verification to be carried out in the two perpendicular directions)

S_{fs}, action area of the fibres, equal to 0.2 x c x c'

Cross sectional area A_s obtained using Expression (J.108) is to be provided in the immediate vicinity of the anchorage in addition to the cross sectional area A_s obtained using Expression (J.106). Likewise, the strength given by the fibres expressed by the term $\sigma_{Rd,f}$ cannot be used both in Expression (J.108) and in Expression (J.106).

(104) If applicable, the minimum reinforcement derived from the tests described in Annex S should be provided.

(105) The general distribution is illustrated by Figure J.104 below:

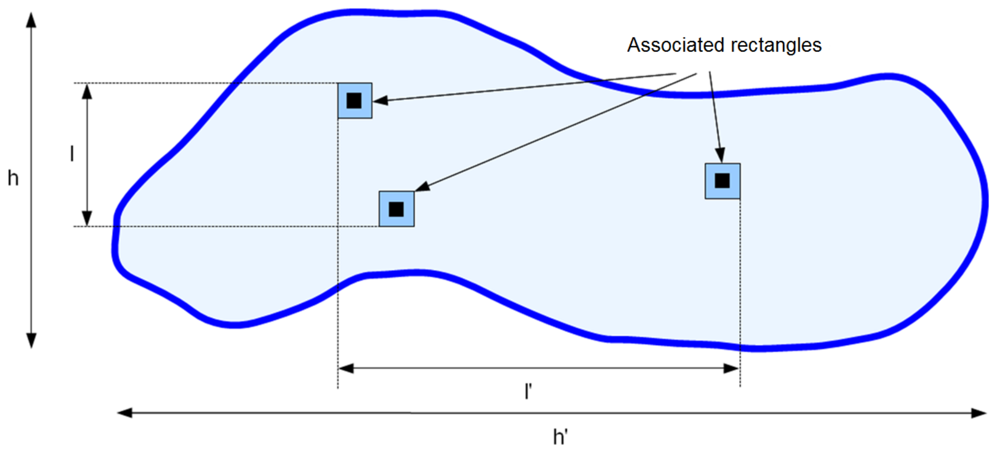


Figure J.104 — General distribution

The regularization of forces is considered over a length L_R in the vertical plane and L_R' in the horizontal plane (it is however possible to take two other orthogonal directions).

L_R and L_R' are calculated:

$$L_R = \max(h - l; h/2) \tag{J.109}$$

$$L_R' = \max(h' - l'; h'/2) \tag{J.110}$$

In each design direction (for example, the vertical direction), different sections should be considered and the forces applied on these sections calculated, as shown on Figure J.105 below.

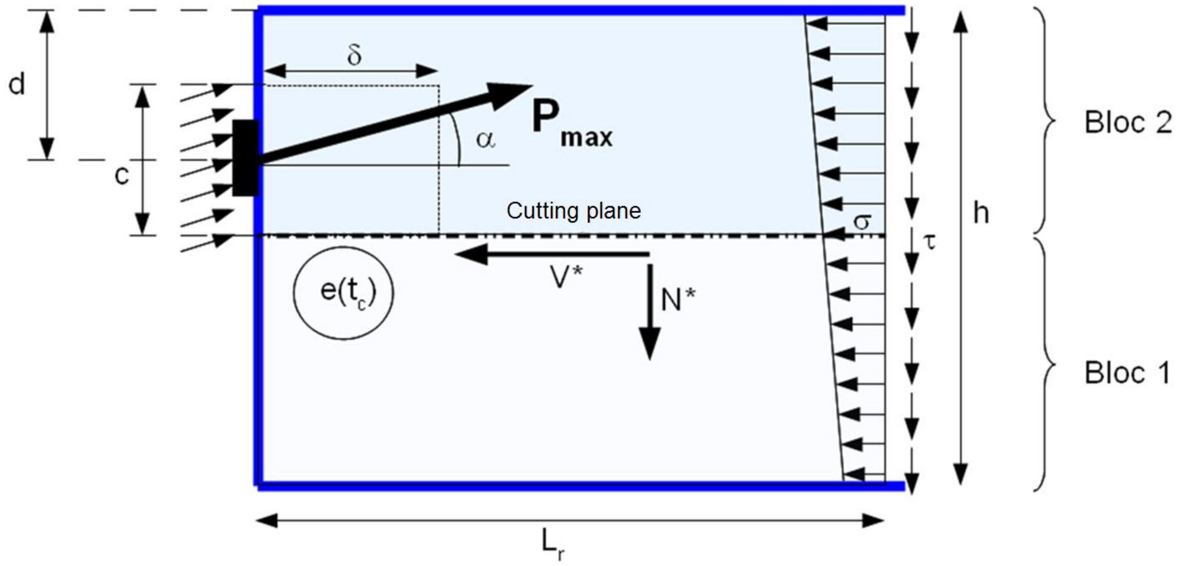


Figure J.105 — Calculation of forces at the cutting planes

N* and V* are the forces applied by block 1 (part located below the cut) on block 2 (part located above the cut).

The average stresses σ* and τ* are obtained by dividing the forces N* and V* by the surface area of the section, i.e.:

$$\tau^* = \frac{V^*}{L_R.e(t_c)} \tag{J.111}$$

$$\sigma^* = \frac{N^*}{L_R.e(t_c)} \tag{J.112}$$

The mean stresses σ* and τ*, calculated at the characteristic SLS with $F = P_{max}$, must comply with the following condition:

$$|\tau|^* - \sigma^* \leq f_{ctk,el} \tag{J.113}$$

Verification of the reinforcement and the efficiency of the fibres is carried out at ULS by taking $F = \gamma_{p,unfav} P_{max}$.

The general equilibrium reinforcement A_s and the fibres must satisfy:

$$A_s.f_{yd} + e(t_c).L_R.\frac{2}{3}\sigma_{Rd,f} \geq (|V^*| - N^*)_{red} = (|V^*| - N^*)\ \min\left(1\ ;\ 0.2 + 0.8\sqrt{\frac{|\tau^*| - \sigma^*}{f_{ctk,el}}}\right) \tag{J.114}$$

$\sigma_{Rd,f}$ is the mean value of the post-cracking strength defined in 6.2.1.4 (1) calculated using K_{global}. In addition, for UHPFRCs of class T1* and T2*, $\sigma_{Rd,f}$ is calculated using w* = 0.3 mm.

Annex KK
(informative)
Structural effects induced by the time-dependent behaviour of the concrete

This annex applies.

Annex LL
(informative)
Plate members in concrete

This annex applies.

Annex MM
(informative)
Shear and transverse bending

This annex applies.

Annex NN
(informative)
Damage equivalent stress range for fatigue verifications

This annex does not apply.

Annex OO
(informative)
Standard discontinuity regions for bridges

This annex applies.

Annex PP
(informative)
Safety format for non-linear analysis

This annex does not apply.

Annex QQ
(normative)
Control of web cracking by shear

(101) Control of web cracking by shear is achieved by limiting stresses according to the following formulae:

$$\tau^2 - \sigma_x\sigma_t \leq 0.35 f_{ctk,el}\left[f_{ctk,el} + \frac{2}{3}(\sigma_x + \sigma_t)\right] \tag{QQ.1}$$

and

$$\tau^2 - \sigma_x\sigma_t \leq 2\frac{f_{ctk,el}}{f_{ck}}\left[0.6 f_{ck} - \sigma_x - \sigma_t\right]\left[f_{ctk,el} + \frac{2}{3}(\sigma_x + \sigma_t)\right] \tag{QQ.2}$$

τ shear stress

σ_x and σ_t designate the longitudinal and vertical stresses respectively.

These stresses result from an elastic calculation based on SLS forces.

<u>Sum of shear and torsion forces</u>:

In a solid section or a hollow section with thick wall, it is possible to adopt the square root of the sum of the squares of stresses contributed by torsion and shear as shear stress at SLS: $\tau^2 = \tau_{tors}^2 + \tau_{tranch}^2$

For a hollow section with thin wall, an algebraic sum should be adopted:

$$\tau = \tau_{tors} + \tau_{tranch} \tag{QQ.3}$$

Annex R
(normative)
Structural fire design of UHPFRCs

R.1 General

When the action of fire is to be taken into account in the design in accordance with a given scenario, it is determined by reference to 2.2 of standard NF EN 1992-1-2 and the resistance, compartmentation and insulation (R, E, I) criteria required for the member in UHPFRC are specified with the associated duration by reference to 2.1 of standard NF EN 1992-1-2.

Fire stability is demonstrated as follows:

- either by full scale testing representing the fire scenario with simultaneous loading;
- or by thermomechanical modelling according to design principles analogous to those in standard NF EN 1992-1-2 excluding the tabulated methods. In this case, the physical and mechanical properties required are determined in accordance with 5.5.6 of standard NF P18-470.

As indicated in standard NF P18-470, the control of spalling under the action of fire, generally based on the incorporation of a sufficient quantity of polypropylene fibres, does not constitute an intrinsic property of the material and must be verified experimentally on a representative member or component (both in geometry and load) of the actual structure with regard to the fire scenario considered. The test is satisfied if the losses of material (reduction in cross-section), if any, remain compatible with the fire stability demonstration.

For preliminary or design studies, and in the absence of tests or an identity card, the following assumptions may be made:

- change in compressive strength, Young's modulus and thermal expansion with temperature identical to those of class 2 concretes according to standard NF EN 1992-1-2.
- change in tensile behaviour: for $f_{ctk,el}$, reduction of 0 % at 80°C, 40 % at 150°C, 55 % at 750°C and 100 % at 1000 °C. Regarding the post-cracking part of the curve, 0 % at 80 °C and 100 % at 450°C.

Annex S
(normative)
Adaptation of the European technical approval procedure to prestressing anchorages used in a UHPFRC

S.1 General

(1) The ability of an anchorage required for use in a UHPFRC structure, and which already has a European technical approval according to ETAG013 [ETAG,02], to transfer its force to the UHPFRC shall be verified. This annex describes suitable transfer tests based on tests carried out in the past.

It applies only to mechanical anchorages and not to bond anchorages.

S.2 Test procedure

S.2.1 Number of tests

(1) Given the scattering of results, each size of anchorage or coupler shall be tested 3 times with the UHPFRC with which it is to be used.

Contrary to ETAG013, interpolating between the sizes tested is not allowed due to possible scale effects.

The three transfer tests therefore validate a given size of anchorage (or coupler) used with a given UHPFRC.

S.2.2 Test description

(1) The test specimen is schematically shown in Figure S.1. The specimen shall contain those anchorage components and any bursting reinforcement which will be embedded in the structural UHPFRC. Their arrangement has to comply with the intended application and with the specification as per the ETA applicant's specifications.

The test specimen shall be a concrete prism tested in axial compression. Its concrete cross-section, equal to $A_c = a \cdot b$ shall correspond to the minimum cross-section in axial compression for the particular tendon and for the specified UHPFRC.

The height h of specimen shall be at least three times the largest of the two transverse dimensions a or b (see Figure S.1). The height of the lower part of the specimen, non-reinforced, shall be at least 0.5h.

The concrete used for the test specimen shall correspond exactly to the UHPFRC to be used with the anchorage. Any heat treatment, curing or maturation shall be representative of the applications envisaged. The test cylinders and prisms cast for the determination of the compressive and tensile strength of the UHPFRC shall be cured in the same manner.

In order to represent as faithfully as possible the placement of the UHPFRC around the anchorage during casting, the specimen shall be cast in a horizontal position.

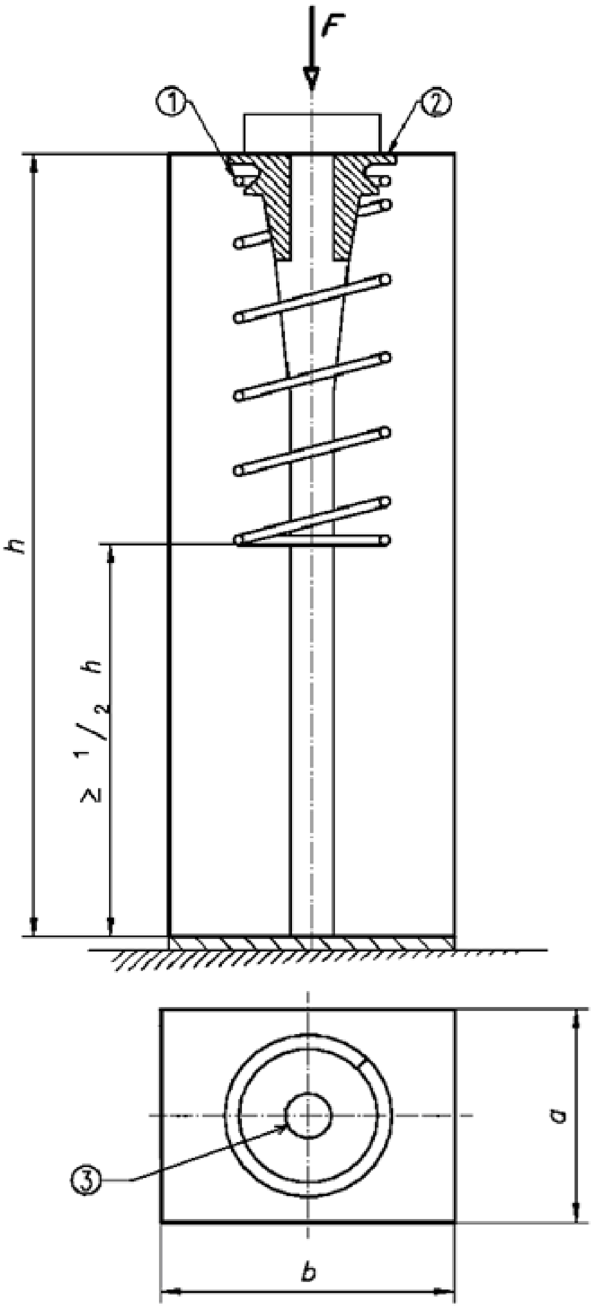


Figure S.1 — Test body for load transfer test

(2) The specimen shall be mounted in a calibrated test rig or testing machine. The load shall be applied to the specimen on an area which simulates the loading condition in a complete anchorage.

The load is increased in steps: 0.2 F_{pk}, 0.4 F_{pk}, 0.6 F_{pk} and 0.8 F_{pk} (see Figure S.2). Once the load of 0.8 F_{pk} has been reached, at least ten slow loading cycles are to be performed, with 0.8 F_{pk} and 0.12 F_{pk} being the upper and the lower load limits, respectively. The necessary number of load cycles depends upon stabilisation of strain readings and crack widths as described below. Following cyclic loading, the specimen shall be loaded continuously to failure.

During cyclic loading, measurements shall be taken at the upper and lower loads of several cycles in order to decide whether satisfactory stabilisation of strains and widths of cracks is being attained. Cyclic loading shall be continued to n cycles until stabilisation is satisfactory, (see part 2.3). Figure S.2 shows the sequence of loading and measurements.

At the final test to failure the mean compressive strength and the mean tensile strength of concrete of specimen shall be:

$f_{cm,e} \leq f_{cm,0} + 15$ MPa

$f_{ctm,el,e} \leq f_{ctm,el,0} + 1$ MPa

$f_{cm,e}$ is determined by conducting a compression test on three specimens

$f_{ctm,el,e}$ is determined by conducting a direct tensile test or 4-point bending test on 3 specimens (see annex D of NF P18-470).

$f_{cm,0}$ is the mean compressive strength of UHPFRC at which full prestressing is permitted.

$f_{ctm,el,0}$ is the mean limit of elasticity of the UHPFRC at which full prestressing is permitted.

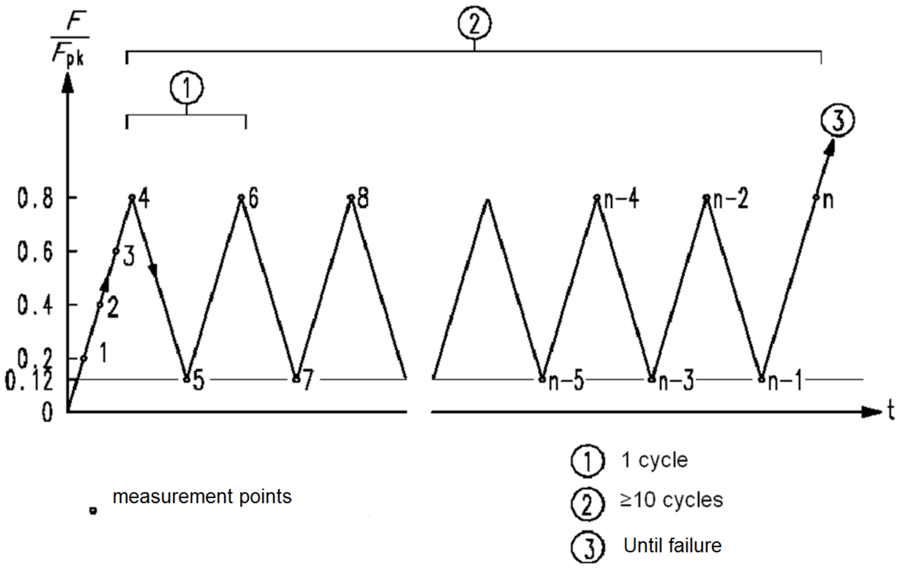


Figure S.2 — Load transfer test procedure

(3) Crack widths can be considered to have stabilised if their width under upper load complies with:

$$w_n - w_{n-4} \leq 1/3\,(w_{n-4} - w_0),\ n \geq 10 \quad \text{(S.1)}$$

or

$$w_n \leq 0.1 \text{ mm} \quad \text{(S.2)}$$

Longitudinal and transverse strains can be considered to have stabilised if the increase of strain under the upper load complies with:

$$\varepsilon_n - \varepsilon_{n-4} \leq 1/3\,(\varepsilon_{n-4} - \varepsilon_0),\ n \geq 10 \quad \text{(S.3)}$$

Figure S.3 gives more details on how to assess stabilisation criteria.

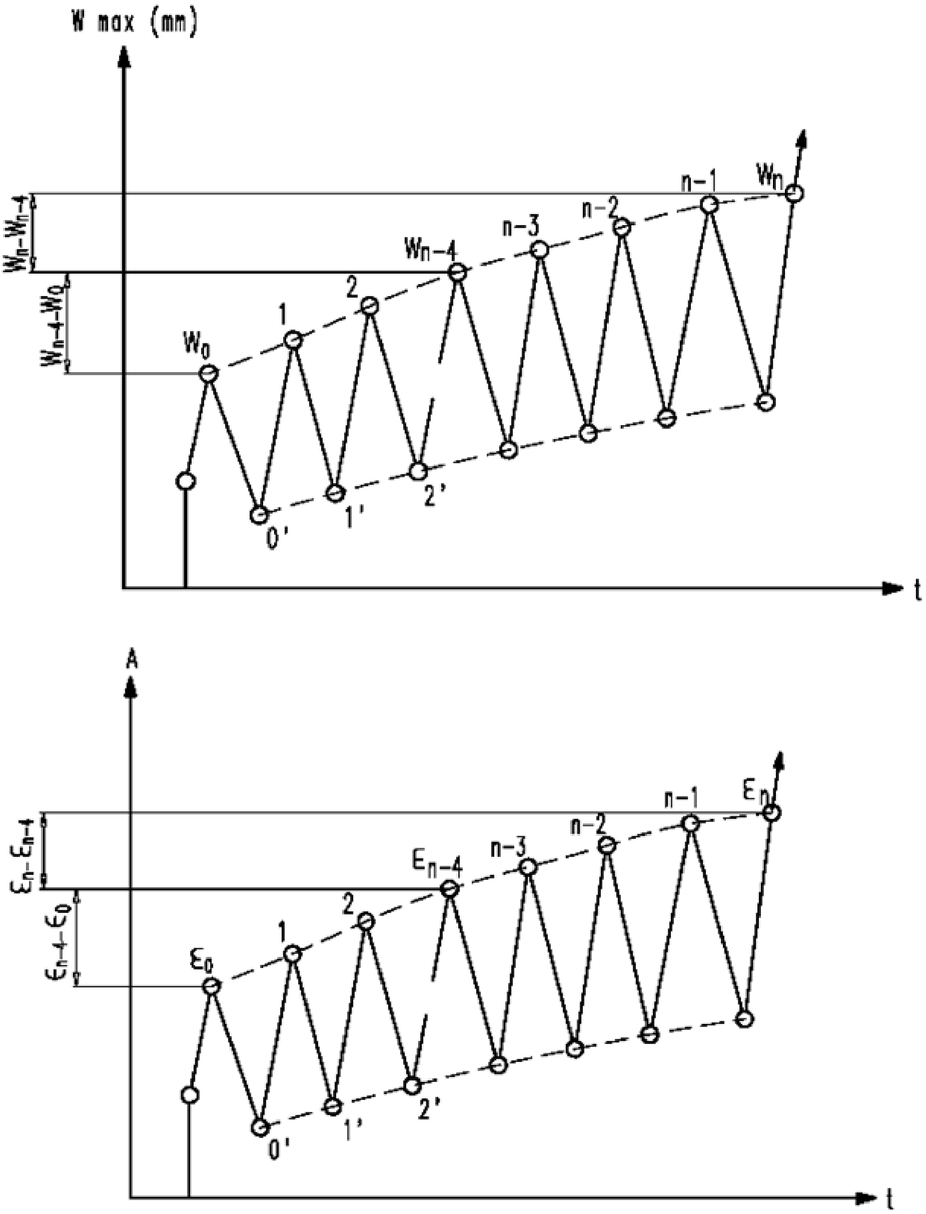
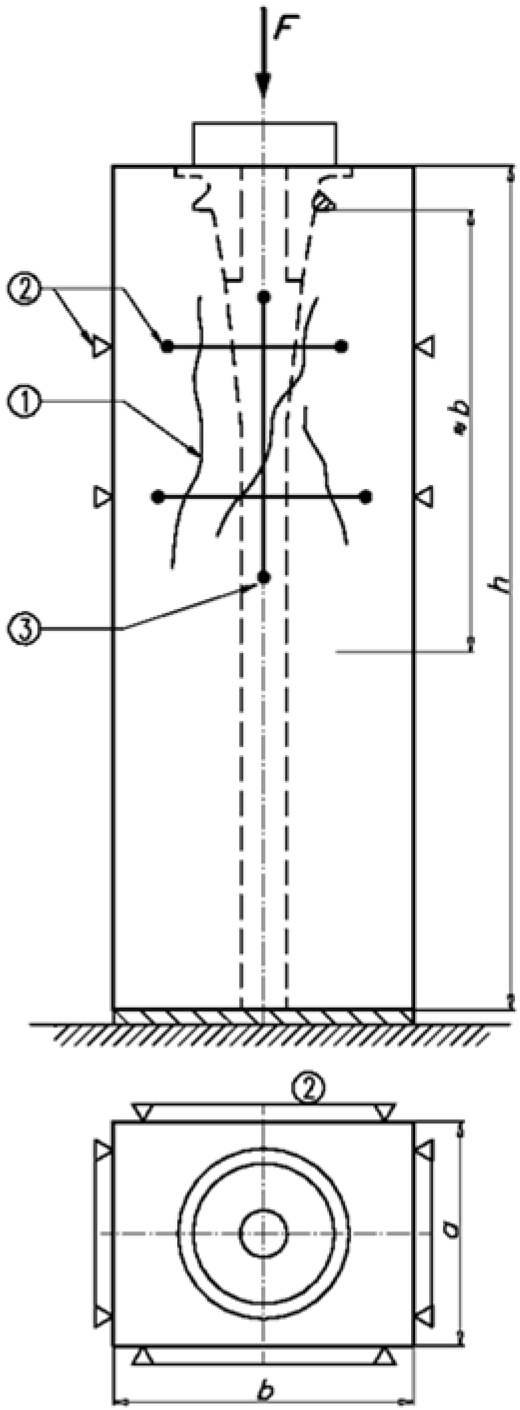


Figure S.3 — Assessment of stabilisation of strain and crack widths

(4) The following measurements and observations shall be made and recorded:

- compliance checking of the components with specifications (materials, machining, geometry, hardness, etc);
- longitudinal and transverse concrete strains on the four faces of the specimen (if this is not possible, on at least three faces i.e. the unformed face, one side face and the lower face during casting), in the region of maximum bursting effect under the upper and lower load, as a function of the number of load cycles;
- formation, width and propagation of cracks on the side faces of the specimen, as mentioned above;
- visual inspection or measurement of deformation of anchorage components in contact with concrete;
- location and mode of failure;
- Ultimate force F_u;

Figure S.4 schematically shows the arrangement of the gauge points for strain measurement on each side of the specimen, etc.



1 Cracks
2 Measurement of ε_t
3 Measurement of ε_v (base lenght $\approx$ 0,6 à 0,8 b)

Figure S.4 — Measuring set-up for load transfer test

(5) The acceptance criteria are as follows:

Crack widths max w:

- upon first attainment of upper load of 80% of tensile element characteristic strength, not more than 0.15 mm
- upon last attainment of lower load of 12% of tensile element characteristic strength, not more than 0.15mm

- upon last attainment of upper load of 80% of tensile element characteristic strength, not more than 0.25mm

Recordings of longitudinal and transverse cracks must stabilise during cyclic loading (criterion to be checked if the cracks observed exceed 0.10 mm).

The recordings of the longitudinal and transverse strains must stabilise during cyclic loading.

The measured ultimate force shall satisfy:

$$F_u \geq 1.3\ F_{pk}\ (f_{cm,e}/f_{cm,0})\ \max(f_{ctm,e}/f_{ctm,0};1) \tag{S.4}$$

S.3 Application to the structure

(1) To apply the result of the tests to the structure, dimensions a and b of the specimen are corrected to take into account a favourable edge effect near the formwork walls (the fibres orientate parallel to the formwork plane). Dimensions a' and b' of the structure are therefore introduced and defined as follows:

$$a' = a + 0.6 * L_f \tag{S.5}$$

$$b' = b + 0.6 * L_f \tag{S.6}$$

where L_f is the length of the longest fibres

From the reference dimensions a, b and a' and b', the minimum centrelines distances x and y of the anchorages in the structure follow the expressions:

$$A_c = x \bullet y = a' \bullet b' \tag{S.7}$$

$$x \geq 0.85\ a' \tag{S.8}$$

$$y \geq 0.85\ b' \tag{S.9}$$

The minimum edge distances x_b and y_b in the structure follow the expressions:

$$x_b.y_b = (a.b)/4 \tag{S.10}$$

$$x_b \geq 0.85\ a/2 \tag{S.11}$$

$$y_b \geq 0.85\ b/2 \tag{S.12}$$

Annex T
(informative)
Indicative values of UHPFRC characteristics

T.1 General

(1) The aim of this annex is to provide indicative values of UHPFRC characteristics which can be used to carry out structure calculations at the preliminary design or project stages other than the execution stages, in the absence of an identity card and/or test results on the materials which will actually be used in the structure.

T.2 Mechanical characteristics

(1) The indicative values of the main mechanical characteristics are given in the table below. Regarding tensile behaviour, as an initial approach, a class T2 UHPFRC may be considered for thick members and class T3 for thin members.

Table T.1 — Indicative values for UHPFRC characteristics

Young's modulus E_{cm}	45 - 65 GPa
Characteristic compressive strength f_{ck}	150 - 200 MPa
Mean compressive strength f_{ck}	160 - 230 MPa
Characteristic tensile limit of elasticity $f_{ctk,el}$	7.0 - 10.0 MPa
Mean tensile limit of elasticity $f_{ctk,el}$	8.0 - 12.0 MPa
Characteristic post-cracking strength f_{ctfk}	6.0 - 10.0 MPa
Mean post-cracking strength f_{ctfk}	7.0 - 12.0 MPa
Global fibre orientation factor K_{global}	1.25
Local fibre orientation factor K_{local}	1.75
Linear coefficient of thermal expansion	11 µm/m/°C
Length L_f	12 - 20 mm

The tensile law which can be adopted for thick members is the conventional law given in 3.1.7.3.2 (1), using w_{pic} = 0.3 mm.

It is possible to adopt $f_{ctf1\%,k} = 0.8\ f_{ctfk}$

The tensile law which can be adopted for thin members is the conventional law n°. 2 given in 3.1.7.3.3 (1), using a value $\varepsilon_{u,lim} = \varepsilon_{lim}$ = 5.0 %.

T.3 Shrinkage and creep

(1) For shrinkage, the following indicative values may be used, in an environment with an average relative humidity of the order of 50 to 70 %:

- for a type STT UHPFRC, an autogenous shrinkage with final amplitude 550 µm/m and dessication shrinkage with final amplitude 150 µm/m
- for a type TT1 UHPFRC, total shrinkage amplitude of 550 µm/m
- for a type TT2 or TT1+2 UHPFRC, total shrinkage amplitude of 550 µm/m, with no shrinkage after heat treatment has ended.

(2) The following indicative values may be used for creep:

- for a type STT UHPFRC, a creep factor of 0.8 or 1.0 if the loads are applied at early age
- for a type TT1 UHPFRC, a creep factor of 0.4 if the loads are applied after thermal curing

For a UHPFRC of type TT2 or TT1+2, a creep coefficient of 0.2 for loads applied after heat treatment.

(3) The time-evolution of shrinkage may then be described using the models in Annex B of standard NF EN 1992-2, through a calibration of the amplitudes and coefficients linked to the kinetics according to B.104 of standard NF EN 1992-2. In the absence of other information, the following indicative values may be used:

Endogenous shrinkage (STT or TT1 UHPFRC)

For $\frac{f_{cm}(t)}{f_{ck}} < 0{,}1$:

$$\varepsilon_{ca}(t) = 0 \tag{T.1}$$

For $\frac{f_{cm}(t)}{f_{ck}} \geq 0{,}1$:

$$\varepsilon_{ca}(t) = \beta_{ca}\left[1 - e^{-\frac{t}{\tau_{ca}}}\right]10^{-6} \tag{T.2}$$

Where β_{ca} is between 300 and 600 µm/m, τ_{ca} is of the order of 100 days

Dying shrinkage (STT or TT1 UHPFRC)

For $t < t_s$ and environmental relative humidities of less than or equal to 80 %

$$\varepsilon_{cd}(t) = \frac{K[80 - RH](t - t_s)10^{-6}}{(t - t_s) + \beta_{cd}.h_0^{\,2}} \tag{T.3}$$

Where β_{cd} is between 0.003 and 0.01 days/mm², K is of the order of 5, RH: relative humidity of ambient environment (%), t_s: age of concrete at start of desiccation (days), h_0: equivalent radius of the cross section (mm)

For $t < t_s$ or environmental relative humidities greater than 80 %, $\varepsilon_{cd}(t) = 0$.

Basic creep (STT or TT1 UHPFRC)

The basic creep law for STT or TT1 UHPFRCs is given by the following expression:

$$\varphi_b(t, t_0) = \beta_{bc1}\varphi_{b0}\frac{\sqrt{t - t_0}}{\sqrt{t - t_0} + \beta_{bc}} \tag{T.4}$$

Where:

t_0: age of concrete at loading (days);

The values of the various parameters are given by the following expressions:

$$\beta_{bc} = \beta_{bc2}e^{2{,}8\frac{f_{cm}(t_0)}{f_{ck}}} \tag{T.5}$$

$$\varphi_{b0} = \frac{3{,}6}{f_{cm}(t_0)^{0{,}37}} \tag{T.6}$$

The values of the coefficients β_{bc1} and β_{bc2} are: β_{bc1} = 1.5 to 2.5 and β_{bc2} = 0.7

Basic creep (TT2 or TT1+2 UHPFRC, load applied after heat treatment)

$$\varphi_b(t, t_0) = \beta_{bc3} \frac{\sqrt{t - t_0}}{\sqrt{t - t_0} + 10} \tag{T.7}$$

Where:

t_0: age of concrete at loading (days)

The value of the coefficient β_{bc3} is: $\beta_{bc3} = 0.2$ to 0.5

Drying creep

For environmental relative humidities of less than or equal to 80 %

$$\varphi_d(t, t_0) = \varphi_{d0}\left[\varepsilon_{cd}(t) - \varepsilon_{cd}(t_0)\right] \tag{T.8}$$

Where:

φ_{d0} is between 20 and 50 for an STT or TT1 UHPFRC,
φ_{d0} is of the order of 20 for a TT2 or TT1+2 UHPFRC loaded after the application of heat treatments
For environmental relative humidities greater than 80 %, $\varepsilon_{cd}(t) = 0$.

Annex U
(normative)
Design of earthquake-resistant structures in UHPFRC

U.1 General

(1) The general provisions of Eurocode 8 are applicable.

NOTE The general provisions are articles 1, 2, 3, 4 and 10 of standard NF EN 1998-1 and its National Annex and articles 1, 2, 3 and 4 of standard NF EN 1998-2 and its National Annex. When these sections cross-reference Eurocode 2 for the design of UHPFRC structures regarding earthquake resistance, reference should be made to this standard. When the paragraphs of Eurocode 8 contradict or overlap U.1 (4) and U.1 (5) of this annex, provisions U.1 (4) and U.1 (5) apply.

(2) The paragraphs of standards NF EN 1998-1 and NF EN 1998-2 and their National Annex which deal with detailing rules related to reinforcement do not apply to members made of UHPFRC or comprising UHPFRC.

(3) Seismic action is defined in accordance with standard NF EN 1998-1 and its National Annex or standard NF EN 1998-2 and its National Annex.

NOTE The value of the design acceleration a_gS is set by the French Authorities.

(4) If the members of a structure, made up of UHPFRC or comprising UHPFRC, form part of the earthquake-resistant system, without justification of ductility, the structure must be justified assuming that its behaviour remains elastic. The determination of seismic forces may be performed by response spectrum analysis taking a behaviour coefficient q equal to 1.0 and the viscous damping coefficient ξ equal to 2.0 %. For the definition of stiffness, the raw (non-cracked) inertia is considered. Forces determined in this way must remain less than the resisting forces calculated with the ULS constitutive laws modulated by the seismic material partial factors γ given below. Displacements must be reassessed taking into account the cracking of the various members according to the stress level reached.

Table U.1 — Seismic material partial factors

Material	Notation	Seismic ULS
Compressed UHPFRC	γ_c	1.3
Tensioned UHPFRC	γ_{cf}	1.2
Reinforcing steel	γ_s	1.0

(5) If the members of a structure, made up of UHPFRC or comprising UHPFRC, form part of the earthquake-resistant system, to demonstrate this has a ductile, limited ductile (DCL) or medium-ductile (DCM) behaviour, these primary seismic members must be able to withstand alternate cyclic stresses in their plastic domain, equating to the ductility requirement. This ductility demonstration must be based on experiments representative of the members operational behaviour (materials, dimensions, stresses, boundary conditions). The seismic analysis will then be carried out on the basis of the test results incorporating the seismic material partial factors γ.

NOTE For the member to be considered as ductile, the force-displacement relationship must show a significant plateau for the resisting force and ensure dissipation of hysteretic energy over at least 5 complete deformation cycles up to the ultimate displacement corresponding to the ductility demand, without a reduction of more than 20% of the ultimate strength of the member.

Annex V
(informative)
Advanced designs

V.1 Control of the combined effects of loading and imposed deformations

(1) Modelling of the UHPFRC at early age may be used to control:

- temperature rises during setting, and their induced geometrical, chemical and mechanical effects
- the risk of cracking by combining loads and deformations or restrained differential deformations.

In this regard, the structural effects of shrinkage must be accounted for and superimposed onto the thermal effects and if applicable to the mechanical loads. The expected value of the structural effects of thermal and viscoelastic deformations must be taken into account both for determining the state at the end of construction and at a final stage for indefinite time, and for the verifications of deformations and forces in the structure during the execution phases.

(2) The analysis must cover the steps extending from the setting of the UHPFRC at form removal and possible handling, to the application of permanent and final loads. The heat calculation requires knowing a heat release master curve, and often requires resetting the duration of the dormant period according to temperature. The mechanical design mainly requires reliable data regarding the increase of the modulus when the concrete is new according to the hydration state reached, and reliable data in terms of shrinkage which assumes that early desiccation does not increase the amplitude of this deformation by hydric shrinkage acting on an incompletely structured material. These data depend strongly on the UHPFRC considered. Standard NF P 18-470 gives information on the experimental determination of these parameters.

V.2 Use of finite element models for designing structures in UHPFRC

V.2.1 Design assuming a linear behaviour of the UHPFRC

(1) The finite element calculation may be used for the design of structures in UHPFRC when these have a "complex" geometry fardifferent from the usual structures allowing the direct application of the Strength Materials underlying the formulae and verifications of this standard. The finite elements also allow a detailed analysis of areas where large and concentrated forces are introduced or stability analyses of complex slender members.

The use of linear elastic models for the finite elements analysis does not raise any specific problems, except the need, for thin members, to pay special attention to spatial discretisation by avoiding badly-conditioned mesh elements, due to the ratio of their dimensions.

In the case of UHPFRC members without reinforcing steel, the possible use of a strut-and-tie design model anyway requires knowledge of the actual orientation of the principal compression and tensile stresses before cracking (in particular through linear elastic finite element modelling) so as to define a kinematic strut-and-tie model compatible with the strains orientation in the structure before cracking.

V.2.2 Non-linear calculations

(1) Non-linear continuous constitutive models, or non-linear models with explicit representation of discrete cracks, enables analyses to be performed using non-linear finite element calculations of UHPFRC members with complex geometry beyond cracking. They may be used in addition or as an alternative to the methods in this standard based on beam analyses, beam lattices, plasticity methods for plates, and strut-and-tie methods subject to the following limitations:

- the anisotropy and spatial lack of uniformity of the post cracking behaviour induced by the distribution and orientation of fibres in the structure must be taken into account conservatively, taking account of the UHPFRC placement;

- the non-linear behaviour adopted in the calculation, if necessary based in the various directions and structure parts on a prediction of the distribution and orientation of fibres from flow models, must be compatible with this standard and standard NFP 18-470, be used as a reference for the specification of the UHPFRC and act as a basis for the acceptance of the material during the design, suitability and control tests;

- the model limit of validity in terms of strain must have been calibrated by reference to the actually experimentally observed behaviour of structures similar to that covered by the design;

- any justification by a non-linear model must be accompanied by a detailed analysis of the sensitivity of the main results of the calculation to the assumptions made.

V.3 Impact design

(1) When the structure in UHPFRC is subject to design loads arising from shock, impact or explosion, the increase in compression and tensile strength of the material may be allowed for, including in the post-cracking phase, as highlighted experimentally up to strainrates of the order of 1 s^{-1}. This allowance is permitted whether the justification is produced through use of a non-linear dynamic calculation or by using simplified methods (modeling of the structure as an oscillator, for example).

As a first approximation during the preliminary design studies, the indicative value of the increase in tensile strength of +1 MPa/u.log.10 (strain rate) may be used.

During the more detailed design phases in which shock behaviour is critical, the high strain rate tensile and compressive behaviour of the UHPFRC may be calibrated using data from the design study or from the identity card of the material. Standard NF P 18-470 "UHPFRC: specification, performance, production and conformity" gives information on this experimental identification.

V.4 Design of composite structures comprising UHPFRC

(1) The design of a composite structure comprising UHPFRC connected to other materials (steel, wood, ordinary concrete, fibre-reinforced polymers, etc.) uses the rules in this standard to justify the part of the structure made up of UHPFRC and adopts connection assumptions between the parts consistent with the type of connection planned and its execution conditions. If applicable, the restrained shrinkage of the UHPFRC associated with this connection is taken into account.

The overall calculation of forces is made by assuming an elastic behaviour of the components, allowing for a possibly reduced stiffness of the UHPFRC (depending on the level of the irreversible deformations expected).