

service d'Études techniques des routes et autoroutes

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METHODOLOGICAL GUIDE _____

APPLICATION TO CONCRETE HIGHWAY BRIDGES





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PREFACE

Guide EC2 - Version V7





On December 6 2004, on the occasion of the National Symposium on Eurocodes, and in the presence of Patrice Parisé, Director of Highways, the French engineering, construction and training professions announced a strong and unanimous commitment to the quick implementation of Eurocodes in the field of engineering structures, thus stressing the importance of a short period of coexistence between old and new regulations.

Involving institutional cooperation with Germany, a follow up to the implementation, Eurocode user guides, heightened awareness of the client, training etc., this guide, like the others, is part of a complete and specific totality of the actions of Sétra destined to the implementation of the Eurocodes in the field of engineering structures.

Whereas the Eurocodes are written in an encyclopedic manner, this guide agreeably takes up the French tradition of didactic regulation, linking texts, comments and national choices, accompanying the designer through the different stages of their task. In an effort to meet the requested demand we decided to write and distribute this guide quickly, drawing upon the strong involvement of engineers from the Scientific and Technical community and Sétra's knowledge of the philosophy of European texts. This guide may certainly be improved upon. The writing team, having proposed the first French bases of bridge design to Eurocodes, will be most satisfied that their work will give rise to a variety of fruitful discussions with the French and European professions.

Finally, beyond the circle of the writing team, this guide also reflects the great work accomplished by experts in standardization, particularly those who, like Ngoc-Vu Bui, a member of the Eurocode 2 European project team, listened to, talked with and convinced others to produce texts accepted by all, without renouncing the basic French positions.

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FOREWORD





After many long and laborious years in gestation, the Eurocodes have finally arrived. Without changing things too much, they do involve an important effort of familiarization, learning and appropriation on the part of the technical community, before becoming a tool for everyday work.

This last part of the road is not without its difficulties, nor its importance. On the contrary, it is even decisive to be able to legitimately crown with success such an ambitious program. Without a doubt many of the preparations were planned and will be implemented for this final step. Sétra, for its part, will contribute fully and will make guides on the subject of bridges available to the technical community. They will accompany the designers through this delicate period between old and new regulations. This is the need that triggered this guide "Eurocode 2 - Application to concrete highway bridges".

The guide begins with chapters 1 to 3 that deal with the generalities and basics for dimensioning and verification of projects.

It continues with discussions on concrete: shrinkage and creep in chapter 4 and prestress in chapter 5.

The justifications to be carried out at the ultimate limit states are then dealt with in chapter 6 where detail will be found on classic subjects such as verifications on bending, shear stress, torsional stress, crushing stress and fatigue stress. They are followed by a new subject: verification of brittle failure.

The justifications to be carried out on the service limit states are dealt with in chapter 7, where there are new developments concerning control of cracking.

Constructive provisions are the subjects of chapters 8 and 9, the first relating to reinforcement and the second to structural elements.

The last chapter, 10, brings together specific justification methods: verification relative to shear in special cases, the use of connecting rods and tie rods for zones of discontinuity, study of the prestress diffusion, 'sandwich method' for plate design and particularly the bending-shear combination. Foundations, treated in a very partial manner by Eurocode 2, should provide useful references for designers.

The guide finally ends with numerous and varied appendices. In effect, it likes to think it's complete without having been able to deal with everything, and to avoid burdening the reader with an excessive amount of information; much non-essential information and detailed developments of examples of application are to be found in the appendices.

The guide's first objective is a detailed description of the instructions that are new relative to previous practices. At this point of implementation of the Eurocodes, this development aims particularly to facilitate their understanding and their use. The numerous pages of the guide given over to them are there to give the maximum of explanation. It is however true that for certain instructions it is still too early to be able to well define their field of use, to estimate their importance and to evaluate the results of their application.

A second objective is trying to make the designer feel at home in this new voluminous entity full of multiple and diverse rules. Hence the reason for all this extra information, not only in Eurocode 2 itself, but also when it is necessary and thus useful to other Eurocodes.

And finally, the austere nature of such a work is inevitable. It is also accentuated by the very large number and diversity of the subjects treated. Further, a particular effort has been made to aim for simplicity and in the reasoning, or the logic in the linking of subjects, with the aim of providing a relative ease of reading. The guide will have succeeded in its aims if the designers quickly find it easy to use and practical, and if they refer to it often. This should not, however, excuse them from referring to Eurocode 2.



Chapter 1 - Introduction to Eurocode 2



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CHAPTER 1 -INTRODUCTION TO EUROCODE 2



I.INTRODUCTION

The European lavel whose aim is to contribute to the standardization of technical design regulations and of structure design. They also contribute to general European harmonization and to the elimination of the various constraints that might exist to the free transfer of products and delivery of services. Following a long gestation period leading to the appearance of experimental European standards (XP ENV or ENV), the present European (EN) are the result of the change over a relatively short time of this collection of experimental texts. They have clearly become more consistent and benefited from an updating of the latest technical and scientific developments.

The Eurocode 2 "EN 1992 Design of concrete structures" (at times designated by an EC2 suffix) deals especially with design and calculation of concrete structures. The concrete may be reinforced or not, prestressed, light or of normal density. Specific regulations for prefabricated concrete are also planned.

Eurocode 2 presently contains four parts:

- Part 1-1: General rules and rules for buildings
- Part 1-2: Structural fire design
- Part 2: Reinforced and prestressed concrete bridges
- Part 3: Liquid retaining and containing structures

Parts 2 and 3 show rules that apply to each structure dealt with.

The complete treatment of a structure necessitates reference to other Eurocodes or parts of Eurocodes, particularly Eurocode 0 "Basis of structural design" for principles, basic requirements and the combinations of actions, the various parts of Eurocode 1 for actions, Eurocode 7 for geotechnical design and Eurocode 8 for seismic design. This guide is limited to aiding in the use of Eurocode 2 applied to the design and calculation of concrete highway bridges.

This guide is based on published texts, and others are being written as it appears. The values of parameters taken from national annexes in preparation for the publication date of this guide are likely to be modified later. This will be announced and it is always advisable to refer to published documents.

Since total harmonization of the rules can only be achieved in practice after a certain period of use, it is planned at the initial stage that the Eurocodes propose options or parameters whose choice will be the responsibility of the various national authorities. These choices may give rise to different values than those recommended by the European texts. For each Eurocode, where necessary, forms the object of a national annex where their values are specified; this guide is based on French choices by referring to them and explaining them where appropriate.

II.NUMBERING PRINCIPLE

It is useful to present as a preamble the numbering principles of the clauses of the bridge section of Eurocode 2, and particularly the association with the main part.





The following principles have been adopted:

- At the beginning of each section, Eurocode 2, part 2 contains the list of the clauses of part 1-1 which apply to bridges.
- Next in the section are solely clauses that are new or have replaced clauses in part 1-1 following changes.

At times it is only the note of a clause that has been changed in Eurocode 2 part 2 (e.g. a recommended value). Even in this case, the corresponding clause of Eurocode 2 part 1-1 is considered as changed and is entirely taken up in Eurocode 2 part 2.

- When a clause from Eurocode 2 part 2 is new, its number is obtained by adding 101 to the number of the last clause of Eurocode 2 part 1-1 of the corresponding chapter.
- When a clause from Eurocode 2 part 2 replaces a clause from Eurocode 2 part 1-1 (changed clause), its number is obtained by adding 100 to the number of the clause it replaces.

Examples:

Clause 3.1.6 (101) of Eurocode 2 part 2 replaces clause 3.1.6(1)P of Eurocode 2 part 1-1 (the text is identical, but the recommended value for α_{cc} is different).

Clause 5.8.4(105) from Eurocode 2 part 2 is a new clause that comes after clause 5.8.4(4) of Eurocode 2 part 1-1.

Section 113 of Eurocode 2 part 2 is a new section that comes after section 12.

The operation of the national annex of Eurocode 2 part 2 is based upon the following principles, mentioned in the foreword to the national annex:

- The clauses cited are those from standard NF EN 1992-2: 2005;
- When the NF EN 1992-2 makes applicable a clause from NF EN 1992-1-1, this clause is applicable with the clause from the national annex of the corresponding NF EN 1992-1-1.



Eurocode 2 - Application to concrete highway bridges

Chapter 2 - Fundamentals of design and justification



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Eurocode 2 - Application to concrete highway bridges

Chapter 2 - Fundamentals of design and justification



I.BASIC REQUIREMENTS

The basic requirements formulated by the Eurocodes for the design and dimensioning of a project are those already found in previous regulations: They aim to assure for each structure adequate levels of **strength**, of **service properties** and of **durability**. However, the requirement relative to durability is formulated in a more explicit way and requires specification of a design working life that, for bridges, is generally taken as 100 years. Account is taken of the environment peculiar to each project via classifications of exposure, previously defined according to the nature of risks of corrosion and attack; it is also assumed that normal maintenance is planned and carried out.

The Eurocodes also assume that the design and the construction of structures are carried out by qualified and experienced personnel and that the monitoring and quality control are effective.

The requirements concerning the execution and the implementation must also be satisfied. As regards concrete structures, these requirements are dealt with in standard EN 13670.

II.JUSTIFICATION PRINCIPLE

The justification principle based on **verification of limit states** linked to the use of **partial factors** (of safety), now familiar to the designers, is retained and applied.

The limit states are classified in two categories:

The ultimate limit states (ULS) concerning the security of persons and of the structure, correspond to the static equilibrium limit, the resistance limit or the limit of dimensional stability. To that is added the fatigue limit state, and a resistance limit state reached under special conditions with service load levels.

- the service limit states (SLS) which concern the structure's operation, its durability, the comfort of its users and the appearance of the constructions, are defined by various appropriate limitations such as:

- the limitation of the concrete's tension or its non-decompression,
- the limitation of the tension in the reinforcement to prevent their plasticizing or their inelastic strain,
- the limitation of the width of the crack openings for control of cracking.

The partial factors are used to define the values of the calculations of the basic variables (actions, strength, geometric data), and to cover in part the many uncertainties that exist, to give the required margin of safety to the structure. Generally they act to increase actions and to reduce resistances.

The values of partial factors adopted by Eurocode 2 and those given in the ECO annexes are considered as leading to structures of reliability classification RC2 [EC2-1-1 2.1.1(2), EC2 Anx. C Tab. C.2]. For information, for bridges, a design and a road design in accordance with the various Eurocodes correspond to a target reliability index β in the order of 3.8 for the ULS resistance and for a design working life of 100 years.



Chapter 2 - Fundamentals of design and justification



III.JUSTIFICATION METHOD

The justification method generally consists of a **structural analysis** to determine the design stresses, but also of other characteristic values such as those of stresses, strains etc. Actions and/or combinations of actions are introduced into the structure design models, actions defined for the limit state studied and the phenomenon whose influence is under study. Then the results obtained from this analysis are compared with the values that are characteristic of attainment of the limit state linked to the phenomenon studied.

III.1.Justification at ULS

At ULS, regarding resistance for example, justification is often done by showing that the design internal forces and moments values (called actions effects, E, in the Eurocodes) are less than the design resistance.

This is shown by verification of the symbolic equation:

 $E_d \leq R_d$

The values E_d of the actions effects are obtained after factoring the actions by the various γ_F . The values R_d of design resistance are obtained after a decrease by the various γ_M of the properties of the materials figuring in determination of their resistance.

It should be noted that the guide adopts the simplified form of these partial factors, the form that includes the partial factor taking into account the model uncertainties. In practice and for most cases the numerical values supplied are normally given in this form.

III.2.

III.3. Justification at SLS

At SLS, generally, the service aptitude is demonstrated by checking that the values of the selected actions effects (E) do not exceed the limit value of the criterion (C) that characterizes the state limit considered.

This is done by verification of the symbolic equation in which the design values are found:

 $\mathbf{E}_{\mathsf{d}} \leq \mathbf{C}_{\mathsf{d}}$

The application of the justification method thus calls upon actions, their various combinations and the structural analysis that is defined and specified below.

IV.

V.ACTIONS AND COMBINATIONS OF ACTIONS

On the date of publication of this guide, the following national annexes appeared



Eurocode 2 - Application to concrete highway bridges

NF P06-100-2	Basis of design	June 2004
NF P06-111-2	General actions – densities, self weight	June 2004
NF EN 1991-	-1-2/NA Actions on structures exposed to fire	February 2007
NF EN 1991-	-1-3/NA Actions on structures. Snow loads	May 2007

V.1.Actions

The actions are the applied loads (forces and torque) or strains imposed on a construction. The more standard classification is shown in the table below:

Permanent actions	Variable actions	Accidental actions
Self weight	Operating loads	Impacts from vehicles or boats
Prestress	Loads during construction	Explosions
Weight, thrust, pressure of earth or water	Snow, wind, temperature	Earthquakes
Differential support displacements		Fall of movable element or of a prefabricated segment

It may be noted that displacements of supports and temperature effects are also considered as indirect actions or imposed deformations.

Shrinkage and creep may be considered as indirect actions but are dealt with separately as if originating from properties of concrete.

* Certain actions such as snow and earthquakes may be considered as accidental and/or variable actions according to the project site.

In France, for highway bridges, an earthquake is a solely accidental force. It is the subject of specific justification regulations developed in Eurocode 8.

V.1.1.Definitions and notations of major actions

• Permanent loads

G in fact symbolizes the minimum characteristic permanent loads $G_{k,inf}$ and the maximum $G_{k,sup}$ It also includes the possible effects of subsidence G_{set} and is broken down as follows:

Self weight:

The structure's self weight may be represented by a unique characteristic value and may be calculated on the basis of nominal dimensions and of mean unit masses [EC0 4.1.2(5)]. The specific density of normally reinforced or prestressed concrete is taken as 25kN/m³ [EC1-1-1 Anx. A Tab.A1]. The Eurocode 1 national annex proposes a weighting of 3% on the self weight of thin, prestressed structures (this clause does not generally apply to bridges).

Superstructure:

Eurocode 1 part 1-1: *Actions on structures - Densities, self weight and imposed loads for buildings* also show the additional provisions peculiar to bridges [EC1-1-1 5.2.3].





The lower and higher characteristic values of the weight of the pavement sealing and surface are obtained by taking a range of $\pm 20\%$ if the nominal value takes account of a surface following construction and of $\pm 40\%$ and -20% in the opposite case [EC1-1-1 5.2.3(3)]. The specific density of the poured asphalt and of the bituminous concrete is between 24 and 25kN/m³ [EC1-1-1 Anx.A Tab. A6].

The characteristic values of the self weight of tendons, pipes and service shaft are evaluated by taking account of a range of $\pm 20\%$ in relation to the nominal value [EC1-1-1 5.2.3(4)].

The characteristic values of the self weight of the other equipment (railings, gates, curbs..) are evaluated by taking account of their nominal values, except where otherwise noted in the project [EC1-1-1 5.2.3(5)].

• Major variable actions :

These are defined in the various parts of Eurocode 1:

TS In-tandem type traffic load and UDL uniformly-distributed traffic load. These two loads are the model for the LM1 major load [EC1-2].

q_{fk} uniform footpath load [EC1-2]

gr_i load group i, this idea allows clear definition of the various load model combinations to plan for in view of their simultaneous application on the structures [EC1-2]

 $F_w \,$ wind force. F^*_w represents the wind force in traffic and F_{wk} the wind force in the absence of traffic $\,[EC1\text{-}1\text{-}4]$

T temperature force. This force merits special treatment [Chapter 3-II.3].

Q_{Sn,k} snow force [EC1-1-3]

A_d accidental force

A_{Ed} seismic force

P a generic symbol, representing prestress as a probable value or as characteristic values [Chapter 5-IV].

V.1.2. ψ factors for highway bridges

The actions (F) normally act in combinations, either in characteristic values or by other representative values that are deducted from them after assignment of appropriate ψ factors:

- Ψ_0 for a combination value
- Ψ_1 for a frequent value
- Ψ_2 for a quasi-permanent value

The factors Ψ_0 , Ψ_1 and Ψ_2 for highway bridges are given in table A2.1 of annex A2 of Eurocode 0 and are shown in the following table:



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Load		Symbol	Ψ_0	Ψ_1	Ψ_2
		TS	0.75	0.75	0
Traffic load	gr1a	grla UDL		0.40	0
		footpath + cycle path	0.40	0.40	0
		grlb1 single axle	0	0.75	0
		gr2 horizontal force	0	0	0
		gr3 pedestrian traffic			0
		gr4 crowd			0
		gr5 special vehicles			0
	F	persistent situation (in service)	0.6	0.2	0
Wind forces	r _{wk}	Transient situation (execution)	0.8	-	0
		F_w^*		-	0
Temperature		T _k		0.6	0.5
Snow		Q _{Sn,k} during execution	0.8	-	0
Construction loads		Q _c	1.0	-	1.0

Design values of the actions are obtained from the partial γ_F factors and are combined between themselves according to the design situations to be examined that are mainly of three types:

- persistent design situation (mainly, the structure at its service start date and at the end of its design working life),
- transient design situation (e.g., the structure is being built or repaired),
- accidental design situation (mainly the structure is subjected to a shock, an earthquake).

Annex A of Eurocode 0 "Basis of structural design" define the rules and the methods adopted for establishment of these combinations of actions. For bridges, it is annex A2; other types of structure have their own appendix such as annex A1 for buildings, etc.

This guide restricts itself to definition of the major combinations most used for bridges.

V.2. Combinations of actions for ULS

V.2.1.Basic combinations for persistent and transient situations

Eurocode 0, for persistent and transient design situations allows a choice between the basic expression

$$\sum_{j \ge l} \gamma_{G,j} G_{k,j} + \gamma_P P + \gamma_{Q,l} Q_{k,l} + \sum_{i>l} \gamma_{Q,i} \psi_{0,i} Q_{k,i}$$
 [EC0 Expr.(6.10)]

and the alternative expressions

$$\begin{cases} \sum_{j\geq l} \gamma_{G,j}G_{k,j} + \gamma_{P}P + \gamma_{Q,l}Q_{k,l} + \sum_{i\geq l} \gamma_{Q,i}\psi_{0,i}Q_{k,i} \\ \sum_{j\geq l} \xi_{j}\gamma_{G,j}G_{k,j} + \gamma_{P}P + \gamma_{Q,l}Q_{k,l} + \sum_{i\geq l} \gamma_{Q,i}\psi_{0,i}Q_{k,i} \end{cases}$$
[EC0 Expr.(6.10a) and (6.10b)]





The permanent loads G_k and the variable loads Q_k are taken as characteristic values, whereas the prestress is taken as a probable value for verifications at ULS [EC2-1-1 5.10.8(1)].

For application to highway bridges, railroads and foot bridges and for **verification of structural elements (STR) not subjected to geotechnical actions,** the national annex requests the use solely of the basic expression (6.10) that translated according to actions, partial and classic factors, takes the following major forms [EC0 Tab.A2.4(B)]:

$$\left\{\sum_{j\geq 1} (1,35 \,\mathrm{G}_{\mathrm{k},j,\mathrm{sup}} + 1,00 \,\mathrm{G}_{\mathrm{k},j,\mathrm{inf}})\right\} + \gamma_{\mathrm{P}} \mathrm{P} + \left\{\begin{array}{l} 1,35 \,(\mathrm{TS} + \mathrm{UDL} + \mathrm{q}_{\mathrm{fk}}) + 1,50 \,\mathrm{Min}(\mathrm{F}_{\mathrm{w}}^{*}; 0,6 \,\mathrm{F}_{\mathrm{wk}}) \\ 1,35 \,\mathrm{gri}_{i=1b,2,3,4,5} \\ 1,5 \,\mathrm{T}_{\mathrm{k}} + 1,35 \,(0,75 \,\mathrm{TS} + 0,4 \,\mathrm{UDL} + 0,4 \,\mathrm{q}_{\mathrm{fk}}) \\ 1,5 \,\mathrm{F}_{\mathrm{wk}} \\ 1,5 \,\mathrm{Q}_{\mathrm{Sn},\mathrm{k}} \end{array}\right\}$$

The alternative expressions [EC0 Expr.(6.10a) and (6.10b)] should thus not be used for bridges and foot bridges.

The partial factor relative to the prestress γ_P is valued at $\gamma_{P,fav} = 1$ if the prestress has a favorable effect, and $\gamma_{P,unfav} = 1.2$ if the effects are unfavorable for verification of local effects [EC2-1-1 2.4.2.2]. During verification of the stability limit state in the presence of an external prestress, if the prestress may be unfavorable $\gamma_{P,unfav} = 1.3$ must be considered (unless there are sufficient deviators along the buckled length, the national annex specifies).

V.2.2.Basic combinations for accidental and seismic situations

2.2.0.a) Accidental situation

The accidental combinaison at ULS is given by:

$$\sum_{j\geq l} G_{k,j} + P + A_d + (\psi_{1,l} \text{ ou } \psi_{2,l}) Q_{k,l} + \sum_{i>l} \psi_{2,i} Q_{k,i}$$
 [EC0 6.4.3.3]

or

$$\left\{\sum_{j\geq l} \left(G_{k,j,sup} + G_{k,j,inf}\right)\right\} + P + A_d + 0.6 T_k + Q_c$$

But generally speaking the precise definition of combinations such as actions to be taken account of may be found in Eurocode 1 part 1-6 and its national annex [EC2-2 113.2] or in the contract documents of particular projects.

An accidental situation, particularly for bridges built as balanced cantilever method is often envisaged and may occur during construction when a segment or a mobile element falls [EC2-2 113.2 (103) and (104)]. The combination of particular corresponding actions may be found in the national annex of Eurocode 1 part 1-6 or in the guide on bridges built as balanced cantilever method.

2.2.0.c)Seismic situation

$$\sum_{j\geq l} G_{k,j} + P + A_{Ed} + \sum_{i\geq l} \psi_{2,i} Q_{k,i}$$

[EC0 6.4.3.4]



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The combination is only mentioned in a formal way and for reference since its use as associated analysis methods is special and peculiar to Eurocode 8 and it is strongly suggested the designer refer to it.

V.2.3. Combination for verification regarding fatigue

The combination that allows calculation of stress range is [EC2-1-1 6.8.3]:

$$C_0 + Q_{\text{fat}}$$

and is thus broken down as:

• a basic combination C₀ of **non-cyclic** loads, presented in Eurocode 2 as similar to the frequent combination used for SLS expressed in the form:

•
$$C_0 = \sum_{j\geq 1} G_{k,j} + P + \psi_{1,1} Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i}$$
 [EC2-1-1 6.8.3(2)P]

If the cyclic loads, which may be a traffic load or a wind action are excluded from this combination, the only variable non-cyclic force left in this combination is the frequent value of thermal action. It is thus the average state of the structure in service, under permanent loads and frequent temperature variations, which serve as the reference point to determine the variations in stress caused by the cyclic fatigue load.

• the cyclic fatigue load Q_{fat} that may be the traffic load or the wind force.

Except for special cases (structures sensitive to wind forces), the cyclic fatigue load Q_{fat} corresponds to the passing of trucks and is represented by the fatigue load models [Chapter 6-V].

As for the prestress, in compliance with [EC2-1-1 5.10.9], it comes in the combination of verification by its characteristic values. This is justified in this case by the verification process that uses SLS type stress calculations and makes up an exception to the general rule decreed for all ULS type verification.

V.3. Combinations of actions for SLS

At SLS the prestress is to be taken into account with its characteristic values [EC2-1-1 5.10.9]. The permanent loads $G_{k,j}$ integrate the subsidences G_{set} as well as the shrinkage and creep effects.

V.3.1. Characteristic combination

•
$$\sum_{j\geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \Psi_{0,i} Q_{k,i}$$
[EC0 6.5.3.a)]

Similarly, in terms of classic parameters, it may take the following forms:

$$\left\{\sum_{j\geq 1} (G_{k,j,sup} + G_{k,j,inf})\right\} + P_k + \begin{cases} (TS + UDL + q_{fk}) + Min(F_w^*; 0, 6F_{wk}) \\ gri_{i=1a,2,3,4,5} + 0, 6T_k \\ gr1b \\ T_k + (0,75TS + 0, 4UDL + 0, 4q_{fk}) \\ F_{wk} \\ Q_{Sn,k} \end{cases}\right\}$$

V.3.2. Frequent combinations

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$$\sum_{j\geq l} G_{k,j} + P + \psi_{l,l} Q_{k,l} + \sum_{i>l} \psi_{2,i} Q_{k,i}$$

[EC0 6.5.3.b)]

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or in a more practical form

$$\left\{\sum_{j\geq 1} (G_{k,j,sup} + G_{k,j,inf})\right\} + P_k + \begin{cases} (0,75 \text{ TS} + 0,4 \text{ UDL} + q_{fk}) + 0,50 \text{ T}_k) \\ 0,75 \text{ gr}4 + 0,5 \text{ T}_k \\ 0,75 \text{ gr}1b \\ 0,6 \text{ T}_k \\ 0,2 \text{ F}_{wk} \\ 0,5 \text{ Q}_{\text{Sn},k} \end{cases}\right\}$$

V.3.3.Quasi-permanent combinations

•
$$\sum_{j\geq 1} G_{k,j} + P + \sum_{i\geq 1} \psi_{2,i} Q_{k,i}$$
[EC0 6.5.3.c)]

or in a more explicit and practical form:

$$\left\{\sum_{j\geq l} \left(G_{k,j,sup} + G_{k,j,inf}\right)\right\} + P_k + 0.5 T_k$$

VI.

VII.JUSTIFICATIONS IN CONSTRUCTION STAGES

VII.1. Generalities

The justifications to be done in construction stages are not explicitly presented in Eurocode 2 part 1-1. On the other hand, Eurocode 2 part 2 dedicates a special section, no. 113.

It is advisable to check the SLS and the ULS in construction as soon as

- the actions, other than those applied on the finished structure, are applied,
- The static diagram is modified between construction stages and the service phase and this causes a redistribution of forces,
- the construction stages have an influence on the stability or the geometry or on the stresses in the finished structure.



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The justifications in construction stages may essentially be distinguished from justifications in operating phases, on the one hand by the nature and/or intensity of the applied loads and on the other hand by the type of construction.

The specific loads to take into account are a series of site loads adapted to the method of construction of the structure [EC1-1-6].

The simultaneousness of the loads should be adapted to the project site situation [EC0 A2.2.1(8)]. In particular, the combination of snow and wind loads with the site loads should be defined for each project.

The combination of personnel and climatic actions (snow, wind) is not to be effected.

It must be noted that account should be taken of a differential vertical (that may rise and fall) and horizontal wind, for non-exceptional structures, particularly in verifications during construction [EC2-2 113.2(102)].

A value of 200 N/m^2 is recommended by Eurocode 2 but the national annex refers to Eurocode 1 parts 1-4 and parts 1-6 and their respective national annexes.

Eurocode 2 part 2 notes, without giving details, that it is advisable for through bridges to take account of the strains imposed [EC2-2 113.2(105)].

The type of construction may also lead to a consideration of accidental situations, as for example in the case of realization of a bridge beam constructed by balanced cantilever method where it is required to take account of the fall of movable elements (case of construction of a deck cast in-situ) or of a segment (case of a prefabricated segment deck).

Finally, we could also point to verifications of the resistance of the structure and verifications of static equilibrium: the design situation to consider is a transient situation.

VII.2. Special combinations for construction stages

VII.2.1.Verification of structural resistance

The basic combination of the ULS is given by the national annexes of Eurocode 0 Annex A2 and Eurocode 1 part 1-6.

VII.2.2. Verification of static equilibrium

The basic combination of the ULS is given by the national annexes of Eurocode 0 Annex A2 and Eurocode 1 part 1-6.

It is particularly necessary to verify the stability of the beams of the structures built by balanced cantilever method. In this case the verifications of support struts and nailing tendons are similar to those described in the Sétra guide on bridges built by balanced cantilever method, with the combinations given by the Eurocode 1 part 1-6 national annex.

VII.2.3. Verification at SLS

Eurocode 2 part 2 requires that the same SLS verifications be done for the execution phases but specifies that certain service aptitude criteria may not be applied or applied with less strict conditions, so long as the durability and the appearance of the finished structure are not affected. The following criteria are cited:





- strain limit [EC2-2 113.3.2(102)]
- tensile limit for concrete to be verified under a quasi-permanent combination [EC2-2 113.3.2(103)]
- control of cracking to be ensured under quasi-permanent combination [EC2-2 113.3.2(104)]
- It is clear that the service aptitude criteria should be adapted for the construction stages. It is necessary to do likewise for the prestress design rules that were added to complete the first ones. The details of these adapted rules are shown in [Chapter 7 III].

VIII.

IX.STRUCTURAL ANALYSIS

The aim of the structural analysis dealt with in section 5 of Eurocode 2 is determination of distribution of stresses, strains, deformations and structural displacements [EC2-1-1 5.1.1(1)P]. To this effect it is necessary to do a modeling of the structure's geometry, but also a modeling of its performance by means of assumptions on the behavior of the materials, and on the connections with the outside environment

Eurocode 2 specifies that for most standard cases the structural analysis will serve mainly to determine the distribution of stresses, strains and structural displacements. It is this more restrictive definition that will be adopted in the follow-up to this guide to distinguish it from justification of the sections.

Moreover, when they have a significant influence, the ground-structure interaction (as for deep foundations for example) and the second order effects (as for structures sensitive to deformations) must be taken into account.

The structural analysis must be completed by local analysis for verification of special points. It is the case for example of the verifications of junction zones or of zones where stresses are concentrated (an application of a prestressed anchored force is given in [Chapter 10-III]).

In any event, the structural analysis must take account of the geometric imperfections that include discrepancies both in relation to theoretical geometry of the structure and in the load positions. These geometric imperfections are to be taken account of only in the ULS.

It is specified that the discrepancies in the dimensions of the sections are in principle already taken into account in the partial factors relative to the materials.

It may also be noted that the information to quantify the geometric imperfections [EC2-1-1 5.2] is supplied for the elements subjected to a axial stress and to vertical structural elements subjected to vertical loads. They are effectively the elements or structures that are most sensitive to these effects.

IX.1.<u>Geometric imperfections</u>

There is no general rule in part 1-1 of Eurocode 2 that gives valid rules for buildings and elements subjected to an axial compression or to vertical loads.



The imperfections are thus represented by an inclinaison θ_i [EC2-1-1 5.2(5)].

For the bridge elements subjected to the same conditions described above, part 2 of Eurocode 2 changes this value and gives:

$$\begin{split} \theta_{i} &= \theta_{0} \times \alpha_{h} \ [\text{EC2-2 5.2(105)}] ,\\ \text{with} \begin{cases} \theta_{0} &= \frac{1}{200} & \text{valeur de base} \\ \alpha_{h} &= \frac{2}{\sqrt{L}} \leq 1 & \text{coefficient de réduction avec L longueur de l'élément en mètre} \end{cases} \end{split}$$

From the inclinaison the effects of the imperfections may be taken into account by an eccentricity equal to:

$$e_i = \theta_i \frac{L_0}{2}$$
 [EC2-1-1 5.2(5)],

for application to bridge piers, L₀ is the buckling length.

The following examples illustrate this rule, and show that the eccentricity to take into account is not necessarily equal to the inclination multiplied by the height:





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Fig./Tab.1.1)Geometric imperfection: case of a pier

For a pier perfectly bi-embedded, the buckling length is $\frac{H}{2}$ and hence the eccentricity value to apply is

$$e_i = \theta_i \times \frac{H/2}{2} = 0.25 \theta_i \times H$$
.

For arch bridges, part 2 of EC2 adds the following supplementary rule:

"It is advisable to establish the form of the imperfections at the horizontal and vertical levels from the deformation of the first mode of horizontal and vertical buckling respectively. Each modal deformation may be represented by a sinusoidal profile whose amplitude is equal to $a = \theta_i \frac{L}{2}$ where L is the "half wave-length"

[EC2-2 5.2(106)].

The wavelength of the buckling mode of an arch corresponds to the period of the sinusoid representing the modal deformation. In the case of a double-jointed arch and for the first mode of buckling, this length is equal to the developed length of the arch. The half wave-length generally corresponds to the buckling length, and hence the amplitude "a" to take into account corresponds to the equivalent eccentricity given in this general case.



Arch double jointed at its ends:

$$\mathbf{e}_{i} = \mathbf{\theta}_{i} \frac{\mathbf{L}_{0}}{2} = \mathbf{\theta}_{i} \frac{\mathbf{L}}{4}$$

Arch double embedded at its ends: $e_i = \theta_i \frac{L_0}{2} = \theta_i \frac{\alpha L}{2}$

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Longueur de flambement	Buckling length
Longueur d'onde	Wave length
Longueur développée = longueur d'arc	Developed length = length of the arch

As with part 1-1 of Eurocode 3, it is possible to generalize the rule $e_i = \theta_i \frac{L_0}{2}$ for all types of structure. The major difficulty, for a complex structure, lies in determining the buckling length value, in order to determine the initial imperfection.

IX.2. Modeling of geometry of structures



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The geometry is normally modeled by considering the structure as made up of linear elements, plan elements and on occasion walls, or in the language to which designers are accustomed beams, slabs, plates and walls.

In certain special cases, it may be interesting for the designer to base himself on some simple criteria given for buildings [EC2-1-15.3.1(3) to (7)] to be able to consider a voluminous element such as a slab or a beam. The idea of a beam- shear wall is also explained.

IX.2.1.Contributing width

The contributing width of the compression flange of a T section or an angle is to be calculated.

Clause [EC2-1-1 5.3.2.1] gives a detailed illustration of the contributing width of the compression flange for a T section with several continuous spans. This width varies according to whether spans or an intermediate support are used. But where extreme accuracy is not required and for a structural analysis, a constant width may be adopted and taken as equal to the value used in spans.

This definition may also be used for an independent span or for the lower member of a box girder.

This participating width intervenes in the dimensions of the sections adopted for the structural analysis, but also for stress calculation.

IX.2.2. Gross sections

The concrete sections resulting from the design document serve to determine the gross sections that may generally be used in structural analyses and for determination of self weight stresses.

IX.2.3.Effective span

The rules for determination of the effective span l_{eff} , for different support conditions of the building elements, are given in [EC2-1-1 5.3.2.2]. These rules proved to be interesting for bridge elements and are made applicable by Eurocode 2 part 2.

IX.3.<u>Modeling of structure behavior</u>

The two major categories of behavior model used for structural analysis – the linear performance or the nonlinear performance – stem particularly from consideration of theoretical or actual stress-strain diagrams of the materials.

Only stress-strain diagrams of materials used for structural analysis are shown in this chapter, since Eurocode 2 distinguishes between these diagrams, for example [EC2-1-1 3.1.5] for concrete) and those to use for verification of sections ([EC2-1-1 3.1.7] still for concrete) that will be described during treatment of calculation of sections [Chapter 6-I]. Similarly, the specific diagrams used by special methods of analysis will be dealt with during description of these methods.

IX.3.1.Stress – strain diagrams

3.1.0.a)For a model of linear-elastic behavior

The hypothesis of a linear and elastic behavior of steel and concrete is assumed, which corresponds to a theoretical situation allowing particularly the use of linear stress-strain diagrams. Further, the sections of the elements are assumed not to be cracked and the structural analysis is carried out using the values of deformation moduli E_s for the reinforcement, E_p for prestressing steel and E_{cm} for the concrete.





3.1.0.b)

3.1.0.c)For a model of non-linear behavior

The hypothesis of a non-linear but elastic performance of steel and concrete is assumed, which is nearer to their actual performance when stresses increase. Further, cracked concrete is not taken into account and the structural analysis is carried out using the stress-strain diagrams shown below:

o Stress-strain diagram of concrete



Fig./Tab.1.1)Stress-strain diagram of concrete

where

 ϵ_c relative variable compressive strain of concrete

$$k = \frac{1,05 \text{ E}_{cm} |\epsilon_{cl}|}{f_{cm}}$$

$$\epsilon_{cl}(\%_{00}) = \min(0,7 (f_{ck} + 8)^{0.31}; 2,8) \quad \text{strain at peak stress}$$

$$\epsilon_{cul}(\%_{00}) = \begin{cases} 3,5 & \text{pour} \quad f_{ck} < 50 \text{ MPa} \\ 2,8 + 27 \left[\frac{98 - (f_{ck} + 8)}{100} \right]^4 & \text{pour} \quad f_{ck} \ge 50 \text{ MPa} \end{cases} \quad \text{relative ultimate strain}$$

$$f_{cm} = f_{ck} + 8 \quad \text{mean compressive strength of concrete at 28 days}$$

$$E_{cm} = 22000 \left(\frac{f_{ck} + 8}{10}\right)^{0.3}$$
 mean value of secant elastic modulus of concrete

o Stress-strain diagrams of prestressing steels





Α

-p0,1k \mathcal{E}_{uk} / E_n

Fig./Tab.1.2)Stress-strain diagrams of prestressing steels

The diagram on the left has a linear part with a slope E_p to $\sigma \le 0.9 f_{p0,1k}$ followed by a curve having as equation:

$$\varepsilon_{p} = \frac{\sigma}{E_{p}} + 100 \left(\frac{\sigma}{f_{p0,1k}} - 0.9\right)^{5}$$

0,1%

The diagram on the right is developed from coordinate limits of the two branches (ε, σ) :

- An elastic branch
- An upper inclined branch

with, according to the appropriate EN10138 or the ATE (Agréments techniques européans Technical European Agreements) , ε_{uk} elongation under maximum load

 $\begin{pmatrix} \epsilon_{uk} ; f_{pk} \end{pmatrix}$

 $\left(\frac{\mathbf{f}_{p0,1k}}{\mathbf{E}_{p}};\mathbf{f}_{p0,1k}\right)$

 f_{pk} is the tensile strength

 $f_{p0,1k}$ is the conventional yield strength at 0.1%

and in the absence of a precise value one may use $\frac{f_{p0,lk}}{f_{rk}} = 0.9$.

For both these diagrams a value of E_p may be taken as equal to 195 GPa for the strands [EC2-1-1 3.3.6(3)].

The two linear and non-linear behavior models are used in four kinds of structural analysis:

- elastic-linear analysis with limited redistribution, plastic analysis
- non-linear analysis.
- Stress-strain diagram of steels for reinforced concrete







Fig./Tab.1.3)Stress-strain diagram of steels for reinforced concrete

With no clear recommendation from Eurocode 2, the diagram used was defined consistent with that given for prestressing steels. It is developed with the coordinate limits of the two branches (ϵ, σ)

• An elastic branch $\left(\frac{f_{yk}}{E_s}, f_{yk}\right)$ • An upper inclined branch $\left(\epsilon_{uk}, k f_{yk}\right)$

with

f_{yk} yield strength

 ε_{uk} strain and k according to reinforcement classification

for classification B	$\epsilon_{uk} \ge 5\%$	k≥1,08
for classification C	$\varepsilon_{uk} \ge 7,5\%$	$1,15 \le k < 1,35$

and a value of E_s that may be taken as equal to 200 GPa [EC2-1-1 3.2.7 (4)].

For the choice of the classification of reinforcement to use see [Chapter 3] of this guide.

IX.3.2.Elastic-linear analysis

- The elastic-linear analysis [EC2-1-1 5.4] is done using the elastic-linear model and serves in the vast majority of cases, for both SLS and ULS. The basic hypotheses seen in [EC2-1-1 5.4(2)] assume:
 - An elastic behavior of materials
 - Non-cracked sections
 - Linear stress-strain relationships for both concrete and steel
 - Mean values of elasticity modulus E_s , E_p et E_{cm} .
- The process of ULS justification is generally done, once the stresses are determined, by a verification of sections' strength, carried out using stress-strain diagrams used for this purpose [I 6-I].

0

0

I



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Clause [EC2-1-1 5.4(2)] allows the used of reduced rigidities corresponding to cracked sections in an elastic-linear analysis for the effects of thermal strain, subsidence and shrinkage at ULS. This allows, where applicable, moderation of the thermal gradient effects when this force proves to be a disadvantage [Chapter 3-II.3IV.3].

IX.3.3.Elastic-linear analysis with limited redistribution

The elastic-linear analysis with limited redistribution [EC2-1-1 5.5] starts with an elastic-linear analysis but ends with a redistribution of forces (generally speaking support moments and consequently an increase in span moments) respecting the equilibrium conditions and if applicable the conditions relative to the rotation capacity of the sections. Destined to be used for a ULS justification, it continues, once the stresses are redistributed, by a verification of the resistance capacity of the sections as explained in the previous paragraph.

• This type of analysis is not normally available for bridges. If the stresses must be determined more accurately including all aspects of redistribution (by creep, by cracking or by plasticizing), non-linear analysis may be used.

In practice this method is used to justify, in an operating situation and at the ULS, the midspan structural sections built by balanced cantilever method. The redistribution is then done by transfer of one part of the bending moments to the sections with intermediate support that have been more often dimensioned during the construction stage and hence have strength in reserve.

IX.3.4.Plastic analysis

Plastic analysis [EC2-1-1 5.6] is strictly for the only justifications of the ULS and for an infrequent use in France on bridges. In practice, this is done almost uniquely by use of the yield lines method (a cinematic method of the theory of plasticity) for verification of slabs.

It may be noted that the justifications regarding shear stress and torsional stress in Eurocode 2 are also in part based upon plasticity theory and that the connecting rod and tie rod method is one application of it. They are based more on the "resistance model" aspect of this theory.

IX.3.5.Non-linear analysis

It is understood that a non-linear analysis of a structure takes account of the non-linear performance of the materials basing on more realistic stress-strain diagrams. [EC2-1-1 5.7(1)]. Also considered is a greater deformability of concrete when the compression level to which it is subjected increases, as with that of steel above its conventional yield strength. Concrete, moreover, when it is in tension, is not considered in the calculations. It is thus the non-linearity of the materials that is taken into account.

Non-linear analysis:

- uses non-linear stress-strain diagrams, for concrete as for steels for reinforced and prestressed concrete that are described in [3.1.0.c],
- ignores the resistance of concrete in tension,
- and may be used for SLS and ULS.

•

In principle, the complete justification process is similar to that using linear analysis: for example, for ULS, it begins by a non-linear structural analysis [EC2-1-1 5.7, 5.8.6(1)P, 5.8.6(2)P, start of 5.8.6(3)] to determine the ultimate stresses. This is then completed by a verification of the resistance capacity. In this second stage the stress-strain diagrams of the material used are modified, particularly by incorporating the various partial factors





for the materials, γ_M . Similarly, a separation is found between the structural analysis strictly stated and the verification of the sections.

Two general non-linear analysis methods are proposed in Eurocode 2. The first, classical, is given in Eurocode 2 part 1-1. The second, proposed for bridges in Eurocode 2 part 2, is more innovative. These two methods will be applied to verification of stability of bridge piers [VII].

It will be noted that Eurocode 8 also develops specific non-linear analysis methods.

IX.3.6. First and second order analysis

Structural analyses may also be classified according to whether they are carried out as first order or as second order. In a first-order analysis, the hypothesis of small displacements applies and the state of equilibrium of the structure is obtained in its initial geometry. In a second-order analysis, the equilibrium of the structure is verified in its deformed geometry, or what is known as geometric non-linearity and the consideration of additional effects of the forces created by deformations of the structure.

The various types of analysis may thus combine, covering a range from the simplest, first-order analysis to the most complex and complete, the non-linear second-order analysis (at the same time non-linearity of materials and geometric non-linearity).

Generally, the simplest combination that gives first-order linear analysis may be used in the majority of cases at SLS and ULS.

Eurocode 2 gives a criterion to ignore or take into account second-order effects: they may be ignored if they represent less than 10% of the corresponding first-order effects [EC2-1-1 5.8.2(6)]. This general rule is seen in practice by simplified criteria relative to the slenderness ratio [EC2-1-1 5.8.3.1] to firstly quickly identify the elements for which a second-order analysis is not necessary; unfortunately they apply only to isolated elements.

The limit of 10% of development of first-order effects is conventional and serves as a basis for grading the simplified rules relative to the slenderness ratio.

When the second-order effects are taken into account, the equilibrium and the resistance should be verified in the deformed state. The non-linear performance model should be used for the structural analysis [EC2-1-1 5.1.1(7)]. The strains should be calculated in taking account of cracking (where the tensile strength f_{ctm} is exceeded, tensioned concrete is ignored), of the non-linear properties of the materials and of creep [EC2-1-1 5.8.2(2)P]. It is for example a study of form stability, with justifications, that might logically be done at SLS.

Second-order effects should be taken into account at SLS in the case of very deformable materials. The verifications to be carried out concern classic criteria of stress limitations, of crack opening or, if the case arises, of deformations. Unlike the ULS justifications, the analysis is carried out with no consideration for the initial imperfections and with the hypothesis of the linear elasticity of the materials. Concrete in tension is still not considered if the tensile stress value is greater than f_{ctm} .







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I.

II.DATA RELATIVE TO EUROCODE 2

III.

Eurocode 2 emphasizes the durability of concrete structures by devoting all its fourth section, entitling it, albeit simplistically, "Durability and cover to reinforcement". This durability, in effect, that is understood from the point of view of technical performance for a given design working life, is closely linked to various other aspects of the project such as the concrete quality (refer to EN 206-1), the environmental conditions (the physical and chemical conditions to which the structure is exposed) or the structure's maintenance program.

Eurocode 2 wished to contribute even more decidedly to the durability objective, by establishing a link between the environmental conditions (through the definition of **exposure classifications**) and the protection of reinforcement (via the rules concerning **cover**). It must be remembered that the major part of the SLS dealt with in Chapter 7 of this guide is also meant to obtain ongoing projects, as the major constructive provisions.

II.1. Exposure classifications

The first important elements of a project are clarification of the exposure classifications for the various concrete walls of the elements of the structure according to their environmental conditions. Six categories of exposure classification are proposed in table 4.1 [EC2-1-1 4.1], in compliance with EN 206-1.

Designation of classification	Description of environment:	Examples illustrating the choice of exposure classifications
1 No risk of corro	osion nor attack	
X0	Non-reinforced concrete; no embedded metal parts: all exposures except freeze/thaw, abrasion and chemical attack Concrete reinforced or with metal parts embedded: very dry	Concrete inside buildings where humidity of ambient air is very low.
2 Corrosion cause	ed by carbonation	
XC1	Permanently dry or wet	Concrete inside buildings where humidity of ambient air is very low. Concrete permanently under water
XC2	Wet, rarely dry	Concrete surfaces subjected to long-term contact with water Large number of foundations
XC3	Moderate humidity	Concrete inside buildings where humidity of ambient air is moderate or high Concrete outside, sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces in contact with water, but not same exposure classification as XC2
3 Corrosion cause	ed by chlorides	
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete elements exposed to chloride-bearing industrial waters





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XD3	Cyclic wet and dry	Bridge elements exposed to sprays containing chlorides Pavements
1 Corresion course	d by ablamidas from see water	
4 Corrosion cause	Euroged to air containing and solt, but not in	Structures on or near a coast
	direct contact with sea water	
XS2	Permanently immersed	Elements of marine structures
XS3	Zones of tidal range, zones subjected to sea spray	Elements of marine structures
5. Freeze/thaw at	tack	
XF1	Moderately saturated in water, without de- icing agents	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderately saturated in water, with de-icing agents	Vertical concrete surfaces of road construction exposed to freezing and to air carrying deicing products
XF3	Heavily saturated in water, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	Heavily saturated in water with de-icing agents or sea water	Bridge roads and decks exposed to de-icing agents Vertical concrete surfaces directly exposed to de-icing agents spray and to freezing Zones of marine structures subjected to spray and exposed to freezing
6. Chemical attac	k	
XA1	Low-level chemical attack environment according to EN 206-1, Table 2	Natural soils and ground water
XA2	Moderate-level chemical attack environment according to EN 206-1, Table 2	Natural soils and ground water
XA3	High-level chemical attack environment according to EN 206-1, Table 2	Natural soils and ground water

Fig./Tab.I.(1): Exposure classifications according to environmental conditions in compliance with EN 206-1 [EC2-1-1 Tab.4.1]

Classification X0 would only under exceptional circumstances corresponds to bridge situations. In effect, it is the case of concrete assumed to be sheltered inside constructions where the humidity in the air is maintained at a low level and where there is little risk of corrosion of the reinforcement.

The last two classifications (XF or XA) that characterize additional specific risks linked to freezing and thaw conditions (classifications XF1 to XF4) or chemical attack (classifications XA1 to XA3) are juxtaposed to the other classifications when these risks exist, and have the effect of requesting appropriate measures to do with the concrete composition [EN 206-1 Anx.F]. Determination of the cover is done exclusively from the remaining classifications.

Thus one turns generally to one of the three categories of classifications XC, XD or XS, depending on whether the concrete reinforcement may be subjected respectively to risks of corrosion by carbonation, by chlorides or by chlorides from sea water.

The final exposure classification XCi, XDj or XSk may thus be determined according to the last column of table 4.1, using examples given for information. The national annex makes this column normative and gives it very useful information via a series of notes to facilitate and better target the choices.

As regards the structural choices, there is linked to each environmental classification a classification indicative of a minimum strength the concrete must have [EC2-1-1 Anx.E Tab.E.1N]. Attention is drawn to the fact that the choice of a concrete of a durability required for its own protection and for the protection of the reinforcement, may lead to a compressive strength of the concrete greater than that required by the design of the structure from the resistance viewpoint.





The major additional information concerning bridges and contained in the national annex [EC2-1-1/AN] is given in the notes in table 4.1. It is repeated below.

Note 3: In XC4 the overhead parts of the engineering structures are to be classified, including the returns of these parts by passage and/or splashing of water.

The classification in XC4 takes account of an external environment where even the parts sheltered from the rain are likely to be subjected to runoff or spray, and where the level of carbonic gas in the air around the structure is quite high.

Note 6: In France, the exposure classifications XF1, XF2, XF3 and XF4 are shown on the map giving freezing zones [EN 206-1/AN NA4.1 Fig.NA.2 and Note].

For these exposure classifications XF, and subject to respect of the provisions linked to concrete (EN 206-1 and national standard documents), the cover will be determined by reference to an exposure classification XC or XD, as shown in 4.4.1.2 (12).

		Exposure classification				
		XF1 XF2 X		XF3	XF4	
Type of salting (cf. Freezing recommendations 2003)	Infreque nt	XC4	Not applicable	For formulated concrete No air-entraining agent XC4 With air-entraining agent XD1	Not applicable	
	Frequent	Not applicable	XD1, XD3 for highly exposed elements*	Not applicable	XD2, XD3 for highly exposed elements *	
	Very frequent	Not applicable	Not applicable	Not applicable	XD3	

The reference classifications to consider, for cover only, are the following:

* for bridges: cornices, longitudinal anchor beams of retaining devices, coverings of expansion joints

The correspondence by the national annex from classes XF to classes XC or XD is relevant only if the reference classification obtained is more severe than the concomitant classification XC or XD. In the case of an XS classification concomitant with an XF there is no correspondence and it is the XS classification of the origin of the project that serves to determine the cover.

Example of a bridge at the seaside

Determination of the environment classifications is done by faces:

- o for exterior faces:
- XC4 corrosion caused by carbonation, wall subjected to an environment cyclic wet and dry;
- XS1 corrosion caused by chlorides present in sea water, wall exposed to air carrying sea salt, but not in direct contact with sea water.
- Under the waterproofing layer [EC2-2 4.2(105)]:
- XC3
- For interior faces [EC2-2 4.2(104)]:



• XC3 (chosen not to classify the interior of the box girder in XS1, but this is according to particular project).



Fig./Tab.I.(2): Example of a bridge at the seaside

II.2.<u>Cover</u>

At first sight, determination of cover values to adopt for reinforcement seems complicated.

In effect two successive steps must be taken:

- determination of exposure classifications (use of table 4.1 as seen above)
- determination of structural classification [EC2-1-1 4.4.1.2(5)],

The structural classification is defined in a conventional manner for determination of the cover It is based upon the design working life that is characterized by a category defined according to various types of construction [EC0 2.3 Tab.2.1], but also on other factors like for example the classification of the strength of concrete.

The reference structural classification recommended is S4, and corresponds to a design working life of 50 years. The classifications of concrete resistance are at least equal to those given in [EC2-1-1 Anx.E]. It is then modulated according to particular project choices (use of table 4.3N modified by the national annex in 4.3NF below).

Criterion	Exposure classification according to table 4.1						
	X0	XC1	XC2 / XC3	XC4	XD1 / XS1 / XA1 ³⁾	XD2 / XS2 / XA2 ³⁾	XD3 / XS3 / XA3 ³⁾
Design working life	100 years: increased b y 2	100 years: increased b y 2	100 years: increased b y 2	100 years: increased b y 2	100 years: increased b y 2	100 years: increased b y 2	100 years: increased b y 2
	25 years and less: reduced by 1	25 years and less: reduced by 1	25 years and less: reduced by 1	25 years and less: reduced by 1	25 years and less: reduced by 1	25 years and less: reduced by 1	25 years and less: reduced by 1



Resistance

classification 1)

 \geq C30/37

> C50/60

and < C50/60: reduced by Chapter 3 - Major project data

≥ C30/37 and < C50/60: reduced by 1	≥ C30/37 and < C55/67: reduced by 1	\geq C35/45 and < C60/75: reduced by 1	≥ C40/50 and < C60/75: reduced by 1	≥ C40/50 and < C60/75: reduced by 1	≥ C45/55 and < C70/85: reduced by 1
\geq C50/60: reduced by 2	\geq C55/67: reduced by 2	\geq C60/75: reduced by 2	\geq C60/75: reduced by 2	\geq C60/75: reduced by 2	\geq C70/85: reduced by 2
Concrete of	Concrete of	Concrete of			

	reduced by 2	reduced by 2	reduced by 2	reduced by 2	reduced by 2	reduced by 2	reduced by 2
Type of matrix cement		Concrete of class≥ C35/45 base of CEM I without fly ash: reduced by 1	Concrete of class≥ C35/45 base of CEM I without fly ash: reduced by 1	Concrete of class≥ C40/50 base of CEM I without fly ash: reduced by 1			
Compactness of cover ²⁾	Reduced by 1	Reduced by 1	Reduced by 1	Reduced by 1	Reduced by 1	Reduced by 1	Reduced by 1

1) For the sake of simplicity, the resistance class here is an indicator of durability. It may be judicious to adopt, on the basis of more fundamental indicators of durability and of associated threshold values, a specific justification of the structural classification adopted, by referring to the AFGC guide "Design of concretes for a given life of structures", or to standards documents based on the same principles

²⁾ This criterion applies only in the case of elements for which a good compactness of the covers can be guaranteed, namely:

- Formwork face of element plans (easily assimilated to slabs, possibly ribbed), cast horizontally on industrial formworks.
 - Elements industrially prefabricated: elements extruded or drawn, or formwork faces of elements cast into metal frameworks.
- Under face of flagstones of bridge, possibly ribbed, subject to accessibility of bottom of framework with vibration devices.
- ³⁾ For exposure classifications XAi, this correspondence is indicative subject to a justification of the nature of the aggressive agent.

Fig./Tab.I.(3): Modulations of the recommended structural classification, in view of the determination of the minimum cover $c_{min,dur}$ in tables 4.4N and 4.5NF [EC2-1-1/AN Tab. 4.3NF]

- determination of the minimum cover,
- •
- and finally, the taking into account of the execution tolerances [EC2-1-1 4.4.1.3] that allow stipulation of nominal cover, the final value to be specified in the plans.

With use the designer will find that with Eurocode 2 he has at his disposal rules that enable him to be more precise in his choices and that the efforts leading to a better quality are in turn rewarded. This may prove to be economically advantageous in materials particularly where repetitive industrial operations are involved.

On the other hand, direct adoption of the proposed values gives roughly the same results as previous practices. The examples given later illustrate this well.

II.2.1.Determination of minimum and nominal cover

The minimum cover is defined as being the greatest of three values $c_{min,b}$, $c_{min,dur}$ and 10mm.

 $c_{min} = max \{ c_{min,b}; c_{min,dur}; 10mm \}$

For bridges, in practice, the value 10mm does not apply and it is the values $c_{min,b}$, value required regarding adherence and $c_{min,dur}$, value required regarding durability that come into play.

2.1.0.a) *Minimum cover relative to adherence*


 $c_{min,b}$ is the minimum cover necessary to guarantee a good transmission of adherence forces. It is defined by table [EC2-1-1 4.4.1.2 Tab.4.2] and repeated in the two following tables for reinforcement and for prestressing steel.

	c _{min,b} for reinforcing steels		
	Biggest aggregate ≤ 32 mm	Biggest aggregate > 32mm	
Individual reinforcement	φ	φ + 5mm	
Bundles *	$oldsymbol{\phi}$ equivalent	$\phi_{\text{equivalent}} + 5 \text{mm}$	

* For determination of equivalent diameter in the case of bundles of bars, refer to chapter 8 of this guide.

	c _{min,b} for prestressing steels
Tendons in round tube	min { ϕ ; 8 cm}, ϕ diameter of tube
Tendons in flat tube	max {a; $b/2$ }, (a,b) dimensions of tube and $b>a$
Prestressing steel	max {2¢; diameter of biggest aggregate}
	ϕ diameter of strand, smooth wire or deformed wire

Fig./Tab.I.(4): Values of c_{min,b} for reinforcing and prestressing steels

2.1.0.b) *Minimum cover relative to durability*

 $c_{min,dur}$ is the minimum cover [EC2-1-1 4.4.1.2] necessary to guarantee protection of the reinforcing steel against corrosion. It is defined for reinforcement by the non-modified table 4.4N of Eurocode 2 and for prestressing steel by table 4.5N of Eurocode 2 modified by the national annex. Both tables are shown below. $c_{min,dur}$ depends on the structural classification " Sn " and the exposure classifications of the structure faces.

Environmental requirement for c _{min,dur} (mm)							
Exposure classification according to Table 4.1				able 4.1			
classification	X0	XC1	XC2 /	XC4	XD1 /	XD2 /	XD3 / XS3
			XC3		XS1	XS2	
S1	10	10	10	15	20	25	30
S2	10	10	15	20	25	30	35
S3	10	10	20	25	30	35	40
S4	10	15	25	30	35	40	45
S5	15	20	30	35	40	45	50
S 6	20	25	35	40	45	50	55

Fig./Tab.I.(5): Minimum cover values *c*_{min,dur} required regarding durability for reinforcement in reinforced concrete in compliance with EN 10080 [EC2-1-1 Tab.4.4N]

Environmental requirement for c _{min,dur} (mm)							
Structural	Exposure classification according to Table 4.1						
classification	X0	XC1	XC2/XC3	XC4	XD1/XS1	XD2/XS2	XD3/XS3

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S1		10	15	25	30	35	40
S2		15	25	30	35	40	45
S3	obje	20	30	35	40	45	50
S4	Sans	25	35	40	45	50	55
S5	01	30	40	45	50	55	60
S6		35	45	50	55	60	65

Fig./Tab.I.(6): Minimum cover values c_{min,dur} required regarding durability for reinforcement in prestressed [EC2-1-1/AN Tab.4.5NF]

These tables should be read as follows:

- The recommended reference structural classification S4 serves as a start; it corresponds to a design working life of 50 years and has served as a calibration basis of the cover values in the table.
- Table 4.3N of Eurocode 2 modified by the national annex is then used to effect structural classification changes taking account of special project conditions:
- Over-classification of two classifications for the structures whose expected design working life is 100 years,
- Under-classification of two classifications by taking account of concrete durability performances by means of criteria based upon the concrete resistance classification, the type of the matrix cement or the compactness of the cover.
- $\circ~$ Once the final structural classification is obtained, the minimum cover values $c_{min,dur}$ to consider may be taken directly from the two tables 4.4N or 4.5NF shown previously according to the exposure classifications accorded to the case studied.

The application of table 4.4N or 4.5NF, according to whether cover for reinforced concrete or for prestressing steels is in question, leads to a value $c_{min,dur}$ that it is advisable to modulate, where applicable, according to other additional aspects. This is the case where an increased safety margin is required, when stainless steel or additional protection is used [EC2-1-1 4.4.1.2(3) et (6) to (13)]. But generally there is no need to modify $c_{min,dur}$ [EC2-1-1/AN 4.4.1.2].

2.1.0.c)Nominal cover

Once c_{min} is obtained by taking the maximum of the values required regarding adherence $c_{min,b}$ and regarding durability $c_{min,dur}$,: added to this is a margin called the execution range Δc_{dev} to take account of the execution tolerances [EC2-1-1 4.4.1] and to obtain the **nominal cover** that must be specified in the plans:

 $c_{\text{nom}} = c_{\text{min}} + \Delta c_{\text{dev}}$

Eurocode 2 recommends a standard value of 10mm for Δc_{dev} . The national annex confirms this value but sets special conditions that may allow a reduction in this value.

For example:

• if the quality assurance system includes monitoring and measurement of the cover, the value may be reduced to 5mm,





5mm $< \Delta c_{dev} < 10$ mm

• if, where prefabrication is involved, there is a guarantee of accurate measurements and the possibility of rejection in the case of non-compliance, it may be reduced to zero:

0mm $< \Delta c_{dev} < 10$ mm

II.2.2.Example of application

Example of a bridge at the seaside

Reminder of environment classifications previously obtained:

Environment classifications for exterior faces :

- XC4 corrosion caused by carbonation, wall subjected to an environment cyclic wet and dry

- XS1 corrosion caused by chlorides present in the sea water, faces exposed to air carrying sea salt but not in direct contact with sea water

- Environment classifications for interior faces and under the waterproofing layer:
 - XC3 according to [EC2-2 4.2(105)]
- Structural classifications:

Initial structural classification = S4

Modulation of this classification

+ 2 for a design working life of 100 years

+0 or -1 according to the concrete resistance classification used compared to the resistance classification recommended according to the exposure classification

 $+0 \mbox{ or -1}$ $\ \ according$ to the concrete resistance classification which should also be a base of CEM I without fly ash.

-1 for compact cover on the underside of the slab

Hence for exterior faces, underside

4 + 2 (100 years) – 1 (concrete C35/45 MPa for XC4) – 0 (CEM I but resistance insufficient)– 1 (compact cover) = S4

4 + 2 (100 years) - 0 (concrete C35/45 MPa for XS1) - 0 (CEM I but not applicable)- 1 (compact cover) = S5. It is the classification in XS1 that gives the most severe result.

(web faces do not benefit from the reduction for compact cover)

For interior faces other than the underside of upper slabs

4+2~(100~years)-1~(concrete C35/45~MPa for XC3) -1~(CEM~I and sufficient resistance)– 0~(compact~cover) = S4

Nominal cover for reinforcing steels:

Hypothesis of a standard case: the biggest aggregate is less	than or equal to 32 mm, e	t ∮ steel ≤25
	Outside faces, underside	Interior faces (except underside of upper slab)
Environment classifications	XC4 / XS1	XC3
Structural classifications	S 5	S4



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Min cover /adherence $c_{min,b} = \phi_{acier}$ [Fig./Tab.I.(4)]	25 mm	25 mm
Min cover /durability c _{min,dur} [Fig./Tab.I.(5)]	40 mm	25 mm
$\mathbf{c}_{\min} = \max \{\mathbf{c}_{\min,b}, \mathbf{c}_{\min,dur}\}$	40 mm	25 mm
Execution tolerances Δc_{dev} (measurement of cover)	5 mm	5 mm
Nominal cover $c_{nom} = c_{min} + \Delta c_{dev}$	45 mm	30 mm

In practice, an effort will be made to limit the number of cover values.

III.

IV.DATA RELATIVE TO ACTIONS

At the date of publication of this document, a methodological guide to the actions on bridges is being prepared by Sétra. It will be advisable to refer to it for more details.

IV.1. Traffic classification

It is advisable to define the traffic classification to be adopted for the structure according to information from the national annex of Eurocode 1 part 2.

IV.2. Exceptional transport

IV.2.1.Generalities

In compliance with the national annex of Eurocode 1 part 2, informative annex A of this Eurocode relative to special vehicles does not apply.

It is replaced by:

- French regulations on exceptional transport for definition of "standard" special vehicles,

- the "Guide pour la prise en compte des véhicules spéciaux sur les ponts routiers", appended to the national annex of Eurocode 1 part 2 for definition of design rules to consider for the special vehicles moving alone or mixed with normal road traffic.

IV.2.2.Special vehicles taken into account

The individual project might possibly define the special vehicles to be taken into account and clarify their conditions of movement.





For special civil vehicles, the project may:

- either consider "standard" special vehicles defined by French regulations on exceptional transport,
- or define particular special vehicles likely to use the structure.

Similarly, the project might show special military vehicles to be taken into account, for example a convoy of Leclerc armored cars, tanks of 72 or 110 tons etc.

The project specifies the conditions of movement of these vehicles:

- movement at low speed (less than or equal to 5km/h) or at normal speed (in the order of 70km/h),
- special vehicles alone on the structure or mixed with normal traffic,
- number and spacing of special vehicles,
- movement according to an imposed traffic corridor,

- movement possible on emergency stop lane or on a leveled lane (if not signed, the vehicle is assumed to be able to move on the whole width of the pavement),

- frequency of movement on structure.

The characteristic values of the loads associated with the special vehicles are the nominal loads of these vehicles multiplied by 1.1.

IV.2.3. Consideration of special vehicles in combinations of actions

The load group corresponding to special vehicles is called group 5 and noted gr5.

With "LM3" designating the characteristic value of the special vehicle considered in the individual project, "LM3 braking" designating the corresponding braking actions and δ designating the coefficient of dynamic increase of the vehicle, group 5 is defined as follows:

- with no concomitant road traffic: group $5 = gr5 = Q_{LM3} \times \delta + Q_{"LM3 freinage"}$
- with concomitant road traffic: group $5 = gr5 = Q_{LM3} \times \delta + Q_{"LM3 freinage"} + Q_{0,4UDL + 0,75TS}$

The combinations of actions to consider for the special vehicles are the following:

ULS	Basic combination for persistent and transient design situations	$\sum_{j\geq 1} (1,35 G_{kj,sup} + 1,00 G_{kj,inf}) + \gamma_P P + 1,35 gr5$
SLS	Characteristic combination	$\sum_{j\geq l} (G_{kj,sup} + G_{kj,inf}) + P_k + gr5 + 0.6T_k$
SLS	Frequent combination	$\sum_{j\geq 1} (G_{kj,sup} + G_{kj,inf}) + P_k + \psi_1 \operatorname{gr5}$

(with ψ_1 = depending on the frequency of movement of special vehicles likely to use the structure. [EC0 Anx.A2/AN]

IV.3. Thermal gradient



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A complete description will not be given here on Eurocode 1 part 1-5 devoted to the thermal loads of climatic origin, but the designer's attention will be drawn to the major elements that distinguish this text from previous practices and to highlight the special Eurocode 2 rules regarding consideration of the thermal gradient.

In the absence of more accurate information, the coefficient of thermal expansion of concrete may be taken as equal to 10×10^{6} /°C [EC2-1-1 3.1.3(4)].

IV.3.1.Definition of the thermal action

The thermal **action** T_k gives rise to a certain temperature distribution in the section $\Delta T(y)$, and may be broken down into three components:

- A component of **uniform temperature variation**. This leads to an extension or a shortening of the structure's deck, and possibly a axial stress; this component is noted as ΔT_N ,
- •
- A component called **thermal gradient**, that corresponds to a temperature difference between the upper and lower deck fibers. It leads to a curvature of the deck, and possibly also to a bending moment; this component is noted as ΔT_M ,

A transverse linear gradient component may normally exist in a concomitant way with the previous one; it is not explained to simplify the description.

• A component called **balanced**, and noted as ΔT_E . It is the temperature field deducted by subtraction of the two previous fields:

$$\Delta T_{E}(y) = \Delta T(y) - \Delta T_{N} - \Delta T_{M} \cdot \frac{y}{h}$$

where h represents the height of the section , and y is nil on the level of the neutral axis. This balanced component leads to no stress in the structure (even hyperstatic): only auto-stresses are formed; they have as expression (compression counted positively): $\sigma_{TE} = E_{cm} \cdot \alpha_T \cdot \Delta T_E$. By definition, the balanced component Δ T_E is not "linear" in the section; similarly the auto-stresses it creates σ_{TE} . Most of the verification rules stated in the Eurocode materials are thus no longer directly applicable.

The first component ΔT_N is considered in a classical manner and presents no particular difficulty. Its value is determined from the table of the national annex relative to clause 6.1.3.2(1) Eurocode 1 part 1-5. Further, in order to prevent reading errors on the part of the designer, the curves in figure 6.1 are replaced by a table (strictly equivalent).

On the other hand, two methods are suggested for consideration of components ΔT_M and ΔT_E that depend on the type of structure, and the thickness of the cladding.

- In method 1 [EC1-1-5 6.1.4.1], the fixed values of ΔT_M are specified in Eurocode 1 part 1-5; the balanced component is simply ignored and no value is given.
- In **method 2** [EC1-1-5 6.1.4.2], the values of components ΔT_M and ΔT_E are deducted by integration from a temperature profile $\Delta T(y)$ varying in the height of the section and given in the tables. This second method is more laborious, but it gives, in most cases, a component ΔT_M lower than that from method 1.

It is **generally acknowledged** that the balanced component should not be taken into account in section calculations. For concrete bridges its effects are covered by an adapted surface reinforcement.





IV.3.2. Thermal gradient sign

The thermal gradient ΔT_M is normally considered to be positive when the upper fiber of the deck is warmer than the lower fiber of the deck. This case (which corresponds to a direct warming by the sun's rays, during the day) is the only one of the previous practices considered. The hyperstatic moments generated by a positive thermal gradient tend to compress the upper fiber of a beam.

Eurocode1 part 1-5 also requires consideration of a **negative** thermal gradient (of an absolute value lower than the previous one). In this case the lower fiber of the deck is warmer than the upper fiber. This case corresponds to the cooling of the deck during the night (the heat losses are greater on the upper face than on the lower). The hyperstatic moments generated by a negative thermal gradient stretch the upper fiber of a beam.

IV.3.3.

IV.3.4.Thermal force: short-term force

The distinction between components of quick and slow variation, introduced in previous practices, has not been used in the Eurocodes. The integrality of the thermal force should be considered as a short-term force and thus calculated with the tensile modulus E_{cm} . Such a provision may be justified by a creep coefficient calculation as in annex B of Eurocode 2 part 1-1.

For this the slab made up of the upper transverse beam of a reinforced bridge, 16 m wide in total, and 38 cm thick, is considered When this structure is fifty years old (half its design working life), an imposed strain is applied for four months (seasonal temperature variations). A simple calculation of the creep coefficient shows that then:

 $\varphi(50 \text{ years} + 4 \text{ months}, 50 \text{ years}) \approx 0.25$

A more unfavorable value, calculated at the beginning of the design working life, at two years for example from being put into service would be:

 $\varphi(2 \text{ years} + 4 \text{ months}, 2 \text{ years}) \approx 0,47$

Put another way, the creep deformations are low, and the stresses resulting from the thermal actions may be calculated with the value E_{cm} of Young's modulus.

IV.3.5. Consideration of the quasi-permanent combination of actions

It is good to stress that in compliance with annex A2 of Eurocode 0, the thermal action intervenes in **all** combinations of actions of SLS, thus **equally in the quasi-permanent combination** (contrary to the specifications of previous practices). The thermal action at ULS could moreover be ignored [EC2-1-1 2.3.1.2].

It should be noted that in certain cases, not only should the temperature not be ignored at SLS but may even intervene as a basic action (e.g. case of a lowered arch).

IV.3.6.Calculation of stresses due to thermal actions

Only the components ΔT_N and ΔT_M are dealt with here, since it has already been mentioned that the component ΔT_E possibly taken into account created neither an axial force nor a bending moment (**balanced** component).

Generally, calculation of stresses may be done by assuming a linear-elastic performance of the structure [EC2-1-1 5.4(2)] (the geometric characteristics are those of **gross** sections).

Eurocode 2 however offers the possibility of taking account of the state of cracking of the section for the calculation of the effects of the thermal force [EC2-1-1 5.4(3)].





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This leads to a reduction in the bending moments due to the thermal gradient by redistribution but causes serious complications in calculation. Safety may be put first by evaluating the increased stresses, obtained from non-cracked sections.

In normal structures in reinforced concrete in France, up to now the effects of the thermal gradient were traditionally ignored for no particular reason. One could base oneself on the clause cited above to consider the favorable effect of cracking in order to minimize the impact of consideration of the thermal force in these structures.

In the simple and particular case of a bent reinforced concrete structure (no axial stress) with a constant ratio of cracked and gross inertias, the reduction in the bending moment corresponds directly to the reduction of the thermal gradient or the curve deducted from ΔT_M by this same value.

$$\omega = -\frac{\alpha_{\rm T}}{\rm h} \cdot \frac{\rm I_{\rm fiss}}{\rm I_{\rm brute}} \cdot \Delta \rm T_{\rm M}$$

(the curve being considered positively when it leads to a lengthening of the lower fiber).

As an example, one may consider the slab made up of the upper transverse beam of a reinforced bridge and in which the axial stress is ignored. The following data are used:

concrete C35/45, $E_{cm} = 34$ GPa

total width b = 16 m,

total height h = 0.38 m,

distance from the upper fiber to the lower bed of steels d = 0,34 m,

transverse section of reinforcement of the lower bed $A_s = 38,1 \text{ cm}^2/\text{m}$ (2HA20 every 0,165m).

The inertias obtained are as follows:

Gross concrete section (reinforcement ignored): $I_{brute} = 73,16 \times 10^{-3} \text{ m}^4$,

Cracked section (concrete in tension and compressed reinforcement ignored): $I_{fiss} = 25,96 \times 10^{-3} \text{ m}^4$.

Or a factor of 2,8 between the two inertias and a reduction of stresses due to the thermal loads in the same ratio.

V.DIVERSE DATA

V.1. Definition of sections and equivalence coefficients

The various sections to use are not introduced explicitly in Eurocode 2, since "regulation" ideas are not in question. They remain however valid and practical and it is useful to remember their exact definition:

V.1.1.Gross/net section

These first two sections do not make for intervention of reinforcement or of prestressing steel.

• Gross sections: these are concrete sections only, as they come from formwork designs, without deduction of hollows, undercuts and ducts to receive the prestressing steels or their anchors, often used for structural analyses as seen in [IX.2.2].



• Net sections: these are obtained by subtracting from the gross sections the hollows such as holes, undercuts and ducts arranged for the passage or the anchoring of the prestressing steels, even if these hollows are subsequently filled

V.1.2.

V.1.3. Cracked or non-cracked section

The choice between calculations in cracked or non-cracked sections is made case by case, according to the maximum tensile stress in the section resulting from a first calculation in a non-cracked section [EC2-1-1 7.1(2)].

- \circ If $\sigma_{min} > -f_{ct,eff}$, calculations are done on a non-cracked section if the case arises.*
- If not, the calculations are done on a cracked section, i.e. ignoring the tensioned concrete.

The value of $f_{ct,eff}$ to use for the calculation of the stresses may be taken as equal to f_{ctm} or $f_{ctm,fl}$

For application to bridges $f_{ct,eff} = f_{ctm}$ is systematically used.

*In the first calculation the maximum tensile stress that is likely to be applied to the structure at SLS must be considered. In effect, once the section is cracked it no longer has any tensile strength. Hence for example for a frequent SLS verification, even if $0 > \sigma_{\min} > -f_{ctm}$ it is advisable to do the calculation on a cracked section if $\sigma_{\min} < -f_{ctm}$ at the characteristic SLS.

V.1.4.

V.1.5.Homogenous section / homogenous reduced

These last two sections bring in reinforcement and/or prestressing steel **insofar as these latter adhere to the** concrete through the coefficients of equivalence **n** that are the ratios of their strain modulus to the strain modulus of the concrete, or $\frac{E_s}{E_{cm}}$ and $\frac{E_p}{E_{cm}}$. They are distinguished according to whether they are in cracked section or in non-cracked section.

• Homogeneous sections: these come into the calculations in non-cracked sections and are obtained by adding to net sections the section of the reinforcement and/or prestressing steel multiplied by a coefficient of equivalence.

It will be remembered that the non-cracked sections are those sections where the tensile stress does not exceed $-f_{ctm}$ under characteristic SLS combination.

• Reduced homogeneous sections: these come into the calculations as cracked sections for which concrete in tension is ignored; they are homogeneous sections obtained with the only compressed part of the concrete.

The sections are cracked in service when they do not meet the criteria of non-cracked sections (they are generally sections in reinforced concrete and here a section of prestressed concrete is known as partial prestressing).

V.1.6.Sections for calculations of stresses at SLS



Calculation of stresses is done using the following basic hypotheses:

- cross-sections stay level,
- tensioned concrete either resists tension (calculation in non-cracked section) or is ignored (calculation in cracked section).
- the materials undergo no relative slippage (the reinforcement have the same linear variation as the concrete at the same level),
- both the reinforcement and the concrete obey Hooke's law

1.6.0.a) Calculations of stresses in reinforced concrete sections

Since the sections are generally cracked, it is the reduced homogeneous sections that are used for the stress calculations.

Moreover the way to factor for creep is not explained, but the simplified method using an effective concrete strain modulus, E_{c,eff} [Chapter 4-II.1.1II.1.1], is acknowledged. The effects of creep may thus be taken into

account in a set manner by using a coefficient of equivalence n, intermediate between $\frac{E_s}{E_{cm}}$ and $\frac{E_s}{E_{c,eff}}$,

and depending on the proportion between permanent loads and variable loads. One more step in the simplification could involve reverting to previous practices and adopting a fixed value for n.

Generally one could take n = 15 for ordinary concretes and n = 9 for high-performance concretes, (

 $f_{ck} \ge 55 \text{MPa}$), a more accurate calculation being possible. A second "short-term" calculation with $n = \frac{E_s}{E_{cm}}$ may be necessary when the compressive stress of the concrete obtained by the previous calculation is near to the acceptable limit. It is then

idvisable to verify that the acceptable compression is not exceeded in the "short-term" calculation.

1.6.0.b)Calculations of stresses in sections of prestressed concrete

For non-cracked sections in service:

The stresses due to permanent actions are generally calculated in net section.

This method of working may serve for a simple manual calculation in predimensioning. In practice in the case of constructions with stages using elaborate software and consideration of creep by a scientific method, the homogeneous sections come into play when there is an injection of certain prestressing steels that make them adherent relative to the later applications of permanent loads.

The stresses due to variable actions are calculated in a homogeneous section, with a coefficient of equivalence ^{cm} for reinforcing steels and E_p/E_{cm} for prestressing steels E.

For cracked sections in service, the calculation of stresses is done as before for permanent loads and in reduced homogeneous section for variable actions. The stresses in the reinforcing steels and the over-stresses in the prestressing tendons from the permanent state are calculated with a modular ratio $\frac{E_s}{E_{cm}}$ for reinforcing steels

and $\frac{E_p}{E_{cm}}$ for prestressing steels.





The previous practice distinguished the part of over-stresses of prestressing steels going with the return to zero of the adjacent concrete stress, and the part of later over-stress, by giving them different coefficients of equivalence. This distinction on the coefficient of equivalence has no grounds; it is thus not used.

It is moreover recalled that the differences in adherence between reinforcement and prestressing steels may have to be taken into account if appropriate [Chapter 3-I.2].

V.2.

V.3. Design working life

The design working life is the period during which a structure must be used with normal maintenance but with no major repairs. A value of 100 years is expected for concrete bridges whose projects are established using Eurocodes.

It must be borne in mind that the minimum cover, an important aspect of the project, has a value that depends directly on the choice of design working life. It is determined with Eurocode 2 as shown in [Chapter 3-I.2].

In case the client has a special need for a different design working life, the Eurocodes offer no complete solution because they allow a change in the level of reliability in only a small number of variables, as for example the representative value of the actions and the partial factors. Other measures are available nonetheless, outside the scope of Eurocodes, such as quality management, assurance of effective maintenance, etc. Finally, specific measures of prevention or protection favor structure longevity.

V.4.<u>Mean relative humidity</u>

V.5.

In the absence of more accurate data, for open-air structures the following values may be taken for the mean relative humidity, RH, expressed in percentage of relative humidity:

RH = 55 in south-eastern France.

RH = 70 in the rest of France

The simplified method of determination of the coefficient of creep in Eurocode 2 [EC2-1 Fig.3.1] is given for values of RH of 50% and 80%. It is understood that it is allowed to make linear interpolations.

V.6.<u>Other data</u>

Other data relative to the materials and necessary for the calculations are as follows:

 \circ for concrete: the characteristic compressive strength of concrete at 28 days f_{ck} (bearing in mind, if applicable, the minimum previously required) and thus all the other parameters stemming from it f_{cm} , f_{ctm} , E_{cm} etc.(see table 3.1).

It will be remembered that the modulus E_{cm} depends particularly on the type of aggregate. Eurocode 2 gives indicative values on the corrections to be made to E_{cm} [EC2-1-1 3.1.3(2)]. When the value of the modulus has a strong influence on the results of the calculations, it is recommended that tests be carried out.





for reinforcement: high-adherence bars and wire are used and the steel required is characterized by its conventional yield strength and ductility classification. For bridges, Eurocode 2 recommends using only steels of high and very high ductility B and C [EC2-2 3.2.4(101P)]. So in principle steels B500B are generally used, and B500C steels where very high ductility is required: for example for a design aimed at resistance to earthquakes. Nevertheless, the use of class A shear and torsion reinforcement is accepted by the national annex. Further information may be found in standard EN 10080.

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- \circ for prestressing steels: it is advisable to use a high-strength steel with particular emphasis put on its tensile strength f_{pk} and its relaxation classification (classification 2 of relaxation Eurocode 2 [EC2-1 1 3.3.2 (4)P]).
- For cement there is a choice between the three classifications of cement S, N et R on which depend certain data that form part of the calculation of the increase in strength of concrete during the first 28 days (characterized by the coefficient β_{cc}) [EC2-1-1 3.1.2(6)], or again in the calculation of coefficients of creep and shrinkage [EC2-1-1 Anx.B 3.1.4]

For normal projects cement classification N may be used. When design of the structure requires tensioning of the prestressed of a new concrete, it is often necessary to use a concrete of R classification to shorten the waiting time before tensioning.

VI.DIMENSIONING CRITERIA

It is advisable to recall that Eurocode 2 is mainly a structure verification standard with some basic elements for design. The design rules are thus presented in verification form and practically not in dimensioning form.

This point is particularly important as regards dimensioning of the prestress. In effect, several levels of prestressing are possible in a structure, from a minimal partial prestressing to a full prestressing. Eurocode 2 merely sets a minimum level of prestressing according to environmental conditions, then shows how to justify the quantity of reinforcing steels required.

It is plain to see that the **choice of prestressing level is the responsibility of the designer**. It is thus advisable to define for each project the desired level of prestressing; the economic optimum depends upon the type of structure and it is not possible to give general rules.

Previous regulations followed the same logic: 3 prestressing levels were defined with their verification rules. The choice of level for each and every project was left to the designer.

Eurocode 2 has not renewed the definition of discontinued levels: it goes progressively from a partial prestressing to a full prestressing, giving the designer total freedom. In order to limit the choices, the [Chapter 7] gives information on the levels of prestressing possible in service.

The same ideas should be applied to the construction stage: it is advisable to define the acceptable tensions according to the load types and the stages considered. Here too, Eurocode 2 gives certain information that may be adapted according to the project. Reference should be made to [Chapter 7] for a description of these rules.



and creep

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CHAPTER 4 -TIME DEPENDENT DEFORMATIONS OF CONCRETE: SHRINKAGE AND CREEP





The time dependent deformations of concrete due to shrinkage and to creep are to be taken into account in the SLS justifications and generally ignored in the ULS, except when their effects are significant, as for example for the ULS verifications for stability and form, for which the second-order effects have a special importance [EC2-1-1 2.3.2.2].

It may be generally stated that shrinkage and creep are very complex phenomena that still today are not under complete control.; codified models of these phenomena that allow calculations, despite their apparent sophistication, are still far from representing reality. Further, the intensity of these phenomena depends largely on such parameters as ambient humidity, the dimensions of the elements, and the composition of the concrete. So care must be taken with regard to the accuracy of the results of the calculations.

Previous practices disregarded shrinkage and creep for design of sections in reinforced concrete: creep is factored in a set manner by the coefficients of equivalence, and shrinkage by expansion joints and appropriate reinforcement.

On the other hand, the effects of creep and shrinkage on the prestress are important and significantly decrease the prestressing forces initially introduced into the structure. Also, for indeterminate prestressed structures, the deformations due to creep may cause large redistribution of stresses. Further, for structures in prestressed concrete, it is truer to say that the shrinkage and creep effects are taken into account for both the SLS and ULS verifications. More precisely, normally, their effects determined for the SLS verifications could be retained for the continuing ULS verifications.

I.Shrinkage

Shrinkage is a reduction in the volume of non-loaded concrete that starts during its hardening and continues until its definite maturing. Eurocode 2 mainly deals with two kinds of shrinkage according to their origin:: endogenous shrinkage (or shrinkage early in its life) of chemical origin, that starts very early and ends quite quickly after several days and which is due to a reduction in volume of the cement paste during its hydration; desiccation (or exogenous) shrinkage, due to a variation in internal hygrometry, which doesn't practically begin until formwork removal and is a slow, long-lasting process.

It is the total shrinkage deformation that must be taken into account in the calculations. As such, the risk of cracking following deformations produced by thermal and endogenous shrinkage in a young cast concrete in contact with hardened concrete is particularly high to be emphasized by Eurocode 2 [EC2-1-1 3.1.4(6)].

Factoring of shrinkage in structural analysis was mentioned at the beginning of this chapter for determination of stresses.

For calculation of stresses:

- *the effect of shrinkage no longer intervenes in the general case where loads other than shrinkage are applied.*
- Conversely, the effect of shrinkage and delayed shrinkage is a cause of cracking and merits a special study; reference is made to the calculation given in [Appendix VII] of this guide.



and creep



II.CREEP

Creep in concrete is the phenomenon according to which deformation of concrete subjected to a constant load continues to increase with time. Creep also depends upon the factors cited above, on the maturity of the concrete at its first loading and the duration and the intensity of the applied load. The simplifying hypothesis of a linear creep of concrete is normally acceptable providing it limits the compressive stress in the concrete to 0,45 f_{ck} (or 0,45 $f_{ck}(t_0)$ if the concrete is loaded at an age t_0) under a combination of quasi-permanent actions [EC2-1-1 3.1.4(4) et 7.2(3)].

II.1. Taking into account of creep by an approximate method

Where great accuracy is not required Eurocode 2 gives in figure 3.1 [EC2-1-1 Fig3.1] a quick method of estimating the final value of the coefficient of creep $\varphi(\infty,t_0)$ in standard conditions of ambient temperature (-40°C,+40°C) and relative humidity (40%<RH<100%).

 t_0 represents the age of the concrete when the load is applied. In these approximate calculations it is understood that the concrete is sufficiently aged for its characteristics at age 28 days to be used. Interpolations are allowed from the given values.

A final value, a little more accurate, of the coefficient of creep may also be obtained from the laws of creep as a function of time, given in annex B of Eurocode 2 Part 1-1 or for high-performance concrete in annex B in part 2 of Eurocode 2.

The final value of the coefficient of creep is used to take account of the effects of creep in an approximate calculation with a small difference according to whether linear or non-linear analysis is used.

The effect on creep where concrete is subjected to a thermal cure is dealt with in [EC2-1-110.3.1.2]

II.1.1.Linear analysis

The taking into account of creep is done by use of the effective modulus of concrete [EC2-1-1 7.4.3(5)], an equivalent of the differing modulus that is not explicitly used in Eurocode 2 for stress calculations but which is defined for calculations of deformations [EC2-1-1 Expr.(7.20)]:

$$E_{c,eff} = \frac{E_{cm}}{1 + \varphi(\infty, t_0)}$$

Eurocode 2 defines from that an effective coefficient of equivalence by $\alpha_e = \frac{E_s}{E_{c,eff}}$ but does not make use of it in its other specifications.

This method of working is valid at SLS as at ULS. It may be specified that for structures entirely of concrete and built without stages it has no effect on the stresses and affects solely the strain results. Conversely for the

structures built in stages or for composite steel-concrete structures, the stresses are obviously also modified. For the structures built in stages, a calculation with scientific creep will give more accurate results (see later). For composite structures, Eurocode 4 gives values of coefficients of equivalence to use according to the type of force.

The method of factoring creep at SLS in the stress calculation in a cracked section is not detailed in Eurocode 2. In the absence of a more accurate calculation, the coefficients of equivalence, used in the determination of reduced homogeneous sections, may be reused. [Chapter 3-III.1].



II.1.2.Non-linear analysis

A simplified way of factoring creep is using an effective coefficient of creep φ_{ef} [EC2-1-1 5.8.4(2)] and [EC2-1-1 Exp.(5.19)]. This applies only to simple cases and when great accuracy is not required.



It is also possible to define φ_{ef} from total bending moments and M_{Ed} but this requires a repeat and a verification of the stability under the permanent state with $\varphi_{ef} = \varphi(\infty, t_0)$ [EC2-1-1 5.8.4(2) note]

If the ratio of the moments varies in the element or the structure, the ratio for the maximum moment section may be calculated by using an representative mean value [EC2-1-1 5.8.4(3)]. The representative mean value will be favored if it may be easily determined, since it is the deformation of the overall structure that is important.



Fig./Tab.II.(1) : Stress-strain diagrams and creep factor]

Contrainte	Stress
Déformation relative	Strain
Loi sans affinité deformation	Law without strain affinity
Loi avec affinité déformation	Law with strain affinity





The effect of creep is thus considered as taken into account by use of the stress-strain diagram of concrete obtained by multiplying all the strain values by a factor $(1+\varphi_{ef})$ [EC2-1-1 5.8.6(4)].

A non-linear analysis proceeds generally to a ULS verification, which explains the definitions of M_{0Eqp} that is the bending moment from a quasi-permanent combination and M_{0Ed} that is the bending moment of a basic ULS combination. An adaptation of the method to SLS consists of taking a characteristic combination moment for this last value.

II.2. Taking into account of creep by a 'scientific' method

For constructions with stages it is necessary to use elaborate software to obtain good accuracy. Taking account of creep involves an iterative calculation, taking account of the laws of the evolution of creep [EC2-1-1 and EC2-2 Anx.B] and of the properties of concrete with time. It is also the case with studies of stability of form involving the non-linearity of the materials and the second order effects. The sections should be modeled by a sufficient number of fibers and their strains are reconstituted at each moment from those of the fibers while ignoring the fibers in tension. The time dependent deformations of the sections are obtained by one of the methods recommended by Eurocode 2, incremental, superposition, etc.

For bridges in prestressed concrete the scientific method is used from a detailed project because a relatively accurate calculation of losses of prestress is necessary. For this same reason clause $[EC2-1-1\ 2.3.2.2(2)]$ must agree that for ULS the effects of creep are significant and are evaluated under the effect of the probable prestress and the permanent loads (as for an SLS verification) and are retained $[EC2-1-1\ 2.3.2.2\ (3)]$. The follow-up of the ULS verification continues by application



of ad hoc variables. The increase in load of self weight (0,35 G) is considered as an additional overall load that is applied in a normal manner to the static diagram of the finished structure, without it creating further creep.

III. ELEMENTS FOR CALCULATION OF SHRINKAGE AND CREEP VALUES

III.1.<u>'Average radius'</u>

The determination of shrinkage and creep values requires an 'average radius' (notional size) of the structure, noted as h_0 .

 $h_0 = 2A_c/u$, where A_c is the area of the concrete transverse section and u the perimeter of the part exposed to desiccation.

In the case of a slab, the 'average radius' thus defined corresponds more or less to four times the average of the distance the water must travel during its evaporation during hardening of the concrete.

a) Case of a slab or a beam with no sealing: the 'average radius' is approximately equal to the thickness e.

e

b) Case of a slab with sealing: generally the upper slab covered with sealant is not considered in the calculation of the part of the perimeter exposed to desiccation and the 'average radius' is thus approximately equal to twice the thickness: 2e.

This point is, however, taken on a case-by-case basis, according to the time before applying the sealant.

e

(the part in bold is not generally subjected to desiccation)

c) Case of a box girder: The inside of the box girder is not generally taken into account in the calculation of the part of the perimeter exposed to desiccation.

This point is, however, taken on a case-by-case basis, particularly in the case of prefabricated arch stones.



(parts in bold not generally subjected to desiccation)

III.2. Presence of reinforcement

Modification of the deformation values of shrinkage and creep due to reinforcement is not explicitly evoked in Eurocode 2. When great accuracy is not required for certain calculations, it is allowed to ignore this phenomenon putting emphasis on safety. Conversely, if it is desired to take it into account, the precise definition of bonded reinforcement present in the sections and the use of elaborate software is necessary. A median option is to use a set factor by using a reducing coefficient, k, to apply to the shrinkage and creep coefficients, defined by:







$$k_s = \frac{1}{(1+20\rho_s)}$$
 for ordinary concretes,

$$k_s = \frac{1}{(1+12\rho_s)}$$
 for high-performance concretes

 ρ_s being the percentage of bonded reinforcement, ratio of the section of reinforcement to the concrete section.





CHAPTER 5 - PRESTRESS





Chapter 5 - Prestress

With Eurocode 2 the clear-cut distinction the French regulations made between prestressed concrete and reinforced concrete fades, to give birth to a unique text dealing globally with constructions using concrete reinforced with reinforcing or prestressing steel. It may be said there is a kind of continuity going from reinforced concrete where prestress is non-existent to fully prestressed concrete, passing via partial prestressed concrete.

Eurocode 2 restricts itself to the only prestress brought by the tensioning of reinforcement (bars, wire and strands) and the essential of the treatment of the prestress appears in the text of Eurocode 2 part 1-1 in [EC2-1-1 5.10]; other indispensable elements are given in a disseminated way, where it is logical and necessary, in the form of specific rules. It is from this method particularly that the peculiarities that characterize the prestress by pretension, post-tension or external and unbonded are introduced. As such, nothing basic has been modified in the factoring of the prestress by Eurocode 2.

But prestress is above all the prestressing kits that are products of construction. There again, the logic of European harmonization is in play, with the appearance of the ATE, on which it is advisable to say a few words since this is indissociable from the use of Eurocodes.

I.I. A T E OF THE PRESTRESSING KITS

The characteristics of the prestressing steels must be in compliance with standard EN 10138 or, failing this, in an ATE.

The prestressing "kits", from the time they're implemented by their producers, must be marked with a CE and an ATE. The ATEs are issued by an accreditation organization (e.g. Sétra in France, DIBT in Germany, OIB in Austria) on the basis of an ATE guide (ETAG 013). The accreditation is issued after a certain number of tests to verify the aptitude for use of the product.

These kits are then subjected to an "attestation of compliance" to certify the concordance of the product put on the market with that tested in the scope of issue of the ATE. This second step concludes with the CE marking issued by a certification organization (e.g. ASQPE in France..). With its CE marking the product is presumed to meet the essential requirements and may be marketed and used without going through a national regulation compliance verification.

I.1.Information appearing in the ATE

The following information is contained in the ATE:

- Possible usage of system (external prestress, replaceable tendons, etc)
- Range of anchorage (from 3 to 37T15 for example)
- Description of components and dimensions
- Minimum curvature radii authorized according to type of tubing
- Free length to be respected behind anchorage or cylinder floor space
- Straight length to be respected at exit of anchorage
- Coefficient of friction k and μ .
- Spacing of duction supports





- Retraction of anchorage upon tensioning
- Mean minimum strength of concrete to respect upon tensioning $(f_{cm,0})$
- Centre distance and cover of anchorages (depends on f_{cm,0})
- Hooping to place behind anchorage (depending on $f_{cm,0}$) and possible adaptation
- Type of injection
- Recommendations on transport, storage and handling

I.2. <u>Useful information for the application of Eurocode 2</u>

The major information used in the calculations is as follows:

I.2.1. Tensioning forces

The tensioning forces are generally given for information or as recommendations in the ATE (except for instructions to the contrary) since the systems are tested to resist in static, in fatigue and in load transfer on the basis of characteristic forces of the tendons F_{pk} and not the forces shown in the ATE. These should thus be deduced from the standard on prestressing steels (EN 10138) giving F_{pk} and $F_{p0,1k}$ and from the national annex of Eurocode 2 giving coefficients k_1 and k_2 to place before F_{pk} and $F_{p0,1k}$. These latter coefficients are used to define the maximum limits of tensioning forces [III.1].

I.2.2. Coefficient of friction k and μ .

The default values given in Eurocode 2 [EC2-1-1 5.10.5.2(2) and (3)] are generally safe. In certain cases, the ATEs may propose more realistic values stemming from on-site measurements.

It is also recommended to take security values in the case of tendons moving over numerous joints.

I.2.3. Retraction of anchorage

The values to be used in the project are fixed by the ATEs.

I.2.4. Strength of concrete and load transfer

The ATEs supply one or several values of minimum strength of concrete to respect at tensioning, according to the respective positioning of the anchorages. This minimum concrete strength influences the distance between centers, the distances at the edges of the anchorages and the associated hooping. In the case where several values of $f_{cm,0}$ are proposed, it is allowable to interpolate on all these parameters.

I.2.5. Geometric stresses

The designer will find, moreover, all the geometric stresses that the project should respect so that tensionings particularly are easily attainable.

Chapter 5 - Prestress



II.

III.PRESTRESS FORCE

Eurocode 2 uses a slightly different vocabulary from the previous practices, whose principal definitions are worthwhile remembering.

III.1.<u>Maximum prestress force</u>

This is represented by P_{max} : It is the force applied to the active end during tensioning (cylinder force). It must, of course, be limited to prevent plasticizing of the reinforcement [EC2-1-1 5.10.2.1]

 $P_{max} = A_p \times min (k_1 \times f_{pk}; k_2 \times f_{p0,1k})$

A_p being the area of the section of the prestressing steels.

 $k_1 \, \text{and} \, k_2$ having values respectively of 0,80 and 0,90, or the recommended values that are retained by the national annex,

but also to prevent excessive compression of the concrete submitted to loads during application of the prestressing [EC2-1-1 5.10.2.2]

 $\sigma_c \leq 0,6 f_{ck}(t)$

III.2. Initial prestress force

This is represented by $P_{m0}(x)$: It is the prestress force applied to the concrete immediately following tensioning (post-tensioning) or after transfer (pretensioning), after deduction of instantaneous losses. The variation according to the abscissa x is due to friction by post-tensioning and to the establishment of the prestress by pretension in the end zones.

Eurocode 2 sets a limit to this initial force, which is new:

 $P_{m0}(x) \leq A_p \times \min(k_7 \times f_{pk}; k_8 \times f_{p0,1k})$

 k_7 and k_8 having for values defined by the national annex, respectively 0,77 and 0,87, for the prestress by post-tension and 0.8 and 0.9 for prestress by pretension.

III.3. Probable prestress force

This is represented by $P_{m,t}(x)$: the prestress force variable along the reinforcement length and with time, that results from the deduction of instantaneous losses and different losses.

IV.PRESTRESS LOSSES

There is no change of principle in the calculation of prestress losses. The adoption by Eurocode 2 of various laws for the different strains of concrete should lead to only a small variance in the final results of prestress



forces compared to previous practices. Moreover, for high-performance concrete (strength classification greater than C50/60), the laws of loss by relaxation like those of loss by creep and shrinkage stayed the same as before.

IV.1.Instantaneous losses

For prestress by post-tension the instantaneous losses are the losses from blockage of the anchorages, the losses by friction and the losses by elastic deformation of the concrete.

Different from post-tension, the implementation of pretension may already lead to prestress losses that are produced between tensioning of the armature and the prestressing of the element. They are particularly losses due to concrete shrinkage, relaxation of the steel and possibly to a thermal treatment that it is advisable to account for in addition to instantaneous losses as such.

It must be noted that the variables μ and k used in the expression of calculation of frictional losses EC2-1-1 Expr.(5.45)] are not, in a positive way, exactly those used to date, but involve factoring of the same physical phenomena. Their values, supplied in the absence of more accurate values, which would be found in the ATEs) are generally safe [EC2-1-1 5.10.5.2(3) and (4)].

IV.2. Different losses

There are two causes of different losses:

- Reduction in elongation of the reinforcement caused by deformation of the concrete from shrinkage and creep
- Reduction in reinforcement stress due to relaxation

Calculation of different losses by relaxation is done in principle for bridges by using relaxation classification 2 [EC2-1-1 3.3.2(4)] defined by a value of ρ_{1000} at the most equal to 2.5% (this is the equivalent of relaxation classification TBR of the previous regulations) and by use of the expression (3.29).

With the exception of very simple cases (isostatic elements, constructed without stages) for which differing and total losses may be estimated in an approximate way from the expression (5.46) from Eurocode 2, that may be used for predimensioning, it is necessary to use the laws of deformation of shrinkage and creep to do the detailed calculation of different losses.

Where there is no need for great accuracy, for example in the study of transverse bending of a bridge slab, the expression (5.46) may be used.

Eurocode 2 part 1-1 gives the corresponding laws in its annex B; Eurocode 2 part 2 specifies in the modified annex B that for high-performance concretes (of strength classification greater than C50/60) use of the laws given in B.103 gives the best results. Further, this modified annex gives methods for determination of coefficients to be used in the expressions from B.103, bearing in mind a better accuracy when experimental measurements obtained from appropriate shrinkage and creep tests are available.

The various methods allowing evaluation of structural effects caused by the differing performance of concrete are briefly described in annex KK of Eurocode 2, part 2, particularly the equivalent time method.





V.

VI.REPRÉSENTATIVE VALUES OF PRESTRESS

From limitations of the prestress force before and after the tensioning operation Eurocode 2 defines, as previously seen, a probable prestress force $P_{m,t}(x)$ with deductions of losses at time t, but also by increase and reduction of this force, two characteristic values $P_{k,sup}$ and $P_{k,inf}$, commonly called range.

- a lower value: $P_{k,inf} = r_{inf} \times P_{m,t}$
- an upper value: $P_{k,sup} = r_{sup} \times P_{m,t}$

The percentages of increase and reduction are respectively 10% ($r_{sup} = 1.1$; $r_{inf} = 0.9$) for the internal bonded prestress by post-tension and 5% for the external prestress or the prestress by pretension $r_{sup} = 1.05$; $r_{inf} = 0.95$).

When the appropriate measurements are taken as for example the direct measurement of the prestress by pretension, one may use $r_{sup}=r_{inf}=1$.

VI.1. Factoring of prestress at ULS

The probable value $P_{m,t}$ is to be used for ULS justifications except for the verification relative to fatigue which uses a combination of actions similar to a frequent ULS combination.

VI.2. Factoring of prestress at SLS

The characteristic values $P_{k,sup}$ and $P_{k,inf}$ are to be used for the SLS justifications and thus for:

- calculations of stress in the concrete to verify relative to specified limits,
- calculations of stress in reinforcing steels to verify relative to specified limits,
- determination of minimum reinforcement,
- calculations of stress in reinforcing steels for calculations of crack openings.

It may be noted that the range is not used for the calculation of stresses in the prestressing steels.

VI.3. Under construction

In situations under construction the national annexes of parts 1-1 and 2 specify that it is possible to take $r_{sup} = r_{inf} = 1.0$ when special precautions are taken, both at the design level and the execution level, so that the probable prestress Pm is attained in the structure.

In the case of post-tension, these precautions consist particularly of:

- planning, from the design stage, empty tubes so that additional tendons may be installed in case the probable tension is not obtained on-site;

- carrying out measurements of the coefficient of transmission on the first tensioned tendons (preferred test) and on a sufficient number of tendons later tensioned (control test);



- establishing a program of work sufficiently spread out to allow determination and repair of any possibly required corrections.

If the structure is very sensitive to prestress effects, it is advisable to maintain the values $r_{sup} = 1.05$ et $r_{inf} = 0.95$.

VII.PRESTRESS ADHERENCE

prestressing steels do not adhere as well to concrete as do high-adherence reinforcing steels. In the stress calculations for cracked sections, the stress increases in the reinforcing steels would thus be greater than those calculated assuming perfect adherence of the prestressing steels.

This is only a problem for stress calculations in partial prestressed structures (i.e. prestressed structures likely to be significantly cracked at SLS).

Two methods are possible to factor this difference in performance of adherence:

- o do a calculation assuming perfect adherence, then correct the stresses obtained,
- do a stress calculation while reducing "at the source" the contribution of stress increase of the prestressing steels.

These two methods are considered in Eurocode 2, the first being recommended for fatigue calculations [EC2-1-1 6.8.2(2)P], the second for calculation of the minimum SLS reinforcement [EC2-1-1 7.3.2(3)].

The first method has the advantage of simplicity but does not allow a state of stress that meets the general equilibrium conditions of the section.

The following modalities of application are recommended:

- For the fatigue calculations of the partially prestressed structures, the method recommended by Eurocode 2 [EC2-1-1 6.8.2] is applied. An upper boundary to the variation of stresses in reinforcing steels may be obtained by totally ignoring the stress increases Δσ_p in prestressing steels beyond the state of nil deformation of the adjacent concrete.
- For SLS stress calculations in the non-cracked prestressed structures, the stress increases $\Delta \sigma_p$ in the prestressing steels are ignored.

For SLS stress calculations in the partially prestressed structures, Eurocode shows no precise method. The second method may be used, which consists of calculating the equilibrium state of the section while balancing the stress increase $\Delta \sigma_p$ by a coefficient ξ_1 (upper limit at 1.0) – or the equivalent which is to reduce the prestress section A_p by this same coefficient ξ_1 .

There too, a limit greater than the stress variation in the reinforcing steels may be obtained by totally ignoring the stress increases $\Delta \sigma_{p}$ in the prestressing steels.

Calculation examples are given in the SLS and ULS chapters on fatigue (application to a PSIDP slab calculated at partial prestress).





Chapter 5 - Prestress

The use of more complex calculation models, adequately representing the differences in adherence between steels is possible. Refer, for example, to the article by F. Toutlemonde and R. Pascu, in the "Bulletin des Laboratoires des Ponts et Chaussées", no. 241, November-December 2002.

For external prestress the stress increase is caused by the average elongation value between two deviators, or on a longer length in the case of slippage on these deviators. The stress increase is thus low and because of the possibility of slippage its estimation is uncertain. For SLS, the stress increase is thus totally ignored. For ULS, the Eurocode allows a fixed factoring of the stress increase [I.1].

Chapter 6 - Justifications at ULS



CHAPTER 6 -JUSTIFICATIONS AT ULS





This chapter deals with ULS justifications, particularly ULS strength of bending, shear stress, torsion and punching.

The ULS justifications for specifics such as fatigue, brittle failure and buckling are also presented.

It is, however, necessary to refer to plate calculations [Chapter 10-IV] to find the elements concerning the combination of longitudinal bending effects and local effects.

The calculation of stresses is generally carried out on the basis of a linear elastic analysis with no redistribution (except for the study of form stability), taking account of the characteristics of the gross sections [Chapter 1-VI.2.2] and if the case arises of the participating width [EC2-1-1 5.3.2.1 and Chapter 1-VI.2.1].

The only difference compared to past practices is in the taking into account of the external prestress.

The Eurocode allows in effect the taking into account of the stress increases in the unbonded tendons. The increase in stress in the reinforcement of unbonded prestress (e.g. for external prestress) may be evaluated:

- in the absence of accurate calculation, taking account of a set increase of **100 MPa** [EC2-1-1 and EC2-1-1/AN 5.10.8(2)],
- if not, in taking account of the overall deformation of the element [EC2-2 5.10.8(103)], by means of a geometric non-linear analysis (second-order, [EC2-1-1 5.7(1)]).

It is generally more unfavorable to ignore the effect of this increase in stress.

When it is useful to evaluate this increase, one may consider a totally slippery tendon (particularly at the same level as the deviators) on the identical route of the actual tendon.

I.JUSTIFICATION RELATIVE TO BENDING

This paragraph refers to clauses [EC2-1-1 and EC2-2 6.1]. The deflected bending is dealt with in [EC2-1-1 5.8.9].

Generally the ULS bending justifications in Eurocode 2 are very close to previous practice:

- calculation of stresses by a linear-elastic model

- verification of sections with material laws (shown below) different than those used for the structural analysis (already shown in chapter 2 VI.3.1)

This paragraph highlights the major differences between Eurocode 2 and the previous regulations. Material laws, taking account of the prestress in the analysis, and verification of the sections are shown here.

I.1. Material laws used for verifications of sections in bending

Calculation of the sections is based on the use of special stress-strain diagrams shown in section 3 of Eurocode 2 part 1-1 [EC2-1-1 3.1.7], [EC2-1-1 3.2.7] and [EC2-1-1 3.3.6].

I.1.1.Concrete

Several laws of behavior may be used:

• The "parabola-rectangle" law [EC2-1-1 Fig.3.3]:



 $\sigma_{c} = f_{cd}$



$$\sigma_{c} = f_{cd} \left[1 - \left(1 - \frac{\varepsilon_{c}}{\varepsilon_{c2}} \right)^{n} \right]$$

for $0 \le \varepsilon_c \le \varepsilon_{c2}$

for $\varepsilon_{c2} < \varepsilon_c \leq \varepsilon_{cu2}$

This law is not in fact made up of a parabola for concretes with strengths greater than 50 MPa, called BHP, since the exponent is not then between 1.75 for C55/67 and 1.4 for C90/105.



Fig./Tab.I.(1): Law of concrete performance for calculation of sections

Two other simple laws are acceptable and considered as equivalent to the "parabola-rectangle" law:

- o the "bi-linear" law [EC2-1-1 3.1.7 (3) Fig.3.4]
- 0





• the simplified rectangular diagram [EC2-1-1 3.1.7 (3) Fig.3.5]:

The hypothesis of a rectangular diagram of compression in concrete is possible, as much for normal concretes as for BHP (*bétons à haute performance*, high performace concrete). The effective depth of the compressed zone and the effective strength are thus a function of the concrete's characteristic strength.

These three laws have been calibrated to give very close results, indeed more conservative for the simplified diagrams.



All these laws are characterized by use of the parameter f_{cd} , design value of concrete compressive strength defined by:

$$f_{cd} = \alpha_{cc} \times \frac{f_{ck}}{\gamma_{c}}$$

with α_{cc} coefficient taking account of the effects of the length of application of the load on the compressive strength of concrete, a coefficient whose value was fixed by the national annex at $\alpha_{cc} = 1.0$

 γ_C partial factor of concrete [EC2-1-1/AN 2.4.2.4(2)] with a value of 1.5 in persistent and transient situations and 1.2 in an accidental situation.

The coefficient α_{cc} allows taking account of the importance of the permanent actions in the combinations. The national annexes of Eurocode 2 have retained the value $\alpha_{cc} = 1.0$ for all concrete structures.

The values of the strain parameters ϵ_{ci} a function of the resistance classification of concrete are to be found in [EC2-1-1 Tab.3.1].

I.1.2.Reinforcing steels

The following laws of behavior may be used [EC2-1-1 Fig.3.8]:

- The bi-linear law of horizontal landings, for which there is no limit of steel strains;
- The bi-linear law with strengthening.

The design strength of reinforcing steels is given by:

$$f_{yd} = \frac{f_{yk}}{\gamma_S}$$

with γ_s partial factor of the steel of reinforced concrete [EC2-1-1/AN 2.4.2.4(2)] of a value 1.15 in persistent and transient situations and 1.0 in accidental situations.









Fig./Tab.I.(3): Stress-strain diagrams of steels of reinforced concrete for calculation of sections

I.1.3. Prestressing steels

Several behavioral laws may be used:

- The horizontal landing bi-linear law [EC2-1-1 3.3.6 Fig.3.10],
- The bi-linear law with strengthening [EC2-1-1 3.3.6 Fig.3.10],
- A law representing the actual diagram of the steels, subject to it applying the coefficient $1/\gamma_s$ over $f_{p0,1k}$) [EC2-1-1 3.3.6(7)]. For the strands, the law of the past regulation (in compliance with figure [EC2-1-1 3.3.6 fig.3.9] may thus be used: it is thus mentioned in the following figure.
- 0





The design strength of prestressing steels is given by:

$$f_{pd} = \frac{f_{p0,01k}}{\gamma_S}$$

with γ_s partial factor of the prestressing steel [EC2-1-1/AN 2.4.2.4(2)] with value of 1.15 in persistent and transient situations and 1.0 in accidental situations.





I.2. Calculation of sections

The calculation of sections at ULS is based on the following hypotheses [EC2-1-1 6.1(2)]:

- cross-sections remain on same level
- tensile strength of concrete is ignored
- there is no relative slippage of materials except for unbonded prestress
- The pivot principle is retained:
- - pivot A (when available): strain limitation in reinforcing steels and/or prestressing steels
- - pivot B: strain limit of concrete for bent parts
- - pivot C: strain limit of concrete in pure compression.
- •



C - concrete pure compression strain limit

Fig./Tab.I.(5): Diagram of possible strain distribution in the ultimate limit state

It must be assured that the acceptable ultimate strains are not attained, if the case arises.

Moreover, for the compressed chords of the box girders (load relatively centered verifying $\frac{e}{h} < 0,1$), it must be

verified that the <u>average</u> strain in compression in the chord is less than ε_{c2} (or ε_{c3} according to the diagram used) – [EC2-1-1 6.1(5)].

This clause may be important for dimensioning of highly stressed parts (lower slabs on supports). It may be necessary to limit the strain of the furthest fiber to a value less than $\varepsilon_{cu2(ou 3)}$ to be able to meet this condition.

I.3. Conclusion





The principle of calculation of sections at ULS is in compliance with the calculation methods previously used in France, with a slight reduction in quantities for the following reasons:

- $\alpha_{cc} = 1.0$

- possibility of using a bi-linear diagram inclined for steels, which allows a reduction of a small percentage of reinforcement.

It seems possible to gain 5 to 8%, but actually the gain is less due to the limitation imposed on the concrete.

- possibility of taking account of the stress increase of the tendons of the external prestress.

On the other hand, removal of pivot A when the horizontal landing diagrams are used for concrete reinforcement and for prestressing steel, does not generally allow a reduction in the quantities of steels, since in this case it is the pivot B that imposes the limits.

II.JUSTIFICATION RELATIVE TO SHEAR

This paragraph concerns the study of shear and torsion strength at ULS. It does not deal with the combination of shear and local bending stresses that is dealt with in [Chapter 10-IV.3]. Similarly, it is necessary to refer to [Chapter 7-II.5] to find the verification of the sections relative to shear at SLS.

It is completed by the treatment of certain special cases in [Chapter 10-I] and numerical applications in [Appendix III].

II.1.<u>Design value of shear force</u>

II.1.1.Definition of shear force to be considered

The design value of the applied shear force V_{Ed} , used in justification of the sections is the sum of:

- \circ the shear force due to exterior sources,
- o the shear force due to prestress,
- o components of the shear force in the case of variable height due to the Résal effect [Chapter 10-I.1].

II.1.2.Design web width b_w

 $b_{\rm w}$ is the smallest width of the cross-section in the zone under tension or in the zone between the chord under tension and the compressed chord.

With injected prestress metallic ducts of diameter $\phi > \frac{b_w}{8}$, the shear resistance $V_{Rd,max}$ should be calculated using a reduced nominal web width:

 $b_{w,nom} = b_w - 0.5 \sum \phi$ with $\sum \phi =$ space for ducts at most unfavorable level.

In the case of non-injected ducts, injected plastic ducts or unbonded prestressing steels:





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$$b_{w,nom} = b_w - 1,2\sum \phi$$

Eurocode 2 part 1-1 specifies that in this latter case, if the adapted transverse reinforcement are planned to prevent splitting of the struts, the coefficient 1.2 may be reduced to 1.0 [EC2-1-1 6.2.3(6)]; this is confirmed by the national annex.

Subsequently the expressions show up only b_w ; it should be read as $b_{w,nom}$ where appropriate.

II.1.3. Principle of verification

These verifications at ULS are defined in [EC2-1-1 6.2 and EC2-2 6.2].

The general verification procedure includes [EC2-1-1 6.2.1]:

- a verification of the strength of the section **without** shear reinforcement. If this is conclusive, only the minimum reinforcement ratio [EC2-1-1 9.2.2] noted in [Chapter 9-III.1] is to be implemented and the verification below is not to be done; it may however be omitted in the case of slabs (plain, ribbed or honeycombed) when a transverse load redistribution is possible [EC2-1-1 6.2.1(4)].
- \circ a verification of the strength with shear reinforcement that includes a verification of both the strength of the concrete struts and of the strength of the shear stress reinforcement;
- verification of the additional longitudinal tensile stress that should be involved in the dimensioning of the longitudinal reinforcement.

The general rules decreed for the justification of the sections apply when loads are applied to the upper parts of the elements.

When the loads are applied to the lower parts, vertical reinforcement should be added that are sufficient to transmit the load to the upper parts [EC2-1-1 6.2.1(9)].

These conditions may obtain when beams cross.

They also appear in box bridges:

- the weight of the lower slab exerts a tension directly on the bottom of the webs.

- for box girders of variable height with curved slabs, the axial stress N_i resulting from the whole of the axial stresses on it, causes a radial force $Q = N_i/R$ where R is the radius of curvature of the slab. Where N_i is a tension Q exerts tension on the webs.

- the prestressing tendons in the lower slab or at the bottom of the webs, also cause a radial force $Q_P = \Sigma F_P / R$ to the bottom.



Chapter 6 - Justifications at ULS







Fig./Tab.II.(1): Radial force of a slab

II.2. Verification of standard sections

The first thing to verify is that the elements without shear reinforcement have sufficient strength. Elements without shear reinforcement are those elements with only a minimum reinforcement as shown in [Chapter 9-III.1].

II.2.1.Members not requiring design shear reinforcement

2.1.0.a)General case

It is advisable to check:

 $V_{Ed} \leq V_{Rd,c}$

 V_{Ed} is the design value of the applied shear force

 V_{Rdc} is the design value for the shear resisistance of the element in the absence of transverse reinforcement:

$$\mathbf{V}_{\text{Rd,c}} = \left[\mathbf{C}_{\text{Rd,c}} \ \mathbf{k} \left(100 \ \rho_1 \ \mathbf{f}_{\text{ck}} \right)^{\frac{1}{3}} + \mathbf{k}_1 \ \boldsymbol{\sigma}_{\text{cp}} \right] \mathbf{b}_{\text{w}} \ \mathbf{d} \qquad [\text{EC2-1-1 } 6.2.2 \ \text{Expr.}(6.2.a)]$$

with a minimum value

$$V_{Rd,c} \ge (v_{min} + k_1 \sigma_{cp}) b_w d$$
 [EC2-1-1 6.2.2 Expr.(6.2.b)]

where

 $k = 1 + \sqrt{\frac{200}{d}} \le 2,0$ with d expressed in mm

$$\rho_1 \ = A_{sl}/b_{w} d \ \le 0{,}02$$

The values adopted are as follows:

- $C_{Rd,c} = 0.18/\gamma_C = 0.18/1.5 = 0.12$
- k₁=0.15
- $v_{min} = 0.34 \gamma_c f_{ck}^{1/2}$ for slabs with a transverse redistribution effect in the case of considered loads •
- $v_{min} = 0.053/\gamma_C k^{3/2} f_{ck}^{1/2}$ for beams and slabs other than those above
- $v_{min} = 0.35 / \gamma_C f_{ck}^{1/2}$ for shear walls

These are the values recommended except those concerning v_{min} in the case of slabs with a transverse redistribution effect and the shear walls, which are values proposed by the national annex.




The expression (6.2a) brings into play the ratio of longitudinal reinforcement $\rho_1 = A_{sl}/b_w d$ in which may be included the bonded prestressing steels. This ratio is particularly planned for rectangular sections and has little significance for box girders, for which it is preferable to ignore it and so retain only the resistant stress given by the expression (6.2b).

The expressions (6.2a) and (6.2b) differ only in the first term of the sum in parentheses. Comparison of these two terms shows that for slabs with a transverse redistribution effect, the expression (6.2b) is always preponderant.

The values of v_{min} specified in the national annex bring up the following comments:

The partial factor γ_{C} is brought to bear which allows treatment of the cases of accidental situations.

The specified value for the slabs with a transverse redistribution effect give shear resistance much greater than for beams with equal dimensions.

2.1.0.b)Special cases

The special case of the non-cracked elements at ULS is examined in [Chapter 10-I.4], that of prestressed members with only one span in [Chapter 10-I.5].

II.2.2. Members requiring design shear reinforcement

This verification is based on a truss model as shown below [EC2-1-1 6.2.3]:



Fig./Tab.II.(2):	Truss model	of shear stress
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Armatures	Reinforcement
Armatures longitudinales	Longitudinal reinforcement
Bielles	Struts

This requires:

- verification of strength of struts
- verification or determination of shear reinforcement





Eurocode 2 shows both the case of perpendicular reinforcement with average fiber ($\alpha = 90^{\circ}$) for which the expressions are simplified and that of reinforcement inclined at a given angle α . Then the expressions with a given angle α are explained; their simplification is easy and quick.

When it is wished to minimize the shear stress reinforcement, the choice is the smallest inclination of the struts compatible with their compressive strength. This may however lead to a large increase in the longitudinal steels. Further, if the chosen direction of the struts ULS is too far from the elastic direction of the major compressive stresses at ULS, large cracks may form under the service shear stress, along with fatigue problems.

In the case of elements of reinforced concrete bridges, it is thus recommended to not incline the struts too much at ULS, so as not to create excessive cracking problems at SLS. It may be possible to limit the inclination down to 34° (cot $34^{\circ} = 1.5$), in compliance with the national annex [EC2-1-1/AN 7.3.1(10) and EC2-2/AN 6.8.1(102)].

In the case of prestressed concrete elements, where there are only a few extra longitudinal reinforcing steels, it is generally advantageous to incline the struts as much as possible.

It should finally be mentioned that the diagram shown in the figure above is only valid for a standard part of the beam. Near the supports, a special study of the end strut should be carried out. The choice of a strut too inclined may necessitate anchoring large quantities of longitudinal steels, and it may be advisable to reduce the inclination of the last struts.

2.2.0.a) Verification of strength of struts

First will be chosen the angle of inclination of the struts defined by the following conditions, fixed by the national annex: [EC2-1-1/AN 6.2.3 (2)]:

• in compression or simple bending $1 \le \cot\theta \le 2,5$ (in compliance with recommended values) or

 $21.8^\circ \le \theta \le 45^\circ$

• in tension $\sqrt{1 + \sigma_{ct} / f_{ctm}} \le \cot\theta \le 2.5 \sqrt{1 + \sigma_{ct} / f_{ctm}}$

where σ_{ct} is the tensile stress at the center of gravity (-f_{ctm} < \sigma_{ct} < 0)

The case of a section where $\sigma_{ct} < -f_{ctm}$ *is not dealt with.*

It is advisable to verify:

 $V_{Ed} \leq V_{Rd,max}$

 V_{Ed} is the design value of the applied shear force $V_{Rd,max}$ is the resistant stress of the concrete strut given by:

 $V_{Rd,max} = \alpha_{cw} b_{w} z v_{1} f_{cd} (\cot\theta + \cot\alpha) / (1 + \cot^{2}\theta)$

[EC2-1-1 Expr.(6.14)]

 b_w is the web width (net width to be taken if the case arises of prestressing steel shafts).

 v_1 is a reduction factor of the resistance of cracked concrete to shear stress. Its recommended value is taken as equal to v [EC2-1-1 6.2.2(6)]; it is to be seen in the national annex and is given by:

 $v_1 = v = 0.6 (1 - f_{ck}/250)$ [EC2-1-1 Expr.(6.6N)]

It may be stated that a high resistance f_{ck} reduces v.

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 α_{cw} is a coefficient taking into account the state of stress in the compressed chord. The recommended values, validated by the national annex, are as follows:

1	for non-prestressed structures	
$(1+\sigma_{cp}\!/f_{cd})$	for $0 < \sigma_{cp} \leq 0,25~f_{cd}$	[EC2-1-1 Expr.(6.11.aN)]
1.25	for 0.25 $f_{cd} < \sigma_{cp} \leq 0.5 ~f_{cd}$	[EC2-1-1 Expr.(6.11.bN)]
$2.5 (1 - \sigma_{cp}/f_{cd})$	for 0.5 $f_{cd} < \sigma_{cp} < 1.0 f_{cd}$ [EC2-1	-1 Expr.(6.11.cN)]

The figure below represents α_{cw} according to axial stress:



Fig./Tab.II.(3): Variation of α_{cw} , with the mean compressive axial stress of concrete

The national annex specifies that in the case of elements in bending combined with a tensile stress, but with one chord in compression, one might take:

 $\alpha_{\text{cw,t}} = 1{+}\sigma_{\text{ct}}{/}f_{\text{ctm}}$

The case of an element in total tension is however not dealt with.

For determination of α_{cw} , σ_{cp} is the mean compressive stress (>0), σ_{ct} is the mean tensile stress (<0), each being determined under the axial design stress on the concrete section, taking account of the reinforcement.

More precisely, the axial stresses may be determined by a conventional elastic calculation although the calculation is taken at ULS. Thus we have $\sigma_{cp} = N/S$ (stress at center of gravity of complete section). For non- prestressed sections, it is found that $\sigma_{cp} = 0$ and $\alpha_{cw} = 1$.

To take reinforcement into account, the values of the modular ratio n given in [Chapter 3-II.1.4] are used. In the case of highly compressed sections, it is safer to ignore the involvement of steels. In the case of little-compressed sections, a variation of 15% of the section (obtained for a ration of longitudinal reinforcement of 1%), involves a difference of only about 3% on α_{cw} .

It is specified that the value of σ_{cp} has not to be calculated at a distance less than $0.5d \times \cot\theta$ from the bare part of the support.

If the design value of the applied shear force V_{Ed} is greater than $V_{Rd,max}$ calculated above, the strut has insufficient resistance. It may be too inclined in relation to the vertical and/or the thickness of the webs is insufficient. The strut must thus be straightened. When this reaches 45°, and if the strength is still insufficient, the web thickness must be increased.

However, in certain cases, the resistance of the struts may be increased [Chapter 10-I.6].

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It may be worthwhile expressing the resistance of the concrete struts by a resistant shear stress from the expression (6.14) of Eurocode 2 part 1-1. One thus obtains:

$$\tau_{\rm Rd,max} = \frac{V_{\rm Rd,max}}{b_{\rm w}z} = \alpha_{\rm cw} \nu_1 f_{\rm cd} \left(\cot\theta + \cos\alpha\right) / (1 + \cot^2\theta) = \alpha_{\rm cw} \nu_1 f_{\rm cd} \frac{\sin 2\theta}{2} \left[1 + \cot\alpha \frac{1 - \cos 2\theta}{\sin 2\theta}\right]$$

and in the case of reinforcement perpendicular to the centroidal axis

$$\tau_{\rm Rd,max} = \alpha_{\rm cw} \nu_1 f_{\rm cd} \, \frac{\sin 2\theta}{2}$$

2.2.0.b) Verification of the resistance of transverse shear reinforcement

The resistance due to the reinforcement is given by:

$$V_{Rd,s} = \frac{A_{sw}}{s} z f_{ywd} (\cot \theta + \cot \alpha) \sin \alpha \qquad [EC2-1-1 Expr.(6.13)]$$

The determination of the reinforcement is done by equaling this resistance to the design value of the applied shear force V_{Ed} :

$$A_{sw}/s = \frac{V_{Ed}}{z f_{ywd} (\cot \theta + \cot \alpha) \sin \alpha}$$

and if the reinforcement steels are perpendicular to the the centroidal axis:

$$A_{sw}/s = \frac{V_{Ed}}{z f_{ywd} \cot \theta}$$

The effective section of the reinforcement steels reaches a maximum at the value given by the following expression:

$$\frac{A_{sw, max} \cdot f_{ywd}}{b_{w} \cdot s} \le \frac{1}{2} \alpha_{cw} \cdot \nu_{1} \cdot f_{cd} / \sin \alpha \qquad [EC2-1-1 \ Expr.(6.15)]$$

This expression is to be used by taking into account the values of f_{cd} and f_{ywd} obtained in persistent and transient situations. It does not apply to accidental situations.

It translates the equality of $V_{Rd,max}$ and $V_{Rd,s}$ when the resistance of the struts is reached for their maximum inclination angle of 45° or $\cot \theta = 1$, *i.e.* when V_{Ed} totally exhausts the concrete resistance of the section.

In the case where there is no discontinuity of V_{Ed} (uniform loading for example), determination of the shear reinforcement on a basic length $1 = z (\cot \theta + \cot \alpha)$ may be carried out by considering the smallest value of V_{Ed} on this length [EC2-1-1 6.2.3(5)].

This provision involves moving the envelope curve of the shear stress reinforcement sections towards the supports.

2.2.0.c) Case of structures built by prefabricated segments with unbonded prestress.

This case is dealt with in [EC2-2 6.2.3(109)].





In this case, the opening of the joint in the tensioned chords reduces the height of the compressed concrete that allows transmission of the struts. The shear reinforcement section should take account of this reduced height.



A – Theoretical tie axes

B - Theoretical compressive strut axes

C – Tensioned chords of truss (exterior reinforcement)

D – Field A: arrangement of frames or stirrup straps with θ_{max} (cot $\theta = 1.0$)

E - Field B: arrangement of frames or stirrup straps with θ_{min} (cot $\theta = 2.5$)

Fig./Tab.II.(4): Inclined fields of stresses in the web across the joint.

The reduced height h_{red} is the height of the compressed concrete, calculated during verification of the section in bending at ULS.

It must be verified that $h_{red} > 0.5h$. If not, the prestress must be increased to recompress the joints.

The angle of inclination of the struts is deduced from the value of h_{red} by the following expression:

$$h_{red} = \frac{V_{Ed}}{b_w \nu f_{cd}} (\cot \theta + \tan \theta) \qquad [EC2-2 Expr.(6.103)]$$

The section of the shear reinforcement is then given by:

$$\frac{W}{h} = \frac{V_{Ed}}{h_{red} f_{vwd} \cot \theta}$$
 [EC2-2 Expr.(6.104)]

It is observed that these expressions are deduced from those of the calculation of the strength of current sections by replacing z by h_{red} .

II.2.3. Additional tensile stress in longitudinal reinforcement

2.3.0.a) Stress calculation

The inclination of the truss struts causes an additional tensile force in the chords:

$$\Delta F_{td} = 0.5 V_{Ed} (\cot \theta - \cot \alpha) \qquad [EC2-1-1 Expr.(6.18)]$$

Moreover, its taking into account should be such that $(M_{Ed}/z + \Delta F_{td}) \leq M_{Ed,max}/z$ [EC2-1-1 6.2.3(7)] [EC2-2 6.2.3(107)].





For elements with a shear reinforcement, this force ΔF_{td} may be obtained by a discrepancy in the moments curve of:

 $a_1 = z (\cot \theta - \cot \alpha)/2$

[EC2-1-1 Expr.(9.2)]

With: θ inclination of shear struts

 α inclination of reinforcement steels along beam's longitudinal axis

z lever arm of elastic couple.

It is easily demonstrated that the expression giving the discrepancy corresponds to the hypothesis of a multiple truss (which is true enough for bridge structures in which the elements are very high in relation to the spacing of the transverse reinforcement). For a simple truss and non-inclined reinforcement the discrepancy is $zcot\theta$.

This arrangement is represented by the figure [EC2-1-1 Fig.9.2] reproduced below.



A- Envelope of $M_{Ed}/z + N_{Ed}$ B - acting shear force F_s C - resistant shear force F_{Rs}

Fig./Tab.II.(5): Illustration of discrepancy rule

However, where the applied shear forces are such that there is no need for shear reinforcement, it is advisable to take:

 $a_1 = d$ with d = effective depth [EC2-1-1 6.2.2(5)]

2.3.0.b)Renewal of stress

In the compressed chord, it is conceivable to renew this force by decompression, insofar as the chord remains compressed at ULS.

This possibility is foreseen in [EC2-1-1 6.3.2(3)] for torsion, but is also applicable in the case of shear force.





In the tensioned chord, the tensile stress is balanced by the longitudinal reinforcement (reinforcing steels and possibly bonded prestress) [EC2-1-1 6.2.3(7) and EC2-2 6.2.3(107) note]. The tensile stress in the concrete reinforcement should remain at less than the stress limit defined by [EC2-1-1 3.2.7] and the total tension of the prestressing steel should remain at less than the stress limit defined by [EC2-1-1 3.3.6].

For the taking into account of the bonded prestressing steels in the case where they are inclined, Eurocode 2 proposes a truss system explained by the figure reproduced below [EC2-2 Fig.6.102N].



Fig./TabII.(6): Superposition of resistance models for the shear force [EC2-2 Fig.6.102N]

This type of diagram is particularly useful for study of the role of the prestress in the equilibrium of the end strut. The previous French regulations explained this point in more complete detail, by taking into account the actual route of each tendon.

In the current part on the other hand, since the angular differences between longitudinal reinforcing steels and prestressing steels are small, it may generally be sufficient to use the drawing of the classic strut without distinguishing the diagrams of the longitudinal reinforcing steels and the prestressing steel.

Nota: The resistance of the reinforcement must also take account of the concomitant torsional stresses. The prestress bonded reinforcement may contribute to the resistance but in any case their total stress increase is limited to $\Delta \sigma_p = 500$ MPa [EC2-2 6.3.2(103)]. This limit is rarely attained in practice. On the other hand, the stress increase should be determined from the permanent state.

II.3.<u>Resistance to shear force near to supports</u>

II.3.1.Current case

Eurocode shows that for the elements subjected to predominantly uniformly distributed loads, the verifications of the shear force are unnecessary for sections situated at a distance less than d (effective depth) of the face of the support [EC2-1-1 6.2.1(8)]. Any shear reinforcement required at this distance should however continue to the support. It is advisable to verify that the shear at the support does not exceed $V_{Rd,max}$ [Chapter 6-II.2.22.1.0.b].

For concrete bridges, the effects of the permanent loads are generally greater than those of the live loads, and may thus be considered as mainly subjected to distributed loads.





II.3.2. Reduction of concentrated loads near supports

The taking into account of concentrated loads near supports follows the rules [EC2-1-1 6.2.2(6) and 6.2.3(8)].

The loads are those on the upper face of the element, at a distance a_v between 0.5d et 2d inclusive from the face of the support if this is rigid, or from the center of the support if this is flexible. Elastomeric bearings and pot bearings come in the second category.

The design value of the applied shear force V_{Ed} may be reduced by a factor $\beta = a_v / 2d$. If $a_v < 0.5d$, it is advisable to adopt $a_v = 0.5d$. A force is thus obtained, V_{Ed} reduced (later called $V_{Ed,r}$).

This is only valid however if the necessary longitudinal reinforcement are totally anchored at the level of the support.

3.2.0.a)Case where shear reinforcement are not required

The value $V_{Ed,r}$ calculated as above should be compared to the shear resistance $V_{Rd,c}$ given by the expressions (6.2a) and (6.2b) Eurocode 2 part 1-1.

If $V_{Ed,r}$ is less than $V_{Rd,c}$, reinforcement are not required.

Moreover the **non-reduced** shear must verify the following condition:

$$V_{Ed} \le 0.5 b_w dv f_{cd}$$
 [EC2-1-1 Expr.(6.5)]

where v is a reduction coefficient of the resistance of cracked concrete to the shear force given by

$$v=0.6(1-f_{ck}/250)$$
 [EC2-1-1 Expr.(6.6N)]

3.2.0.b) Case where the shear reinforcement are required

If the reduced shear force $V_{Ed,r}$ is greater than the values of $V_{Rd,c}$, shear reinforcement are required.





To verify:

 $V_{Ed,r} \le A_{sw} f_{ywd} \sin \alpha$ [EC2-1-1 Expr.(6.19)]

where A_{sw} is the section of shear reinforcement situated on a length $0,75a_v$ centered on a_v , as shown in the figure above.



The resistance of the struts should be verified according to the general expression (6.14) of Eurocode 2 part 1-1 (see above) by taking account of the non-reduced shear force.

The condition described in expression (6.5), previously noted, must also be verified for the non-reduced shear force.

II.3.3.Anchorage of lower reinforcement at support level

The corresponding clauses are in [EC2-1-1 9.2.1.4 and 9.2.1.5].

3.3.0.a)Single end supports

Figure [Fig./Tab.II.(7)] shows that the section of reinforcement present at the supports must be anchored there, the anchorage length being measured from the face of the support.

This section may be calculated from the discrepancy rule, or by applying the following expression:

 $F_E = |V_{Ed}| \times a_l / z + N_{Ed}$ [EC2-1-1 Expr.(9.3)]

Where N_{Ed} is the axial concomitant acting stress

For the elements with shear reinforcement, $a_1 = z (\cot \theta - \cot \alpha) / 2$ and the expression becomes:

 $F_E = 0.5 |V_{Ed}| (\cot\theta - \cot\alpha) + N_{Ed}$

It is the expression (6.18) of [EC2-1-1 6.2.3(7)], seen in the article [Chapter I-II.2.3] to which the axial concomitant force is added.

The anchored section should not be less than $\beta_2 \times A_{\text{max}}$, with

A_{max} area of reinforcement in bay

 $\beta_2 = 0.25$ recommended value, subject to validation by the national annex

The national annex gives $\beta_2 = 0$ subject to verifying the following condition:

 $F_{\text{E}} = |V_{\text{Ed}}| \times a_{\text{l}} \, / \, z + N_{\text{Ed}} + M_{\text{Ed}} \! / z$

where M_{Ed} is the concomitant moment (to be taken with its sign)

Thus is obtained:

 $F_{\text{E}} = 0.5 ~|V_{\text{Ed}}|~(\text{cot}\theta - \text{cot}\alpha) + N_{\text{Ed}} + M_{\text{Ed}} / z$

It has been seen that the expression giving the discrepancy a_1 corresponds to the hypothesis of a multiple truss [Chater 6-II.2.3] which risks no longer being verified in the end support zone. A specific study of the equilibrium of the end strut should be carried out for each case; this will allow a more accurate determination of the longitudinal stress to anchor. A simple model with one truss leads to anchoring a force of V_{Ed} cot θ for the part brought by shear force. This corresponds to the previous practice which required the anchorage of the stress V_{Ed} for a strut angle $\theta = 45^{\circ}$, by simplification, which augers well for safety. The precise study of the end strut should lead to a stress intermediate between the values of $0.5V_{Ed} \times \cot\theta$ and $V_{Ed} \times \cot\theta$.

3.3.0.b)Intermediate supports

The recommendations concerning the end supports apply [EC2-1-1 9.2.1.5].





III.JUSTIFICATION RELATIVE TO TORSION

The corresponding clauses are in [EC2-1-1 6.3].

Further in the text the symbol $\tau_{t,i}$ of Eurocode 2 was replaced by $\tau_{T,i}$ (*T* for torsion) that is more consistent with $\tau_{V,i}$ (*V* for shear force).

III.1.<u>Principles</u>

Eurocode 2 deals explicitly only with the resistance to a circular (or pure) torsion of an element of a solid or hollow section and states that the warping torsion may be ignored in the case of box girders and solid sections [EC2-1-1 6.3.3].

Moreover, Eurocode 2 carries out the justification of the resistance in circular torsion in a thin-walled closed section, from equilibrium with the shear flow exerted; the case of a solid section is dealt with, as with previous regulations, by assimilating it to a equivalent thin-walled section. Each section wall is thus verified separately, according to the principle of a truss resistant to the shear force applied to it.

In the case of opening of joints without bonded reinforcement, instructions for the resistance and distribution of torsion stresses must be modified $[EC2-2\ 6.3(106)]$. This concerns box girder structures built in prefabricated segments without bonded prestress in the tensioned zone. The diagram of force distribution may be similar to that of an open section.

The study of concrete bridge decks relative to torsion, when their section is complex like a multi-beam or multi-box girder, should be preceded by an appropriate structural analysis allowing determination of torsional stresses for each longitudinal element. If these sections may be considered as non-deformable, then they may be justified according to the specifications of Eurocode 2.

A T section, if it may be considered as non-deformable, may be broken down into basic sections, each one modeled by a section with equivalent thin walls. The torsion resistance of the unit is taken as equal to the sum of the resistances of the basic sections. In this case, the redistribution of the torsion moments in the basic sections should be adjusted to the torsional rigidity in the non-cracked state of the sections. Each basic section may be calculated separately.

In the case of deformable sections, the study should be done using appropriate methods.

The warping torsion causes axial stresses that may not be ignored in the case of thin, open sections and very slender sections. In these cases, it may be studied using models of networks of beams or finite elements models.

III.1.1.Calculation of torsional shear stress flow in a hollow or solid section







B - outer edge of effective crosssection, perimeter u,

A - center line

C – concrete cover

Fig./Tab.III.(1): Torsion – equivalent hollow section

This calculation requires knowledge of the wall thicknesses. For a hollow section, $t_{ef,i}$ are the actual thicknesses of the walls.

In the case of a solid section the figure above represents the principle of determination of the hollow section that is its equivalent. The thickness of the walls $t_{ef,i}$ is thus assumed to be constant.

 $t_{ef,i} = A/u$ in general

A is the total area of the section defined by the exterior perimeter, the hollow part included.

u is the exterior perimeter of the section

 $t_{\text{ef},i}$ must be greater than twice the distance between the outside facing and the axis of the longitudinal reinforcement.

The shear stress flow in pure torsion is given by:

$$\tau_{T,i} t_{ef,i} = \frac{T_{Ed}}{2A_k}$$
 [EC2-1-1 Expr.(6.26)]

 T_{Ed} is the applied design moment of torsion

 $\tau_{T,i} \qquad$ is the tangential torsional stress in the wall i

 A_k is the area enclosed by the centre-lines of the connecting walls, including inner hollow areas

The tangential stress $V_{\mbox{\tiny Ed},i}$ in a wall i from the torsion is given by:

 $V_{Ed,i} = \tau_{T,i} t_{ef,i} z_i$ [EC2-1-1 Expr.(6.27)]

 z_i is the length of the wall i, defined by the distance between intersection points of adjacent walls.

There is a risk of confusion here since the symbol chosen by Eurocode 2 for the length of the wall is similar to that used for the lever arm of internal forces.

The justifications are then done for each wall, in the same way as for the shear force.



III.1.2. Principle of shear/torsion combination

In all cases, the effects of torsion and shear stress may be combined by taking the same value for the inclination θ of the struts [EC2-2 6.3.2(102)]. The limit values are those defined for the shear force.

In the case of box girders, it is advisable to verify each wall separately by taking account of the algebraic combination of the shear and torsional stresses.



A Torsion B Shear stress C Combination Fig./Tab.III.(2): Combination of stresses in the various walls of a box girder [EC2-2 Fig.6.104]

In the case of solid sections, the shear-torsion combination can no longer be done simply by combination of corresponding stresses as shown above. The shear stress exerts itself, in effect, on the walls of the equivalent hollow section of the element whereas the torsional shear is exerted on the walls of the equivalent hollow section. It is then necessary to revert to the shear and torsion stresses to carry out the verification, as shown below.

III.2. Verification of resistance to combined torsion and shear

III.2.1.Resistance of struts

The method of verifying the resistance of the concrete struts of elements subjected to shear and torsion stresses is defined by clause [EN1992-2 6.3.2(104)].

It distinguishes solid sections from hollow sections.

2.1.0.a)For solid sections

It is advisable to verify:

 $T_{Ed} / T_{Rd,max} + V_{Ed} / V_{Rd,max} \le 1.0$

where:

 T_{Ed} is the applied design moment of torsion

 V_{Ed} the applied design shear force

 $T_{Rd,max}$ is the design resistant moment of torsion given by:

 $T_{Rd,max} = 2 \nu \alpha_{cw} f_{cd} A_k t_{ef,i} \sin \theta \cos \theta \qquad [EC2-2 Expr.(6.30)]$

v is given in 6.2.2 (6) of Eurocode 2 part 1-1 and α_{cw} by note 3 of the expression (6.9); they were also shown in [Chapter I-II.2.2a)].

[EC2-2 Expr.(6.29)]



 $V_{Rd,max}$ is the maximum value of the design shear resistance according to expressions (6.9) or (6.14) of Eurocode 2 part 1-1. This concerns solid sections, and the total width of the web may be used to determine $V_{Rd,max}$.

2.1.0.b)For box girders

It is advisable to verify each wall separately for the combined effects of torsion and if any exists, of the concomitant shear force applied to the wall. The ultimate resistance of the concrete wall naturally corresponds to the resistance to the design shear force $V_{Rd,max}$ [EC2-2 6.3.2 (104)]

The tangential stress in a wall i due to the torsion $V_{\text{Ed},i(T)}$, is given by:

$$V_{Ed,i(T)} = \tau_{T,i} t_{ef,i} z_i$$
 [EC2-1-1 Expr.(6.27)]

Must be verified:

 $V_{\text{Ed},i(T)} + V_{\text{Ed},i(V)} \! < \! V_{\text{Rd},\text{max},i}$

With:

 $V_{\text{Ed},i(V)}\!:$ fraction of the applied shear force acting upon the wall i

V_{Rd,max,i}: resistant shear force of the wall i according to [EC2-1-1 Expr.(6.14)]

For example, in the case of a symmetrical box girder with 2 webs, one may determine for each web the resistant force and apply to it half the total force acting on the whole.

2.1.0.c) Other formulation of the verification criterion

The verification criterion may be explained by use of the shear stresses.

Thus, from the torsional shear stress flow in a wall:

$$\tau_{T,i} t_{ef,i} = \frac{T_{Ed}}{2A_k}$$
[EC2-1-1 Expr.(6.26)]

the corresponding shear stress is deduced:

$$\tau_{T,i} = \frac{T_{Ed}}{2A_k t_{ef,i}}$$

And it is advisable to verify:

 $\tau_{T,i}$ + $\tau_{V,i}$ \leq $\tau_{Rd,max,i}$

where $\tau_{T,i}$ and $\tau_{V,i}$ are the torsional and shear stresses in the wall i respectively, $\tau_{Rd,max,i}$ the acceptable shear stress whose value was previously expressed in article [Chapter I-II.2.2b)]. It must however be made clear that the shear stress is an mean stress, to be calculated on the wall element having characteristics used for the determination of torsional stress.

III.2.2. Transverse reinforcement

According to [EC2-1-1 9.2.3(1)], reinforcement that take up torsional stresses should be perpendicular to the axis of the structural element, and hence it is recommended to keep this arrangement in the case of a combination of torsional and shear stresses.

The shear and torsional stresses are combined and the calculation principles defined for the shear force $[EC2-2\ 6.3.2(102)]$ are applied.





Expr.6.13]

Chapter 6 - Justifications at ULS

The combined acting force should be balanced by the resistant force $V_{Rd,s}$ from the reinforcement. The reinforcement section is thus given by:

$$A_{sw}/s = (V_{Ed,i(V)} + V_{Ed,i(T)})/(z f_{ywd} \cot\theta)$$
[EC2-1-1]

The attention of the designer is drawn to z which should be strictly the lever arm of the internal forces of the wall studied..

As for calculation of the resistance of the struts, a calculation may be adopted from shear stresses for determination of the steel sections.

In fact the expression

 $A_{sw}/s = (V_{Ed,i(V)} + V_{Ed,i(T)})/(z f_{ywd} \cot\theta)$

 $A_{sw}/s = (\tau_{V,i} + \tau_{T,i}) b_w/(f_{vwd} \cot\theta)$

May also be written

On the subject of stress, one may directly add shear and torsional stresses and compare them to the stress acceptable at ULS as previously seen, and then calculate the steel sections by the expression given above.

III.2.3.Longitudinal reinforcement

They are obtained from the expression:

$$\frac{\sum A_{sl} f_{yd}}{u_k} = \frac{T_{Ed}}{2A_k} \cot \theta \qquad [EC2-1-1 \text{ Expr.}(6.28)]$$

where

 u_k is the perimeter of the area A_k

 f_{yd} is the design yield strength of the longitudinal reinforcement A_{sl}

 θ is the angle of the compressive struts

In the compressed chords, the longitudinal reinforcement may be reduced in proportion to the compressive force in the chord.

In the tensioned chords, it is advisable to add torsional longitudinal reinforcement to the other reinforcement, calculated for a same-load case. It is generally advisable to distribute the longitudinal reinforcement along the length of the wall z_i , but for small-dimension sections, they may be concentrated near the corners.

The bonded prestressing steels may be taken into account but the increase in their stress $\Delta \sigma_p$ reaches a maximum at 500 MPa.

In this case, $\sum A_{sl} f_{yd}$ in the expression (6.28) is replaced by $\sum A_{sl} f_{yd} + A_p \Delta \sigma_p$.

Nota: It is advisable, to be consistent with the limitation to f_{yd} for the reinforcement, to limit the increase in stress in the prestressing steels to f_{pd} . This is not required by Eurocode 2. It must also be remembered that this increase applies from the permanent state.

For application of the expression [EC2-1-1 Expr.(6.28)], it is observed that each of these members is equivalent to a force per linear meter of wall.

It may thus be written:

$$\Delta F_{td,T} = T_{Ed} \cot\theta / (2 \times A_k)$$

In a slab of thickness e whose mean compressive stress is σ_h , it may be written:

 $F_h = e \sigma_h$





The residual stress to be taken by the torsional reinforcement and per linear meter of slab is thus:

$$\Delta F = \Delta F_{td,T} - F_h = T_{Ed} \cot \theta / (2 \times A_k) - e \sigma_h$$

If $\Delta F > 0$, there is a residual tension to be taken by the reinforcement. In the inverse case, there is no reason to plan for additional longitudinal torsional reinforcement.

This verification initially concerns slightly-compressed slabs, but it may also extend to the bottom of the webs.

The combination of the longitudinal torsional reinforcement and the other reinforcement should normally be planned for where there are concomitant loads.

III.2.4.Special case of slightly-stressed rectangular sections

These are sections for which the condition given below is verified [EC2-1-1 6.3.2(5)]. They require no shear reinforcement and only a minimum longitudinal reinforcement:

 $T_{Ed} / T_{Rdc} + V_{Ed} / V_{Rdc} \le 1,0$

[EC2-1-1 Expr.(6.31)]

where

 $T_{\text{Rd,c}}$ is the cracking moment in torsion (moment of torsion before cracking of the element), deduced from the expression [EC2-1-1 Expr.(6.26)] by putting $\tau_{\text{T,i}} = f_{\text{ctd}}$.

or $T_{Rd,c} = 2f_{ctd}t_{ef,i}A_k$

 $V_{Rd,c}$ is the design resistant shear of the element in the absence of shear reinforcement given by [EC2-1-1 Expr.(6.2)]

IV.JUSTIFICATION RELATIVE TO PUNCHING

IV.1. Principle of justification

The justification relative to punching is to be done at the ULS resistance. It involves verifying that the shear flow produced by a concentrated load on a slab is acceptable. If the case arises, the quantity of shear strength steel to put in place to ensure resistance of the slab should be determined.

For the decks of highway bridges, this justification is carried out under the effect of the wheel LM2, that represents a heavy, localized load. Consideration will also be given to the additional dynamic increase coefficient near the pavement joints.

The wheel LM1, with a smaller impact area but a lower load, is a priori less unfavorable.

The same calculation principles may be used to justify the resistance to punching of a slab or a foundation footing relative to loads coming from the pier.

IV.1.1.Basic control perimeter





The spreading of stresses in the concrete has the effect of distributing the load effects. To take account of this favorable effect, basic control perimeters are defined [EC2-1-1 6.4]. It is then considered that the shear will be distributed in a uniform manner along the whole perimeter u_1 :



Fig./Tab.IV.(1): Basic control perimeter around loaded areas

d is calculated from d_y (center of gravity of longitudinal steels) et d_z (center of gravity of transverse steels):

$$d = \frac{d_y + d_z}{2}$$

It is important to note that spreading of the load occurs along the total height of the concrete but also on the pavement thickness.

IV.1.2.Shear calculation on the control perimeter

Shear develops on an area of concrete $u_1 \times h$, the expression of shear is thus as follows:

$$v_{Ed} = \beta \cdot \frac{V_{Ed, red}}{u_1 \cdot d}$$
 where:

 $V_{\text{Ed,red}}$ is the punching stress

 β is the offset of the load, $\beta = 1$ is taken in the case of a centered load.

IV.1.3. Calculation of shear resistance vRd, c of concrete without punching shear reinforcement

In the absence of punching reinforcement, the shear resistance is given by [EC2-1-1 6.4.4(1)]:

$$\mathbf{v}_{Rd,c} = Max \left\| \left(C_{Rd,c} \ k \left(100 \ \rho_1 \ f_{ck} \right)^{\frac{1}{3}} + k_1 \ \sigma_{cp} \right) \right\| ; \left(\mathbf{v}_{min} + k_1 \ \sigma_{cp} \right) \right\|$$

where:

- f_{ck} is given in MPa
- $k = 1 + \sqrt{\frac{200}{d}} \le 2,0$, with d in mm
- $\rho_1 = \sqrt{\rho_{ly} \rho_{lz}}$ (maximum at 2%)
- $\sigma_{cp} = \frac{\sigma_{cy} + \sigma_{cz}}{2}$ in MPa, with a minimum value of -1.85MPa [EC4-2].





•
$$C_{Rd,c} = \frac{0.18}{\gamma_c} = 0.12$$

•
$$k_1 = 0,10$$

• If
$$\sigma_{cp} < 0$$
: see Eurocode 4 part 2

•
$$C_{\text{Rd,c}} = \frac{0.15}{\gamma_c} = 0.10$$

• $k_1 = 0.12$

•
$$v_{min} = 0.035 \times k^{3/2} \times f_{ck}^{1/2}$$

The calibration of the formula of punching resistance was done with the v_{min} value based on the shear stress resistance of beams without shear reinforcement. The correction made for v_{min} by the national annex of Eurocode 2 part 1-1 for slabs should not be applied to punching resistance.

If the verification is not satisfied shear stress steels must be accounted for. The control perimeter for which this relationship is satisfied is then looked for, and steel reinforcement calculated with the expression (6.52) is placed up to a distance of 1.5d of this contour; adjacent to the column punching control is made according to expression (6.53).

IV.2. Example of application

Example is done on a highway bridge slab of 22cm thickness with a pavement thickness of 11 cm. The most unfavorable case is treated with an LM2 wheel and a maximum dynamic increase (near the pavement joint).

• The wheel LM2 load is:

$$\Delta \phi_{\rm fat} \, \frac{\beta_{\rm Q} \, Q_{\rm ak}}{2} = 1,30 \times \frac{400}{2} = 260 \, \rm kN$$

- Its impact zone is a rectangle of 0.35×0.6 . The impact surface on the upper face of the concrete slab is a rectangle 0.57×0.82 (diffusion at 45° through the 11cm of pavement).
- The average position of the two lower layers of transverse steel is taken as equal to d = 0.16m
- The basic control perimeter is defined with the help of figure 6.13 of Eurocode 2 part 1.1, from the impact zone. We obtain $u_1 = 2 \times (0.57 + 0.82) + 4\pi d = 4.92m$
- Along the perimeter, the shear value is thus:

$$v_{Ed} = 1 \times \frac{0,260}{4,92 \times d} = 0,31 MPa$$

- Punching resistance of slab:
 - $\rho_1 = \sqrt{\rho_{1y} \rho_{1z}} > \sqrt{0.13\% \times 0.13\%} = 0.13\%$ (most unfavorable hypothesis) • $k = Min \left(1 \pm \sqrt{200} \cdot 2\right) = 2.0$
 - \circ k=Min $\left(1+\sqrt{\frac{200}{170}};2\right)=2,0$
 - \circ $\sigma_{cp} = 0$ MPa (the favorable effect of the possible longitudinal compression is ignored)
 - thus $C_{Rd,c} = 0.12$ et $k_1 = 0.10$





$$v_{min} = 0.035 \times 2.0^{\frac{3}{2}} \times \sqrt{30} = 0.54 \text{MPa} > 0.38 \text{ MPa}$$

- $\circ \quad v_{Rd,c} = v_{min} + k_1 \sigma_{cp} = 0.54 \text{ MPa}$
- Justification:

We have: $0,31MPa=v_{Ed} \le v_{Rd,c} = 0,54MPa$

The punching resistance is sufficient: punching steels are unnecessary.

V.FATIGUE VERIFICATION

It is advisable to carry out a justification relative to fatigue for structures and structural elements subjected to repeated load cycles. It must be carried out separately for steel and concrete.

Justification of fatigue of compressed concrete is not dealt with in this chapter. The national annex allows this verification to be dispensed with for sections whose stress in the concrete is limited to $0.6 \times f_{ck}$, under a combination of characteristic loads in the form of a rule h) added to the list shown below.

Accordingly, justification of fatigue is not generally necessary for the following structures and elements [EC2-26.8.1(102) a) to g) and EC2-2/AN 6.8.1(102) h) to k)]:

a) footbridges, with the exception of structural components very sensitive to wind action;

b) buried arch and frame structures with a minimum earth cover of 1.00 m and 1.50 m respectively for road and railway

bridges;

c) foundations;

- d) piers and columns which are not rigidly connected to superstructures;
- e) retaining walls of embankments for roads and railways;

f) abutments of road and railway bridges which are not rigidly connected to superstructures, except the slabs of hollow abutments;

g) prestressing and reinforcing steel, in regions where, under the frequent combination of actions and Pk, only compressive stresses occur at the extreme concrete fibres.

h) compressed concrete in road bridges when $\sigma_c < 0.6 \times f_{ck}$ under the characteristic combination for SLS;

i) tensile reinforcement in sections of reinforced concrete sections of road bridges when σ_s < 300MPa under a characteristic combination for SLS.

j) prestressing steels and reinforcement, in the areas where, under frequent combination for SLS with P_m , the boundaries of the concrete sections remain compressed;

k) shear reinforcement for the reinforced concrete structures, when these reinforcement have been dimensioned at ULS with a diagram of struts inclined at θ so that $1.0 \le \cot \theta \le 1.5$.

Sétra





The shear stress reinforcement subjected to variations in stresses should also be verified relative to fatigue. But for the prestressed concrete where the criteria of annex QQ have been respected, there is no fatigue problem since the section is not cracked in SLS and for reinforced concrete the application of the limit of inclination of the strut cited above means the verification may be dispensed with.

Accordingly, in the following examples, only fatigue verifications of longitudinal bending reinforcement or prestressing are dealt with.

V.1. Justification principle and elements

[EC2-1-1 and EC2-2 6.8]

Justification to fatigue of a reinforced or prestressed concrete section consists essentially of preventing the failure of tensile reinforcement subjected to repeated stress variations under the effect of cyclic loads during the design working life of the structure. In the case of road structures in service, the stress variations causing fatigue in reinforcement are mainly due to the passing of heavy vehicles (the high level of loads and hence the stress variation created, a high number of cycles plays an important part in resistance to fatigue).

The principle of the justification brings up ideas of stress range, application cycles, curves of resistance to fatigue, damage, fatigue load model etc., and is summarized in the form of the verification of the Palmgren-Miner combination rule.

V.1.1.Fatigue load models

Eurocode 1 proposes five fatigue load models, FLM1 to FLM5 [EC1-2 4.6].

Model FLM1 is built from model LM1 whose loads have been reduced. The model FLM2 is made up of five 'standard' trucks that must be used separately. Model FLM3 is made up of a single truck. These three models allow a simple fatigue justification based on the determination of a single maximum stress range. The first two are, however, on the one hand very safe and on the other hand are only suitable where there is a fatigue limit of a constant amplitude defined on the S-N curves (it is generally the case for steel construction units, not for reinforcement or for prestressing steel).

The FLM4 and FLM5 are more elaborate, and planned to obtain a spectrum of stress ranges destined to allow a calculation of damage. The first is made up of 5 trucks that allow generation of artificial traffic by proportional adjustment and representation of an overall traffic situation. The second is directly actual recorded traffic.

Only the two models FLM3 and FLM4 are used for a 'usual' justification to fatigue of concrete bridges.

V.1.2.Combination of actions – State of reference

As mentioned in [Chapter 2-IV.2.3] of this guide, the combination of actions for the fatigue verification breaks down into:

- a basic combination C₀ of non-cyclic loads representing the average state of the structure in service, empty (permanent loads, temperature effect)
- to combine with the cyclic fatigue load Q_{fat} represented by the model of adequate fatigue load.

The empty state of structure state is thus represented and expressed by the combination:

 $C_0 = G + P_k + 0.6 \Delta T_M$ where ΔT_M represents the thermal gradient effect.

This reference state changes with the passing of fatigue load Q_{fat} on the structure causing short-term variations of stress.





•

The empty state of structure changes during the whole of its operating time due to the evolution of prestress, shrinkage, creep and fluctuations of thermal gradient. These slow variations of the reference state however should not be combined with the rapid variations due to the passing of trucks.

This may be schematized in the figure below:



Fig. 1 ab. v. (1): Development of stress in steels in a deck section		

Circulation des poids lourds	Movement of heavy vehicles
Etat à vide	Empty state
Mise en service	Start of service
Temps	time

 σ_0 is the stress under prestress and permanent loads, developing slowly under long-term effects such as shrinkage, creep, different losses of prestress, or medium-term effects (thermal gradient).

 $\Delta \sigma_{PL}$ are the rapid variations of stress under the effect of the heaviest trucks passing around stress level $\sigma_{0.}$

Eurocode 2 then specifies that:

"The cyclic force Q_{fat} should be combined with the unfavorable basic combination." [EC2-1-1 6.8.3(3)]

We will look for the combination that combined with the fatigue load produces a maximum stress variation. We will thus retain the combination giving a maximum of tension in the reinforcing steels or a minimum stress in concrete in the covering area, when this stays compressed under empty state of reference.

For permanent loads G, it is the maximum value G_{max} that is used.

For the prestress, [EC2-1-1 5.10.9] the lower characteristic value $P_{k,inf} = r_{inf}P_{m,t}$, will be retained, $P_{m,t}$ being the mean prestress force at time t. Moreover, the concrete creep and shrinkage have the effect of reducing the prestress and the compression of the concrete in the covering area of the reinforcing steels studied. Accordingly



the long-term situation of the structure is in general more unfavorable. It is thus by agreement that long-term stat will serve as a reference situation.

It is logical enough to consider this state. The stress variations are applied during the whole design working life of the structure. On the other hand creep and shrinkage of concrete will cause stress development only in the first 20 years. For a structure having a design working life of 100 years, the 'long-term' state, which involves 80% of design working life, will thus be the most representative.

The empty state of reference of the structure is thus represented and expressed by the combination:

 $C_0 = G_{max} + P_{k,inf} + 0.6.\Delta T_M$ where ΔT_M represents the thermal gradient effect.

V.1.3. Stress calculations

Once the reference state has been chosen, the stress variations in the reinforcement are now just due to translation of fatigue load that causes variations of action effecs at the origin of stresses. For a given section the bending moment may be used to illustrate this in the following figure:



Fig./Tab.V.(2): Variation of moment due to translation of Q_{fat}

 $M_{fat} = M_0 + M_{Qfat}$ is the total bending moment; it fluctuates according to the position of the fatigue load, just as M_{Qfat} the bending moment due to the fatigue load alone, whereas the bending moment of the reference state, M_0 stays constant.

The stress ranges are then calculated from the fatigue combination of actions, obtained from an elastic-linear analysis.

Although justification of fatigue is justification at ULS because failure of reinforcement by fatigue is an ultimate limit state, Eurocode 2 rightly recalls that the combination of actions for the fatigue justification is similar to SLS with frequent frequent loads.

V.2. Methods of verification

Eurocode 2 proposes several methods for verification of the fatigue strength of reinforcement:

• A general method with determination of the spectrum of stress ranges by using fatigue load models FLM4 or FLM5 and a calculation of damage;



- \circ a method of the range of equivalent stress [EC2-1-1 6.8.5] and [EC2-2 Anx.NN], later called equivalent method, with determination of the stress range that would give an equivalent damage by using the fatigue load model FLM3 for road bridges;
- an alternative method [EC2-1-1 6.8.6] for a simplified verification of reinforcement using a frequent cyclic load that , more precisely, may be done with a combination of frequent actions, using the main load model LM1 for road bridges.

These three methods are applied to the two following examples:

- in longitudinal bending: verification of reinforcing and prestressing steels of a PSIDP,
- in transverse bending: verification of reinforcement of the cantilever of a box-girder

The different steps of verification require a long development. For more clarification, they are shown in detail in appendix [Appendix IV].

VI.JUSTIFICATION RELATIVE TO BRITTLE FAILURE

Justification relative to brittle failure is little developed in part 1-1 of Eurocode 2 [EC2-1-1 5.10.1(5)P; 5.10.1(6)]. It is found though in two places in Eurocode 2 part 2, in sections 5 and 6: clause 5.10.1(106) defines the general objective; clauses 6.1(109) and 6.1(110) describe more precisely the verification methods to use.

In this chapter, after a description of the principle and the basic requirements relative to justification relative to brittle failure, the two verification methods proposed by Eurocode 2 part 2 are in turn described and detailed. A digital application from the case of a beam from the VIPP bridge illustrates these two methods. **Appendix VI** develops the calculation detail for the PSIDP examples and of the box girder bridge built by balanced cantilever method.

VI.1. Principle and basic requirements

The criterion of brittle failure laid down by Eurocode 2 part 2 for prestressed concrete bridges has as an objective the prevention of brittle failure in structural elements from the appearance of the first crack.

The targeted risk concerns the consequences of the failure of a number of prestressing steels mainly by corrosion, if this failure occurs near the same section of an element and cannot be observed until the appearance of the first bending crack. When cracking occurs, the reinforcement must take over in helping the concrete resist tension, with a sufficient margin for intervention in effective time.

This principle may be considered as satisfied if the requirements defined below are respected by the linear structural elements of the structure (beams, box girders, joists..) prestressed by internal prestress of the concrete, implemented by post-tension [EC2-2/AN 6.1(109)]. These rules are also applicable to slabs functioning as beams when their use is specified. Their extension to this type of structure is only recommended, however, for narrow slab (e.g. less than 4m wide, excluding lateral cantilevers.





Justification of the brittle failure criterion may be used by any one of the two following alternative methods (the national annex rules out the use of method c seen in Eurocode 2):

- Method a): verify that in case of successive failure of tendons or strands, cracking will occur before the ultimate strength is exceeded, under the effect of frequent loads.
- Method b): plan for minimum adequate reinforcement capable itself of taking up the cracking moment in the assumed absence of all prestress.

These two methods have the same objective, which is to allow the detection of possible deterioration of the prestress by the appearance of cracks that are detectable during normal monitoring of the structure, so that the client may be alerted and traffic interrupted for replacement of the corroded tendons before collapse of the structure.

The criterion of brittle failure concerns only tensioned zones under the effects of the characteristic SLS, determined by ignoring primary effects of the prestress.

Eurocode 2 is only explicit for method b, but it is even truer with method a): if all the tendons have been removed and the section is compressed in characteristic SLS, it is compressed in frequent SLS and thus its ultimate strength is not exceeded.

The criterion of brittle failure applies only to longitudinal internal prestress. The tendons making up the external prestress, protected by supple products (grease or wax), may be regularly monitored and damage to them is more easily detectable. Hence this criterion does not apply to them. Corrosion of the transverse prestress, generally injected by wax and unbonded, leads only to local problems and is consequently no victim to this criterion.

Moreover, the national annex dispenses elements prestressed by pretension from justification for brittle failure, considering that the tendons of pretensioned prestress are protected from corrosion by concrete cover, as with reinforcing steels, and that the risk of generalized corrosion of a tendon is less than in post-tension.

If clauses [EC2-1-1 5.10.1(5)P] and [EC2-2 5.10.1(106)] are general, , clause [EC2-2 6.1(109)] on the other hand restricts their application to bending. It is the part that is retained in the examples presented later. However, in the VIPP example studied, it would be pertinent to consider brittle failure by shear stress near the support. An indication of the procedure is given at the end of the digital application.

VI.2. Verification according to method (a)

When the condition of the prestressing tendons deteriorates under corrosive effects, the strength of the section drops (breakage of wire, strands or tendons). The principle of verification according to method (a) consists of guaranteeing that the ultimate strength level corresponding to a detectable cracking under frequent loads, stays greater than the level of actions effects imposed by these same frequent loads. In other words, to guarantee that under the effect of frequent loads, there is a sufficient margin of safety between cracking and collapse of the element [Fig./Tab.VI.(1)].

The ultimate strength of the section is evaluated by partial factors of materials associated with an accidental design situation.





Fig./Tab.VI.(1): Principle of verification according to method (a)

Niveau de résistance	Strength level
Fissuration	Cracking
Fissuration suivie de la ruine de l'élément	Cracking followed by failure of element
Ruine de l'élément	Failure of element
Nombre de fils rompus	Number of broken wires

Practical application of the method is done on the basis of stresses $\sigma_{c,f}$ obtained under the effect of frequent SLS loads on outer tensioned fibers and is in 2 steps:

<u>Ist step</u>: Determine, under the effect of frequent SLS loads, the percentage of prestressing tendons to consider to be damaged to cause the first crack in the element. This quantity is expressed, for each tendon bed (i) as a percentage α_i of the total prestress force $P_{m,t}$:

$$\alpha_i \text{ as:} \quad \sigma_{c,f} - \sum_i \alpha_i P_{m,t} \left(\frac{1}{S} + \frac{e_{0i} \times y}{I} \right) = -f_{ctm}$$

In this calculation, it is first advisable to consider removal of strands on the bed nearest to the outer tensioned fiber (i.e. the most exposed to corrosion). If the removal of all tendons from this bed is not enough to cause cracking, the next bed will be considered, and so on...

So four combinations are to be considered, which will successively bring in $P_{m,0}$; $P_{m,\infty}$; $M_{ELS\,freq,max}$; $M_{ELS\,freq,min}$. In practice, it will be sufficient to study only combinations ($P_{m,\infty}$, $M_{ELS\,freq,max}$) and ($P_{m,\infty}$, $M_{ELS\,freq,min}$),more representative of the phenomenon that may occur only after a certain time.

It should be noted that only the primary effect of the prestress is removed, the secondary effects being kept in the structure for the internal bonded tendons.





<u> 2^{nd} step:</u> Verify that with this reduced prestress and a proportional reduction of the prestressing steel section, the ultimate bending strength is greater than the moment given by the frequent combination of actions. If the conclusion is negative, reinforcement should be added to satisfy the condition.

The sections that may require additional reinforcement relative to verification of the criterion according to this method (a) are generally those situated where the bending moment corresponding to the frequent combination of actions is low, between $\frac{1}{4}$ and $\frac{1}{3}$ of the indeterminate structure, and near to the supports in the case of determinate structure. This is why it is recommended to refine the analysis at the level of these sections, by interpolation between sections studied.

For the calculation, simulation of the reduction of the prestress force (strand breakage) by an external loading culminating in the frequent SLS vector is recommended:

 $N_{tot} = N_{ELS \ freq} - \Sigma \ \alpha_t \ .P_{m,t}$

 $M_{\text{tot}} = M_{\text{ELS freq}} - \Sigma \; \alpha_{\iota} \; .P_{\text{m,t}} \; . \; e_{0 \, \iota}$

Where $N_{ELS freq}$ et $M_{ELS freq}$ represent the vector of actions effects applied on the section for the frequent SLS combination studied (including the full effects of prestress).

It is thus verified that the couple $N_{to b}$ M_{tot} is inside the resistance diagram, obtained by applying to the materials the partial factors corresponding to the accidental design situation.

Eurocode 2 part 2 specifies that the effects of the possible redistribution of the forces' linked to the cracking, may be taken into account. However these results from a non-linear analysis that is not normally part of the usual calculations of the ULS resistance, and may be ignored.

VI.3. Verification according to method (b)

Method (b) of the brittle failure criterion verification consists of applying a minimum reinforcement $A_{s,min}$ defined by the expression (6.101a) of Eurocode 2 part 2 mentioned below:

$$A_{s, min} = \frac{M_{rep}}{z_s \times f_{yk}} \qquad [EC2-2 \text{ Expr.}(6.101a)]$$

In this expression:

- M_{rep} represents the cracking moment, given by the equation: $M_{rep} = \frac{-f_{ctm} \times I}{y}$;

- z_s is the lever arm of the reinforcing steels at the ULS resistance (≈ 0.9 d in the case of a rectangular section).

It should be stated that from the point of view of the reinforcement sections calculated, method (b) makes up an envelope of method (a) when it is assumed all the prestressing tendons are worn. In fact it is reinforcement to avoid brittle failure calculated with N = 0.

 $A_{s, min}$ should be arranged in all the tensioned zones under the effects of the characteristic SLS, determined in ignoring the prestress primary effects.





Moreover, it is advisable to count in $A_{s, min}$, all the longitudinal reinforcing steels arranged for other reasons (reinforcement of longitudinal bending, minimum, fatigue etc..)

Eurocode 2 part 2 also anticipates, under certain conditions, counting in $A_{s, min}$ the prestressing tendons [EC2-2 6.1(110) ii)]. This should be referred to if appropriate.

- \circ In the case of indeterminate beams, A_{s,min} of the lower fiber should be extended on intermediate supports, except if it may be demonstrated that plasticizing of the tensioned steels in the top fiber of the section at support comes before breakage by crushing of the compressed concrete in the bottom fiber.
- This condition is considered as settled if the following relationship is verified:

 $A_s \times f_{yk} + A_p \times f_{p0,1k} < t_{inf} \times b_0 \times \beta_{cc} \times f_{ck}$ [EC2-2 Expr.(6.102) and EC2-2/AN 6.1(110)iii)]

In this expression:

- A_{s} et A_{p} represent respectively the areas of reinforcement and of prestressing tendons in tensioned fiber ;

- t_{inf} et b_0 are respectively the height and the width of the lower chord of the section ($t_{inf} = b_0$ in the case of T sections, and $t_{inf} = 0.2h$ in the case of rectangular sections).

In general, in the case of bridges, this criterion is verified without too much problem because the high-compression zones of concrete (on supports) also correspond to zones where the density of reinforcement is very high.

In the joints of precast segments, where for obvious reasons linked to the method of construction, it is impossible to arrange a reinforcement, the formulae of Eurocode 2 part 2 logically lead to no reinforcement to avoid brittle failure.

Digital application: case of a VIPP

The beam to be considered is represented below. In this example sections considered will be those situated near the supports: at ¹/₄ of the span and at 1m from the support.











Section at $\frac{1}{4}$ and at $\frac{1}{2}$ span

Characteristics of the section:

 $S = 1.66 m^2$ I = 0.886 m⁴ v' = 1.55 m

Materials:

Concrete: C35/45; $f_{ctm} = 3.2$ MPa

reinforcing steels in lower fiber: $f_{yk} = 400 \text{ MPa};$ $A_s = 13.84 \text{ cm}^2; \text{ c}^2 = 5 \text{ cm}$

Prestress:

$$\begin{split} & 6 \times 4T15S \; (Ap = 6 \times 600 \; mm^2) \\ & f_{p0,1k} = 1660 \; MPa \\ & c_{0_lit\;lower} = 10 \; cm \\ & c_{0_lit\;lupper} = 20 \; cm \end{split}$$

In the section situated at 1 meter from the support, it is assumed that the center of gravity of the tendon assembly is at the center of the web, at a distance of 1.10m from the lower fiber.

	at 1 m from support	at ¼ of spam
P _{m,∝}	5.02 MN	5.02 MN
$M_{ m ELS\ Freq}$	-1.02 MNm	0.48 MNm
$\sigma_{\rm c,f}$ en fibre inf	4.81 MPa	2.18 MPa

Stresses obtained at frequent SLS:





•Calculation of number of strands to remove to obtain cracking at frequent SLS (method a): α_i on the bottom bed as:

$$\sigma_{c,f} - \alpha_{i} P_{m,\infty} \left(\frac{1}{S} + \frac{e_{0i} \times y}{I} \right) = -f_{ctm}$$

or:
$$\alpha_{i} = \frac{\sigma_{c,f} + f_{ctm}}{P_{m,\infty} \left(\frac{1}{S} + \frac{e_{0i} \times y}{I} \right)} = \frac{\sigma_{c,f} + 3.2}{5.02 \times \left(\frac{1}{1.66} + \frac{(-1.55 + c_{0}) \times (-1.55)}{0.886} \right)}$$

The number of strands to remove to reach cracking in the lower fiber is then obtained by multiplying α_i by the total number of strands, or:

$$n_i = \alpha_i \times (6 \times 4)$$

	at 1 m from support	at ¼ of span
$\sigma_{\rm c,f}$ en fibre inf	4.81 MPa	2.18 MPa
C 0	1.10 m	0.10 m
α	114.9%	34.2%
n _i	24	8.2

Since the lower tendon bed contains only 20 strands, this comes to removing the equivalent of:

8.2 strands from the strands on the lower bed at $\frac{1}{4}$ of the span;

24 strands (the total of the tendons) at 1m from the support.

•Verification of ultimate strength of the section with "reduced prestress" under the cumulative effect of the combination of frequent actions and of the reduction of the calculated prestress (method a):

The forces and moments to apply to the section is obtained by deduction from the frequent SLS actions of the primary effect of the prestress assumed damaged determined at the previous stage:

$$N_{tot} = (1 - \Sigma \alpha_i) P_{m,\infty}$$

$$M_{tot} = M_{ELS Freq} - \Sigma \alpha_i \cdot P_{m,\infty} \cdot e_{0i}$$

with: $e_{0 \text{ lit inf } 1/4 \text{ travée}} = -(1.55 - 0.10) = -1.45 \text{ m}$

$$e_{0_cdg_1m} = -(1.55 - 1.10) = -0.45 m$$

N _{tot}	0 MN	3.30 MN
M _{tot}	1.24 MNm	2.97 MNm

The last step in the calculation is verification , from a section calculation, that the value couples (N_{tot} ; M_{tot}) are in the ULS resistance diagram, after removal of the tendons assumed to be corroded, and if the case arises to determine the additional reinforcement to be added.



	at 1 m from support	at ¹ / ₄ of span
M _{maxi} acceptable	1.20 MN	11.07 MN
Coeff. Of safety	0.97	3.72
Additional reinforcement necessary	0.58 cm ²	none

This calculation leads to the following results:

The only sections requiring a few additional reinforcement are thus those in immediate proximity to the support. The other sections are verified in relation to the brittle failure criterion (method a) in appropriate safety (safety factor of 3 to 4).

•Calculation of minimum reinforcement according to method b):

The minimum reinforcement to be arranged according to method (b) is determined from equation [EC2-2 Expr.(6.101a)]:

$$A_{s,min} = \frac{M_{rep}}{z_s f_{yk}}$$

where: $M_{rep} = \frac{-f_{ctm} \times I}{-v'} = \frac{-3.2 \times 0.886}{-1.55} = 1.83 \text{ MN.m};$ $f_{vk} = 400 \text{ MPa}.$

The lever arm of the reinforcing steels at ULS, z_s , is directly obtained from the section calculation software :

 $z_s = 2.15 \text{ m}$ (to compare to 0.9 d = 0.9 × (2.22-0.05) = 1.95 m)

whence:

$$A_{s,min} = \frac{1,83}{2,15 \times 400} = 21.28 \text{ cm}^2,$$

or an additional area of reinforcement of: $2128 - 1384 = 744 \text{ cm}^2$, to be arranged along the whole length of the beam. Method (b) is thus much more restricting than method (a).

It may be verified in this example that there is no risk of brittle failure with shear, by extrapolating method a) to the shear. The digital application below shows how to proceed.

The section at 1m from the support is considered. The ULS shear stress in this section is:

 $V = 1.35.Vg + Vp + 1.35.Vq = 1.35 \times 0.68 - 0.63 + 1.35 \times 0.87 = 1.46 MN$

Now all the prestress is removed. The shear stress under frequent load is now:

V = Vg + Vq, fréq = 0.68 + 0.52 = 1.20 MN.

(this value is greater than the shear under characteristic SLS load V = Vg + Vp + Vq = 0.92 MN)



The value stays less than the ULS shear. The shear reinforcement are thus sufficient; there is no risk of brittle failure.

If this value had been exceeded, it would have been possible to refine the verification by determining more accurately the quantity of prestress that would need to be removed to cause cracking of the web (in the sense of the criterion given in the annex QQ of EN1992-2). It is possible that cracking under shear would be visible after removal of all the prestress. It would then have been possible to compare this value with the ultimate strength of the beam in an accidental situation to verify if the shear reinforcement was sufficient.

Chapitre 1-

VII.NON-LINEAR AND SECOND-ORDER ANALYSIS – STABILITY OF FORM OF A PIER

VII.1. Generalities on linear and second-order analysis

The general instructions concerning **non-linear analysis** [Chapter 2-VI.3.5], the taking into account of **second order effects** [Chapter 2-VI.3.6] and the **geometric imperfections** [Chapter 2-VI.1] are entirely applicable.

Similarly, the taking into account of **creep** follows the principles in [Chapter 4-II].

Nevertheless, Eurocode 2 adds for the subject dealt with here, that the creep effect may be ignored [EC2-1-1 5.8.4(4)] if the three following special conditions are satisfied:

 $\phi_{ef} = 0 \quad si \quad \begin{cases} \phi(\infty, t_0) \le 2 \\ \lambda \le 75 \quad E \text{ lancement} \\ \text{Moment de la combinaison considérée 1er ordre} \\ e_1 = \frac{\overbrace{M_{0Ed}}}{\underset{effort normal sollicitant}{\underbrace{N_{Ed}}}} \ge h \begin{cases} \text{hauteur} \\ \text{section} \end{cases}$

The simultaneous verification of these three criteria is relatively conservative and should only happen in simple and unquestionable cases.

To deal with , at ULS, non-linear and second-order analysis, part 1-1 of Eurocode 2 [EC2-1-1 5.8.5] suggests a general method followed by two simplified methods for simple cases , mainly isolated elements or those that come down to isolated. These simplified methods are, strictly speaking, linear calculations with fixed and simple factoring of non-linear effects.

For bridges, Eurocode 2 part 2 suggests a less classical method for non-linear analysis, founded on the concept of a format of global safety, and whose safety level is determined in relation to the calculation of the structure's collapse.







To facilitate reading of the guide, only the two general methods will be presented here. The general method of part 1-1, mostly classic, will be dealt with quite briefly. The new approach of the general method of part 2 with the global safety format will be treated in more detail. As for the two simplified methods, they are dealt with in appendix [Appendix VI], where as an example the complete study is shown of the stability of two bridge piers with the use of the four methods from Eurocode 2.

VII.2. General method of non-linear and second-order analysis from Eurocode 2 part 1-1

This method is described in clauses [EC2-1-1 5.8.6].

VII.2.1.Principle of method

The principle of the method consists of demonstrating, as with previous regulations, that there exists a state of internal stresses in the structure that balances the external stresses, including those of second order effects. The verification of the sections is done with the stresses obtained at equilibrium of the structure by situating them in relation to the field of resistance, determined with the characteristics of the materials given by the stress-strain laws defined to this effect and mentioned in [Chapter 6-I.1].

It may be noted that the structural analysis and section verification use two different systems of stress-strain laws.

When the designer does not have access to software allowing him to combine geometric non-linearity with material non-linearity, it is possible, for a section judged to be critical, to find the equilibrium state by expressing in two ways the relationship linking the bending moment in the critical section with its curvature:

• The external moment-curvature law where the bending moment acting upon the section is the sum of the first-order moment M_{0Ed} and the second-order moment. Since the second-order moment depends upon the whole of the curvatures along the structure, it is quite difficult to determine such a curve according to only the curvature in the critical section studied.

To simplify, it is first possible to assume that distribution of the curvatures along the structure is linear, which allows determination of the second-order effect according to the curvature of the critical section only, and finally verify the hypothesis. It should be noted that this method is quite conservative since the actual curvatures along the structure are over-evaluated. If the result of the calculation is too unfavorable, account must be taken of the actual distribution of the curvatures along the structure and the process repeated.

- The internal moment-curvature law where the resisting bending moment results from the state of stress of the reinforced concrete section subjected to an imposed curvature, at a given axial force.
- The intersection or not of the two curves representative of the external law and the internal law allow verification whether or not a state of equilibrium exists. If so, the intersection of the two curves gives the value of the total moment M_{Ed} at equilibrium.





Fig./Tab.VII.(1): Verification of state of equilibrium and et determination of total moment M_{Ed}

Loi moment-courbure externe simplifiée, supposant la distribution linéaire des courbures	Simplified external moment-curvature law, assuming linear distribution of curvatures.
Loi moment-courbure interne	Internal moment-curvature law
Loi moment-courbure externe réelle	Actual external moment-curvature law
Plastification armatures	Yielding of reinforcement
Fissuration béton	Cracking of concrete
Courbure y	Curvature y
Répartition des courbures le long de la structure	Distribution of curvatures the length of the structure
Zone non-fissurée	Non-cracked zone
Zone fissurée	Cracked zone

VII.2.2.Materials

The method uses the concrete stress-strain diagram dedicated to a non-linear analysis [EC2-1-1 5.8.6(3)] that was described in [Chapter 2 V.3.1]. This is not entirely satisfactory because this diagram requires the strain modulus E_{cm} of concrete; the analysis could thus under-estimate the strains and not give sufficient safety, particularly when the second order effects is taken into account.

This is why Eurocode 2 offers a better alternative by suggesting the use, at the end of the same clause [EC2-1-1 5.8.6 (3)], of stress-strain diagrams based on design values.

These diagrams use the strain modulus E_{cd} for concrete obtained by dividing E_{cm} by $\gamma_{cE}=1.2$ and f_{cd} in place of f_{cm} , or the behavioral law defined by the following parameters and figure:







where

 $\epsilon_{\rm c}$ strain of concrete

$$k = \frac{1,05 \ E_{cd}}{f_{cd}} \frac{|\varepsilon_{c1}|}{|\varepsilon_{c1}|}$$

 $\epsilon_{cl}(\sigma_{00}) = min(0.7(f_{ck} + 8)^{0.31}; 2.8) \qquad \text{strain at peak of stress}$

$$\varepsilon_{cul}(0/_{00}) = \begin{cases} 3,5 \text{ pour } f_{ck} < 50 \text{MPa} \\ 2,8 + 27 \left[\frac{98 - (f_{ck} + 8)}{100} \right]^4 \text{ pour } f_{ck} \ge 50 \text{MPa} \end{cases}$$

ultimate strain

 $f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_C}$ design compressive strength value of concrete

 $E_{cd} = \frac{E_{cm}}{\gamma_{cE}}$ design value of modulus of elasticity of concrete

$$E_{cm} = 22000 \left(\frac{f_{ck} + 8}{10} \right)^{0,3}$$
 secant modulus of elasticity of concrete



Fig./Tab.VII(2): Concrete behavior law with design values

Contrainte	Stress
Loi de type Sargin	Sargin-type law
Deformation relative	Strain





For reinforcement and prestressing steels, the laws anticipated for the verification of sections [Chap.6 I.1.2] and [Chap.6 I.1.3] and defined by f_{yd} and f_{pd} are used.

A single set of stress-strain diagrams is thus used for structural analysis as for verification of sections, which is effectively more consistent and rational. The special diagrams anticipated for concrete (parabola-rectangle, bi-linear) are not to be used. In this case the structural analysis may be done with a simultaneous verification of sections in the calculation process. This has the advantage that it allows fusion of the two stages of justification, and hence the calculated "collapse load" may be directly obtained.

VII.3. <u>General Eurocode 2 part 2 method of non-linear analysis</u>

This method is described in clauses [EC2-2 5.7(105) and annex PP].

It is useable in first and second order analysis, and is based on two major points:

- use of specific stress-strain diagrams of materials,
- a safety format defined by the use of a global safety factor applied to a calculated collapse situation.

VII.3.1.Laws for materials

The partial factors of materials are modified, on the one hand to have the performance laws of materials nearer to their physical performance, and on the other hand by adjusting them, to obtain a more homogeneous security between the concrete and the steel. This gives materials with a little better performance than those obtained with the normal security level experienced. These laws are then used for the structural analysis, i.e. to obtain stresses and also to determine the strength of the sections.

The stress-strain diagrams for concrete and steel are shown in the following figures together with the values of their characteristic variables.

It may be noted that in place of design stresses f_{cd} and f_{yd} these values are multiplied by the same factor 1,1 γ_s and used in the new diagrams.

3.1.0.a) Law of concrete performance

This is a Sargin-type law, illustrated by the following figure:







Fig./Tab.VII.(3): Concrete behavior law with enhanced characteristics

Contrainte	Stress
Loi de type Sargin	Sargin-type law
Déformation relative	Strain

whose equation is written:

$$\sigma_{c} = \gamma_{cf} f_{ck} \left[\frac{k \left(\frac{\varepsilon_{c}}{\varepsilon_{c1}}\right) - \left(\frac{\varepsilon_{c}}{\varepsilon_{c1}}\right)^{2}}{1 + (k - 2) \left(\frac{\varepsilon_{c}}{\varepsilon_{c1}}\right)} \right]$$

where

strain of concrete $\epsilon_{\rm c}$

$$\begin{split} k &= \frac{1,05 \ E_{cm} \ |\epsilon_{c1}|}{\gamma_{cf} f_{ck}} \\ \epsilon_{cl} (0/_{00}) &= \min \Bigl(0,7 (f_{ck} + 8)^{0.31} ; 2,8 \Bigr) \qquad \text{strain at peak of stress} \\ \epsilon_{cul} (0/_{00}) &= \begin{cases} 3,5 \ \text{pour } f_{ck} < 50 \text{MPa} \\ 2,8 + 27 \biggl[\frac{98 - (f_{ck} + 8)}{100} \biggr]^4 \ \text{pour } f_{ck} \ge 50 \text{MPa} \end{cases} \qquad \text{ultimate strain} \\ \gamma_{cf} f_{ck} &= 1,1 \frac{\gamma_S}{\gamma_C} f_{ck} \approx 0,85 f_{ck} \qquad \text{design compressive strength value of concrete} \\ E_{cm} &= 22000 \biggl(\frac{f_{ck} + 8}{10} \biggr)^{0,3} \qquad \text{secant modulus of elasticity of concrete} \end{split}$$

$$\frac{3}{2}$$
 secant modulus of elasticity of concrete

3.1.0.b) Behavioral laws of reinforcing steels

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It is the bi-linear diagram with an inclined top branch defined by parameters $1.1 \times f_{yk}$ and $1.1 \times k \times f_{yk}$ as illustrated in the following figure:



Fig./Tab.VII.(4): Behavioral laws of steels of reinforced concrete

Contrainte	Stress
Loi bilinéaire avec branche inclinée	Bi-linear law with inclined branch
Déformation relative	Strain

It is constructed with the limiting coordinates of the two branches (ε, σ)

- Elastic branch $\left(\epsilon_{yd} = \frac{1.1 f_{yk}}{E_s}; 1.1 f_{yk}\right)$
- Upper inclined branch $(\epsilon_{uk}; 1, 1 k f_{yk})$

with

 ϵ_{uk} strain value under maximum load.

k minimum value of $(f_t / f_y)_k$ for class B $\epsilon_{uk} \ge 5.0\%$ and $1.08 \le k$

uk — —	for class C	$\epsilon_{uk} \ge 7.5\%$	and	$1.15 \le k < 1.35$
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3.1.0.c)Behavioral laws of prestressing steels

In an analog representation it is a bi-linear diagram with inclined top branch defined by the parameters $1,1 \times f_{p0,01k}$ and $1,1 \times f_{pk}$ that is to be used

VII.3.2.Safety format and principle of method

Calculations to be carried out for a structure are done in a totally classical manner. The only peculiarity consists of proportionately incrementing the loads of the ULS combinations (called project loads), until the calculated




collapse of the structure is reached. The new safety format involves fixing a margin in relation to this "ultimate" level of loads q_{ud} by an overall safety factor. The structure is considered as well-dimensioned if this reduced load level gives the nternal forces and moments values of those obtained with the project loads.

The verification criterion is illustrated by the three symbolic inequalities given in [EC2-2 5.7(105)], [(5.102 aN), (5.102 bN) and 5.102 cN]. The three forms of writing come simply from the more or less explicit way of taking into account the different partial factors linked to the uncertainties of the model.

In practice the values of the partial factors, used in the majority of cases for loads and materials, are supplied in their form (γ_F et γ_M). Thus they already contain the partial factors that cover the model uncertainties linked to the loads and the resistances [EC0 6.3(6.2b) et (6.6b)]. The general verification criterion $E_d \leq R_d$ [EC0 6.4.2(6.8)] is thus expressed in the form of the expression [(5.102bN)] shown below:

 $\underbrace{E(\gamma_{G}G + \gamma_{Q}Q)}_{\text{effetdesactions}} \leq R\left(\frac{q_{ud}}{\gamma_{O'}}\right) \qquad \text{Inégalité(5.102b)}$

The designer's attention is drawn to the fact that the general verification criterion $E_d \leq R_d$ does not exactly reflect the format of verifications in the case of a buckling. This was indicated in Eurocode 0 [EC0 6.4.2(3)P]. In this case, and taken in its broadest sense, the term "resistance" and its associated symbol R are taken to represent the internal forces and moments that correspond to a given load level; Eurocode 2 part 2 chooses it as being equal to that of the calculated collapse load reduced by a coefficient of 1.27.

Of the three verification criteria proposed by Eurocode 2 part 2 this guide adopts thus one only, the simplest and most practical since it does not bring in the partial factors for model uncertainties (this is also, moreover, what is used in the general method of Eurocode 2 part 1-1). For more details and explanations concerning the choice of the "good" inequality the designer should refer to Apendix VI.

VII.3.3.Practical application of the method

The first thing is determining the initial geometric imperfections from the first modes of elastic buckling and by determining the significant loading cases in the direction of buckling considered (see example of an arc given in V). In the case of vertical elements, the geometric imperfections may be represented by an inclination [EC2-2 5.2 (105)]. In the case of isolated elements, it may be practical to use an eccentricity or a transverse load calculated from the previous inclination [EC2-1-1 5.2(7)].

The process for calculating internal forces and moments and verifying the structure may then be carried out (to simplify the study it is described from the finished state of the structure under permanent loads, obtained in a classic manner, all creep carried out)

3.3.0.a) Modalities of load incrementation

Eurocode 2 gives no complete information on the manner of incrementing the loads. It is only said that the level of the loads will be increased from their service values to reach that of the basic ULS combination in a similar calculation step. The incrementation process will continue until the ultimate resistance of a zone or the overall breakage of the structure is reached [EC2-2 5.7(105) Note 1].

The mandatory governing points are however perfectly identified, namely:



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- the situation under permanent loads G (representing G_{min} or G_{max}) that is a state of reference obtained following an appropriate detailed calculation, taking account of all creep effects.
- the service situation G + Q (Q representing symbolically the whole of the variable loads intervening in the basic ULS combination)
- the situation under basic ULS combination $[(\gamma_G) \cdot G + (\gamma_Q) Q]$ (that is called project load for simplicity)
- and finally the calculated collapse situation obtained with the project load increased by a multiplying factor.



Fig/Tab.VII.(5): Illustration of development of loads

Charges d'exploitation pondérées	Factored live loads
Le retour par [équation] ne permet pas de passer sur la	Return by [equation] does not allow passing on the
trajectoire	trajectory
Charges permanentes pondérées	Factored permanent loads

It is observed on a graph illustrating development of loads, that generally, with different values of γ_G and γ_Q , the points S (for service), U (pour ULS) and R (for collapse) are not aligned and thus the incrementation step should be different between S and U on the one hand and between U and R on the other.

A load increment between S and U may be written $\alpha_i [(\gamma_G - 1)G + (\gamma_Q - 1)Q]$, with $\sum \alpha_i$ having a value of 1.

A load increment between U and R may be written $\beta_i [(\gamma_G)G + (\gamma_Q)Q]$, with $1 + \sum \beta_i = \lambda$

The pursuit of the load incrementation from the project load may a priori be done in different ways. One could, for example, continue with the same step used to pass from the calculated ULS situation, or even increment only the non-permanent loads, which constitutes a very different option. These





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different options do not allow the expression of the collapse load as being proportional to the project load $q_{ud} = \lambda q_{ULS}$; this latter choice was retained by Eurocode 2 to express the safety level.

Collapse is reached from the load increment and it is generally impossible to exceed the peak of stress of the Sargin law. But if the calculation is done with software that allows this peak to be exceeded, the state of collapse obtained may be different. This distinction has little effect on the buckling of determinate structures. On the other hand, this may be important for the calculations of internal forces and moments redistribution, where an increase in the capacity of rotation of the sections increases the possibility of redistribution. The cases concerned by these differences are however quite rare.

When it is not known a priori if G has a favorable or unfavorable effect relative to buckling, two calculations should be done, one with $\gamma_G = 1$ and the other with $\gamma_G = 1.35$, linked with G_{min} and G_{max} respectively The load increments are thus to be adapted accordingly.

3.3.0.b)Analysis of results

If the development of internal forces and moments (N, M) of a transverse section of the structure according to the load applied is represented by a two-dimensional figure and if the field of resistance (a) of the section under study, constructed with the same behavior laws of the materials is used for the structural analysis, the following figure will generally apply:



Fig./Tab.VII.(6): Progress of internal forces and momentses (N, M) according to successive loadings to point A (N_A, M_A) obtained with the collapse load and curve of interaction (a) of a section.

Instabilité d'ensemble ou rupture localisée sur une	Overall instability or localized breakage of another
autre section	section
Rupture de section	Breakage of section



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Chemin de chargement	Loading path
chemin de chargement	Douding put

It is stated that the curve (N, M) terminates at the end of calculations, at a point A, generally situated inside the field of resistance of the section studied. But it may also terminate with a point A found on the curve (a).

Meanwhile, this curve passes successively by the points (N_s, M_s) of the service situation and (N_{ULS}, M_{ULS}) of the design ULS situation.

When point A is on curve (a), the calculated collapse of the structure occurs by local breakage in the section studied. This section is the critical section and its limit of resistance has been reached.

Identification of the critical structure in which local collapse occurs constitutes one of the difficulties of application of the method.

When point A is at the interior of the field within the curve (a), there are then two apparent cases:

- Either it is still a localized breakage but one that occurred in a section other than that studied: for this section, that is not the most stressed, the actions effects obtained at the end of calculations are obviously removed from the field of resistance.
- Or it is an overall instability of the structure, and in this case it should be verified that all the points A of all the sections studied are found at the interior of all associated curves (a). This criterion will be moreover the only way to identify the method of breakage of the structure by overall instability.

It may in fact be known that there has been overall instability when it is **certain** that the critical section has been studied and that its point A is at the interior of the curve (a).

3.3.0.c)Verification of safety

With application of the criterion (5.102b) specified by the guide, implementation and control in relation to safety are greatly simplified.

Verification relative to safety is assured as soon as $\lambda \ge \gamma_{O'}$

This is illustrated in the figure below where the development is represented of the determining parameter (generally the bending moment) according to the level of loading.

The loading path passes by the points S (obtained with the service load), U (obtained with the project load) and terminates at a point A obtained with the load q_{ud} , the maximum load reached and called the calculated collapse load. Control relative to safety is satisfied if the point D obtained with the loading $q_{ud} / \gamma_{0'}$ is situated on the loading path beyond the point U obtained with the project load.







Fig./Tab.VII.(7): Verification of safety – Combination (q, M)

VII.4. Summary and comparison of different methods

The two simplified methods presented in a detailed manner in [Appendix VI], are based on the estimation of a curvature 1/r or on the estimation of a nominal rigidity EI and give safe results in relation to the two general methods. Their field of application is restricted, essentially isolated elements of constant section, reinforcement included. They nonetheless allow relatively slender elements to be dealt with and are useful for quick validation of a minimum reinforcement or a structure in project phase. They are however very conservative and it is recommended that the general Eurocode 2 part 1-1 method be used in the case of a very slender structure to obtain more realistic internal forces and moments to better optimize the material to be put in place (e.g. sections of reinforcement and effects on the dimensioning of a pier foundation)).

The general method of Eurocode 2 part 1-1 is an application rather similar to that of previous regulations and hence requires no particular comment. It is recalled that it offers a better alternative in suggesting the use of the same set of stress-strain diagrams for the analysis and for the verification of sections. This represents a real simplification compared to previous regulations.

Eurocode 2 offers the general method of part 2 as an alternative to the general method of part 1-1. Its use presents two major relevant points:

The first is to highlight the safety margin that exists between the project load (basic ULS combination load) and the calculated load that leads to collapse of the structure. In effect the method leads to exceeding the project load to cause collapse. The level of safety desired is then defined in relation to this collapse load and is fixed at the start by the method.

The second relevant point of the method is that it allows one to know if the structure reaches the calculated collapse by breakage of a section or by overall instability. This knowledge may be useful to adjust its redimensioning, locally, if there is a section breakage (e.g. strengthening of reinforcement) or in a more global way, if there is a buckling problem (action on structure's slenderness ratio)

The general Eurocode 2 part 2 method has the same theoretical complexity as the general method of Eurocode 2 part 1-1. It requires practical application modalities that are perhaps a little more elaborate, but certainly allows a keener analysis in the case of overall instability.

In conclusion, the general method of Eurocode 2 part 1-1 may be used a priori: it is well known and it is proposed that the general method of Eurocode 2 part 2 be used in cases where additional information from this method is useful to the designer. In any event, it is suggested that the appropriate method be specified in the technical clauses of the contract for all special projects.

Sétra

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Chapter 7 – Justification at SLS



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CHAPTER 7 -JUSTIFICATIONS AT SLS



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The justifications at SLS aim to ensure for all structures the aptitude for service required during the design working life chosen. They also contribute to protection relative to damage that might harm the durability of the structure.

In principle, the major justification rules of a structure at the service limit states according to Eurocode 2 are near to those used previously in France: stress limitations, control of cracking in tensioned and/ or shear zones and limitation of deformations.

The major novelty is in the expression of control of cracking: instead of a limitation of stresses of bonded steels, it is done by a conventional calculation of crack openings. This allows a unified approach between reinforced concrete and prestressed concrete.

Further, Eurocode 2 part 2 gives a method for control of cracking caused by shear in the webs in the case where this verification is necessary.

In the case of concrete highway bridges the limitation of deformations is in general non-dimensioning. In this guide only two aspects of stress limitation and control of cracking will be dealt with. Reference will be made to Annex VII for examples of detailed calculations.

Finally, the application of previous regulations allowed a dimensioning of the prestress thanks to a precise definition of three classes of justification. With Eurocode 2 the situation is more vague, and it appears necessary to give information on the subject to the designer.

Eurocode 2 gives minimum conditions to respect. Certain may give some conditions on prestress, others not. But the definition of the amount of prestress is up to the designer and a good design is not necessarily one that sticks closest to the limits fixed by Eurocode.

I.STRESS LIMITATIONS

I.1. Compressive stresses in concrete

I.1.1.In persistent design situations

It is recommended that the compressive stress in concrete be limited to 0,45 f_{ck} under a quasi-permanent combination of actions, mainly to limit the effects of creep (excessive deviation, large indeterminate effects). This allows the use of linear creep models [EC2-1-1 3.1.4(4); 7.2(3)]. In the opposite case it is advisable to consider a non-linear creep.

In the absence of containment, the compressive stress in the concrete should be limited to 0,6 f_{ck} under characteristic combination in the parts exposed to environments of classification XD, XF et XS [EC2-2 7.2(102)]

In general, this limitation will also be applied to the overall engineering structures regardless of the environment classification. It should be remembered this limitation allows exemption from fatigue verification of compressed concrete [EC2-1-1/AN].

I.1.2. During construction







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Similarly the compressive stress in the concrete should be limited to 0.45 $f_{ck}(t)$ during construction [EC2-1-1 3.1.4(4); 5.10.2.2(5)].

Clause 5.10.2.2(5) specifies that non-linear creep should be taken into account if the compressive stress permanently exceeds 0.45 $f_{ck}(t)$. At the moment a tendon is tensioned, this value may be temporarily exceeded. The maximum acceptable compression value is specified below.

The compressive stress in the concrete should be limited to 0.6 $f_{ck}(t)$. For pretension it may be increased to 0.7 $f_{ck}(t)$ subject to justification by tests or by the experience that longitudinal cracking is prevented [EC2-1-1 5.10.2.2(5)].

This limitation should be applied regardless of the environment classification.

I.2. <u>Tensile stresses in reinforcement</u>

I.2.1.Reinforced concrete bars

The limitations to respect are as follows: [EC2-2 7.2 (5)]:

 $\sigma_s < 0.8 \, f_{yk}$ under characteristic SLS combination

 \circ (unless the structure is subjected solely to imposed strains, for example due to delayed shrinkage, in which case the limit is taken to f_{yk})

The value of 0.8 f_{yk} is high and this stress limitation in reinforcing steels under characteristic SLS is not generally dimensioning.

I.2.2. Unbonded prestress

When all the prestress is unbonded, the stress increase produced in the prestressing steels at SLS due to the effect of over-loads is negligible. The rules are thus identical to those of the case of reinforced concrete.

It is the case particularly of transverse bending of wide box bridges with unbonded transverse prestress.

I.2.3.Bonded prestressing tendons

 $\sigma_{pm} < 0.80 \text{ f}_{pk}$ under characteristic SLS combination [EC2-1-1/AN 7.2(5)]

The previous French practice consisted of limiting solely the stress increase of the prestressing steels regardless of the level of prestress under permanent loads. The Eurocode adopts more logically a criterion of non-plasticizing of the reinforcement. The more the reinforcement are tensioned under permanent loads, the lower is the stress increase authorized under operating loads.

With the value 0.75 recommended by Eurocode 2, this criterion could lead to a reduction in the initial tension of the tendons, particularly for tendons with few instantaneous losses. The value was increased to 0.8 in the national annex of Eurocode 2 part 1-1 in order to prevent this reduction of initial tension.



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II.CRACK CONTROL

II.1.<u>Principle</u>

The principle adopted by Eurocode 2 for control of cracking consists of defining [EC2-2/AN Tab.7.101NF]:

- a limit value of the design cracks width according to the exposure classification and the nature of the element considered (reinforced concrete, prestressed concrete with unbonded tendons, or prestressed concrete with bonded tendons)
- and/or a criterion of non-decompression
- that should then be verified by a calculation of crack opening and a stress calculation.

II.2. Limits of crack opening

These limits are as follows:

- Reinforced concrete and prestressed concrete elements with unbonded tendons subjected to exposure classifications XC , XD or XS

 $w_{max} < 0.3$ mm under frequent combination for exposure classification XC

 $w_{max} < 0.2mm$ under frequent combination for exposure classification XD or XS

The choice of the quasi-permanent combination is well adapted to buildings, where quasipermanent loads represent a large part of the loads (the factors ψ_2 are in general not zero). In structures, on the contrary, the factors ψ_2 are generally zero. The frequent combination appears thus as more pertinent to evaluation of the crack openings, and it is the choice that has been made in the national annex of Eurocode 2 part 2.

In spite of this correction, the limitation of the crack opening under frequent load will not generally be dimensioning for exposure classification XC. It is particularly the case for transverse bending of the slabs of composite bridges or box girder bridges. Strict application of this rule may lead to relatively high stresses under traffic loads, and fatigue of reinforcing steels may thus be dimensioning. The national annex of Eurocode 2 part 2 exempts from fatigue calculation reinforced concrete structures for which $\sigma_s < 300$ MPa under characteristic SLS. This last condition will be generally dimensioning for structures in exposure classification XC, whereas the limitation of crack opening may be dimensioning for structures in exposure classification XD or XS.

prestressed concrete elements with bonded tendons subject to XC exposure classifications

 $w_{max} < 0.2$ mm under frequent combination

with an accompanying non-decompression control

 $\sigma_c > 0$ in cover zone under quasi-permanent combination

It will be recalled that a fatigue verification of reinforcement and prestressing steels is necessary for prestressed structures that are cracked under frequent SLS.

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prestressed concrete elements with bonded reinforcement subjected to environment classifications XD or XS

 $\sigma_{\rm c}\!>\!0$ in cover zone under frequent combination





The cover zone is defined by a distance of 100mm to the tendon or to its ducts. This condition does not impose a cover of 100mm for prestressing tendons: The Eurocode only demands that the concrete at less than 100mm from the duct, if such there is, be compressed.

It will be recalled that the non-decompression of concrete under frequent loads allows exemption from a fatigue calculation [EC2-2 6.8.1(102)].

II.3. <u>Methods of calculating crack openings</u>

II.3.1. Approach according to European text

In the European text, control of cracking may be done in two ways:

- "Direct" method: control of cracking is considered as assured if the crack width calculated by the recommended method [EC2-1-1 7.3.4], is less than the given limit value, and/or if the non-decompression rule is verified. A minimum section of reinforcement is however required [EC2-1-1 Expr.(7.1)].
- "Simplified" method: Eurocode 2 part 1-1 gives a simplified method for this verification in 7.3.3, for a certain number of hypotheses, particularly the presence of a minimum reinforcement greater than the strict minimum required.

Calculation of stresses in the steels in view of the crack opening calculation is always carried out on a cracked section. The equivalence coefficients to be used are given in Chapter 3-III.1 of this guide.

The two methods are used and presented in appendix through examples of calculation.

It should be stated that the field of use of the simplified Eurocode method is more restricted than that of the direct method: to use the simplified method, a minimum reinforcement is necessary, calculated with values of σ_s read from the tables (and not with f_{yk}), a function of the diameter or the spacing of the steels [EC2-1-17.3.3]; corrections given by the formulae (7.6N) and (7.7N must be made. Finally, the tables generally give unfavorable results compared to direct calculation, particularly for large diameter steel for which the rates of effective reinforcement are much greater than those that were used in establishment of the tables.

For all these reasons, the direct method has been favored in most of the calculated examples in this guide, particularly for verification calculations. The use of the tables of the Eurocode simplified method maintains a significance in the steel dimensioning phase.

The direct calculation method of crack openings has itself certain limitations. It is adapted to rectangular sections in monoaxial bending (it is on the basis of tests on this type of section that calibration of the formulae was done). Moreover, it was not calibrated for determination of crack openings on thick elements.

There are thus problems with generalization of these formulae with more complex cross-section shapes, with biaxial bending sections or with thick structures.

II.3.2.Approach used in the national annex of EN1992-2

In order to solve the difficulties mentioned in the last paragraph, another method has been proposed in the national annex of EN1992-2: it is a more 'rustic' method, but is applicable in all situations. The method proposed is the following:

• limiting of spacing between steels to $5(c+\phi/2)$,





• and limiting of stress in reinforcing steels to 1000 w_{max} in the case of bent sections (i.e. with one face in compression, one in tension, with no transverse cracking), or to 600 w_{max} in the case of sections totally in tension.

In these expressions, w_{max} is in mm and the stress limit obtained is in MPa.

This 'rustic' method of the national annex of Eurocode 2 part 2 may be applied in the general case. If it is desired to optimize the reinforcement dimensioning, the direct calculation formulae may be applied when possible, particularly if the sections are more or less rectangular, or in slightly deviated bending, with standard thickness ranges and relatively regular reinforcement.

For thick structures, those in biaxial bending, or those with complex patterns or reinforcement, the use of the 'rustic' method is recommended.

Note: The direct calculation may lead to larger steel sections, but nonetheless the 'rustic' method is safe.

II.4. Minimum cracking reinforcement

II.4.1.Principle and application to rectangular sections

Regardless of the calculation of crack openings, it is advisable to anticipate a minimum reinforcement in the zones likely to be tensioned under characteristic SLS. The writing of clause 7.3.2(4) is ambiguous and the national annex has clarified the meaning:

- For concrete structures prestressed by post-tension the minimum reinforcement is required in all sections where, under the characteristic combination of actions and for the characteristic prestress value, the concrete is tensioned (i.e. $\sigma < 0$).
- For structures with bonded wires, no minimum reinforcement is required in sections where the absolute value of the tensile stress of the concrete is less than 1,5 $f_{ct,eff}$.

In a reinforced concrete beam, the minimum reinforcement should always be applied.

The principle of the minimum reinforcement calculation is stated in [EC2-1-1 7.3.2(1)P]: the bonded reinforcement placed must be capable of withstanding, without plasticizing, the tensile stresses in the concrete at the moment of cracking.

An application modality is given in [EC2-1-1 7.3.2(2)] in the case of rectangular sections in composite bending:

$$\mathbf{A}_{s,min} \times \mathbf{f}_{yk} = \mathbf{k}_c \times \mathbf{k} \times \mathbf{f}_{ct,eff} \times \mathbf{A}_{ct} \qquad [\text{EC2-1-1 Expr.}(7.1)]$$

It is generally advisable to take $f_{ct,eff} = f_{ctm}$ [EC2-2 7.3.2(102)]. However, when it is certain that cracking will occur solely in early stages (e.g. case of a beam subjected solely to delayed strains), it is possible to take a reduced value $f_{ct,eff} = f_{ctm}(t)$, without however going below 2,9 MPa [EC2-2 7.3.2(105)].

The value of k_c was calibrated relative to the general principle stated above. The value is exact in the case of simple bending, and it is safe in the case of composite bending.

In the case of a rectangular section in simple bending, $k_c = 0,4$. The minimum reinforcement is thus

$$A_{s,min} = 0.4 \times f_{ctm} \times 0.5 \times b \times h / f_{yk}$$

Or, with d = 0.9 h:



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 $A_{s,min} = 0.23 \times (f_{ctm} / f_{yk}) \times b \times d$

This reinforcement is slightly less than the minimum reinforcement given for the beams in section 9 of Eurocode 2 part 1-1 (coefficient 0.23 instead of 0.26). In both cases it is a minimum reinforcement based on the same principles. To clarify this, the national annex of EN1992-1-1 specified that the reinforcement in section 9 was applicable for elements in reinforced concrete elements, and that the minimum reinforcement in section 7 was applicable to prestressed concrete elements.

The formula given in Eurocode 2 contains an additional term (coefficient k). Its value should generally be taken as 1.0, except for wide or high parts subjected solely to imposed strains (see appended example of application).

It should finally be mentioned that the expression (7.1) is written in Eurocode 2 part 1-1 in a more general way with σ_s instead of f_{yk} . This is because this expression may also be read backwards, to calculate a stress:

$$\sigma_{s} = k_{c} \times k \times f_{ct,ef} f \times \frac{A_{ct}}{A_{s}}$$

This calculation allows estimation of the stress in the reinforcing steels of a cracked chord subject to imposed strains. An example of use is appended.

It is thus the value $\sigma_s = f_{yk}$ that should be used to calculate the minimum reinforcement. This point has moreover been clarified by the national annex. It is only when one wishes to use the tables in the indirect method (7.3.3) that it is advisable to adopt a smaller value of σ_s , to comply with the hypotheses according to which these tables were established.

II.4.2. Generalization to any cross-sections

Eurocode 2 shows how to generalize the calculation of the minimum reinforcement to rectangular sections by parts (in practice: box- or T sections). The principle is as follows:

- The sections are divided into rectangular elements that are either slabs or webs: the modalities of cutting are shown in [EC2-2 7.3.2 fig.7.101]
- The cracking moment M_{fiss} , is determined, i.e. the moment creating a tensile stress f_{ctm} in the end fiber, assuming the axial exterior stress N is unchanged (in a prestressed beam, N is generally taken as = $P_{k,inf}$)

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For a section where N and M both vary (case of a column for example), it may be safer to determine the cracking stresses by considering an excentricity M/N equal to that of the most unfavorable service situation.

- \circ the stress diagram is drawn in the section under this couple (N, M_{fiss})
- \circ in a web-type element, the minimum reinforcement is calculated according to the general formula by taking σ_c equal to the stress at the web's center of gravity.
- in a flange-type element, the minimum reinforcement is obtained by the relationship:

 $A_{s,min}f_{vk} = max[0,9kF_{cr};0,5kf_{ctm}A_{ct}]$





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The first part of the equation balances 90% of the tensile stress in the chord before cracking; it is considered that the remaining 10% goes back to the webs by changing the lever arm. The second part of the equation ensures a minimum reinforcement of the chords allowing balancing of a 'local' cracking moment (one face under tension at f_{ctm} and one face under stress nil).

Generalization of the calculation of the minimum reinforcement for any cross-sections may be done by reverting to the principle stated in $[EC2-1-1 \ 7.3.2(1)P]$: it is verified that under the cracking stresses the steels in place do not plasticize.

II.5. Control of cracking caused by shear

Eurocode 2 part 1-1 does not anticipate specific verifications of the resistance to shear at SLS. It is supposedly assured by constructive provisions [EC2-1-1 7.3.3(5)].

On the other hand, Eurocode 2 part 2 anticipates in clause [EC2-2 7.3.1(110)] a cross-reference to an annex QQ (informative, made normative by the national annex) that gives a method allowing, where necessary, prevention of cracking caused by shear. It essentially concerns the webs of prestressed concrete structures and does not generally apply to reinforced concrete sections.

Cracking of the webs is allowable for reinforced concrete sections, but should not be excessive. For that it is advisable either to verify explicitly the cracks opening and to check fatigue due to shear stresses, or to adopt 'reasonable' inclinations of the struts at ULS, not moving far from the natural orientation of the cracks at SLS (45° in standard cases). Previous French regulations put the struts at 45° with no other verification. A lower limit corresponding to $\cot\theta = 1.5$ was proposed in the national annex [EC2-2/AN 6.8.1(102)] [see Chapter 6-II.2.2].

II.5.1. Principle of justification

The principle used in the annex QQ consists of determining the maximum principal tensile stress in the concrete and verifying that it is less than a concrete tensile strength to be determined depending on the applied stresses, to ensure no cracking under shear.

It is obvious that the verification above should be done taking into account all stresses caused by a combination of bending, shear and torsion. The verification is to be done all over the web, or in general at the center of gravity and at the gussets.

Although the text does not specify it, the justification is to be done under the characteristic SLS combination.

Non-cracking criterion

The criterion compares the greatest major tensile stress σ_1 to f_{ctb} , concrete tensile strength at web level, in a state of bi-axial stress before cracking, and given by the expression:

$$f_{ctb} = \left(1 - 0.8 \frac{\sigma_3}{f_{ck}}\right) f_{ctk;0,05}$$
 [EC2-2 Expr.(QQ.101)]

where: σ_3 is the maximum principal compressive stress (positive value)

 f_{ck} is the characteristic compressive strength of concrete

 $f_{ctk,0.05}$ is the fractile 5% of the tensile strength of concrete

In standard cases where there is no bi-axial compression (no vertical prestress in the webs for example) the state of stress is characterized by the tensor:





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$$\begin{pmatrix} \sigma_x & \tau \\ \tau & 0 \end{pmatrix}$$

The principal stresses σ_1 and σ_3 are then calculated by the expressions:

$$\sigma_1 = \frac{\sigma_x}{2} - \sqrt{\left(\frac{\sigma_x}{2}\right)^2 + \tau^2}$$
$$\sigma_3 = \frac{\sigma_x}{2} + \sqrt{\left(\frac{\sigma_x}{2}\right)^2 + \tau^2}$$

The expressions above are applicable if $\sigma_x \ge 0$. When a verification is necessary at a point where $\sigma_x < 0$, σ_1 and σ_3 may be calculated by taking $\sigma_x = 0$; the verification is thus written

$$\tau \le f_{ctb}, \text{ or } \tau \le \frac{f_{ctk\,0,05}}{1 + 0.8 \frac{f_{ctk\,0,05}}{f_{ck}}}$$

This comes back to controlling web cracking under tangential stresses regardless of the longitudinal tension, which is balanced directly by the longitudinal reinforcement. It is not explicitly stated in annex QQ but this principle is found in other annexes (especially annex F), and in the previous French practice (BPEL).

If the criterion $\sigma_1 > -f_{ctb}$ is verified (σ_1 maximum principal tensile stress), then the section is not cracked under shear stress. No shear reinforcement is to be anticipated at SLS apart from the minimum reinforcement.

If the criterion is not verified, the annex QQ requires that web cracking be controlled according to the methods in section 7 of Eurocode 2 part 1-1 [EC2-1-1 7.3.3 or 7.3.4 and 7.3.1] taking into account the deviation between the direction of the principal stress and the directions of the reinforcement. However, reference to these clauses gives no practical information on doing the complete calculation for crack opening.

It thus appears very desirable to dimension the webs so that they don't crack at SLS.

Moreover, having webs too thin may cause other problems (reduction of lever arm of reinforcing steels relative to transverse bending, difficulties of pouring concrete, reduction of torsional stiffness, etc). It is advisable to take account of these phenomena in the choice of web thicknesses, and not to be limited to the sole SLS criterion of annex QQ of EN1992-2.

The equation (QQ101) may express the shear stress limitations in another way, more familiar to designers:

$$\tau_{adm} = \sqrt{\sigma_{x} \times \sigma_{y} - \frac{5f_{ck} \times f_{ctk;0,05} \times (\sigma_{x} + \sigma_{y} + f_{ctk;0,05}) \times (4\sigma_{x} + 4\sigma_{y} - 5f_{ck})}{(5f_{ck} + 4f_{ctk;0,05})^{2}}}$$

with $\sigma_x = axial longitudinal stress$

 τ_{adm} = acceptable shear stress.

 $[\]sigma_y$ = axial transverse stress





III.RULES FOR PRESTRESS DIMENSIONING

The justifications given previously should be used in verification of structure, but do not always suffice for a correct dimensioning of prestress.

III.1.<u>Case of cast-in-place structures</u>

This question rises especially for prestressed structures in environment classification XC3 or XC4, for which the tensions in the concrete under frequent and characteristic loads are not limited. A dimensioning of the prestress on the basis of the only criterion of quasi-permanent non-decompression will lead to small quantities of prestress and large quantities of reinforcing steels, a dimensioning that may turn out to be non economical.

The project designer must thus in general use additional dimensioning rules in particular for beam-type structures where partial prestressing is not economical. These rules may be as follows:

 $\sigma_c > 0$ under frequent combination in the whole section

or

 $\sigma_{\rm c}$ > -f_{ctm} under characteristic combination in the whole section

One could generally use only the more favorable of these two rules above.

These two limits appear in the Eurocode, since the first one exempts from fatigue calculation, and the second one allows a calculation of stresses in the non-cracked section.

These rules are to be taken as recommendations for dimensioning except in the case where a structure operating in partial prestressing is more economical.

When these rules are added to the project, one may choose to refer only to the mean prestress force.

III.2.<u>Case of prefabricated segments</u>

The justification rules of prefabricated segments at SLS are given in standard EN 15050, G.2.3.1: under characteristic combinations, with the characteristic values of prestress, the concrete section should remain entirely compressed (full prestressing).

III.3. Rules during construction

The rules given above should be adapted for the construction stages for several reasons: the idea of frequent combination does not exist during construction, and the limit states relative to durability are not as pertinent as during service, particularly for short-term phases.

On the other hand, it will be recalled that the ULS should be verified at all stages of construction.

It is thus advisable to define specific rules for the SLS verifications during construction. These rules are not shown explicitly in Eurocode 2 part 1-1. Conversely, Eurocode 2 part 2 devotes a special section to it, no. 113.

The table below shows how to adapt the rules for a few special examples, using the principles of section 113:

SLS rule du	ring design working life		SLS rule during construction
$\sigma_c > 0$	under frequent SLS	σ_{c} > - $f_{ctm}(t)$	under probable construction combination
$\sigma_{\rm c} > 0$	under QP SLS	σ_{c} > -f _{ctm} (t)	under probable construction combination
$w_k < 0.2mm$	under frequent SLS	$w_k < 0.2mm$	under probable construction combination

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$\sigma_{s}{<}0.8~f_{yk}$	under characteristic SLS	$\sigma_{s}{<}0.8~f_{yk}$	under characteristic construction combination
$\sigma_{\rm c} < 0.6~f_{\rm ck}$	under characteristic SLS	$\sigma_{\rm c} < 0.6 \ f_{\rm ck}(t)$	under characteristic construction combination

In the above table, the tensile stress limits in construction are to be respected in the tendon cover zone. Outside this zone, higher tension is acceptable and a verification of crack opening is carried out.

It is advisable to set these rules during project establishment, since they may have an effect on dimensioning of the cross-section and on the prestress. These rules should be shown in the contract documents. The mean value of prestress may be chosen for use with these rules.



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Chapter 8 – Detailing rules relative to reinforcements



CHAPTER 8 - DETAILING RULES RELATIVE TO REINFORCEMENT





This chapter essentially concerns section 8 of Eurocode 2 "detailing of reinforcement and prestressing tendons - general" of parts 1-1 and 2. This chapter analyses some of the details shown in this section 8. It is not exhaustive and it will not replace reading of the Eurocode.

Paragraphs I to VI are devoted to single bars and paragraph VII to bundles of bars.

Some of the subjects dealt with in section 8 of Eurocode 2 are not taken up in this chapter. It is particularly the case of the following points:

- Welded mesh fabrics
- Anchorage by welded bars
- Prestress

Other parts of Eurocode 2 also show the provisions corresponding to minimum requirements. These are dealt with in the corresponding chapters of this guide.

It should be noted that, in compliance with clause 8.1 of part 1-1 of Eurocode 2, section 8 deals only with ribbed reinforcement, mesh and prestressing tendons.

As regards reinforcement of the 'smooth round' type, and in the absence of new texts on the subject, it is well to keep the constructive provisions shown in the BAEL 99 and in standard NF A35-027of January 2003.

The reinforcement used should be in compliance with standard EN 10080.

I.SPACING OF BARS

The minimum clear distance between bars is the same horizontally and vertically.



e_h, e_v: clear distances, horizontal and vertical respectively Fig./Tab.I.(1): Clear distances between single bars

These clear distances should verify: $e_h, e_v \ge e_{mini} = sup \{ \phi; (d_g + 5mm); 20mm) \}$

with d_g dimension of largest aggregate and ϕ bar diameter





II.PERMISSIBLE MANDREL DIMAETERS FOR BENT BARS

II.1.<u>General</u>

In compliance with clause (3) of 8.4.1 of Eurocode 2 part 1-1, bends and hooks do not contribute to compression anchorages.

This chapter applies to all parts of bent, tensioned bars be they those:

- of bends, hooks or loops of reinforcement anchorages,



Fig./Tab. II.(1): Reinforcement anchorages

- bends of continuous bent bars (bent-up bars and other bent bars),



Fig./Tab.II.(2): Elbows of continuous reinforcement (bars turned up and other bars bent)

- stirrups, joist hangers and frames (including their anchorages).

Coudes	bends
Console	cantilever beam
Poteau	post



Fig./Tab.II.(3): Link, stirrups and rectangular hoops

Épingle	link
1 0	





Étrier	Stirrup
cadres	Rectangular hoops

The permissible mandrel diameter for bent bars depends on two criteria: no damage to reinforcement and no concrete failure.

II.2. Criterion of no damage to reinforcement

The corresponding clause is in [EC2-1-1 8.3(2)]

In the general case of bent bars with no welded reinforcement, the minimum mandrel diameter relative to the no damage to reinforcement criterion is given in the following table:

φ: bar diameter	φ _m : Minimum mandrel diameter			
φ ≤ 16 mm	4 φ			
φ > 16 mm	7φ			

Fig./	Га b.II.(4) :	Minimum	mandrel	diameter	relative	to no	damage	to reinfo	rcement

II.3. Criterion of no concrete failure

The corresponding clause is in [EC2-1-1 8.3(3)].

In compliance with the national annex of Eurocode 2 part 1-1, this criterion is not applicable to links, stirrups and rectangular hoops.

• Field of use

For a bar of diameter ϕ this criterion should be verified if at least one of the following three conditions is not filled:

- <u>condition no. 1</u>: The anchorage of the bar does not require a length more than 5φ past the end of the bend
- <u>condition no.2</u>: The bar is not positioned at the edge and there is a cross bar with a diameter at least equal to φ inside the bend
- <u>condition no.3</u>: The mandrel diameter is at least equal to the recommended values for the no damage to the reinforcement criterion.
- Formula of no concrete failure criterion

The mandrel diameter should verify: $\phi_m \ge F_{bt}\left(\frac{1}{a_b} + \frac{1}{2\phi}\right) \cdot \frac{1}{f_{cd}}$,

with F_{bt} the tensile force from ultimate loads at the start of the bend

 $F_{bt} = \alpha \times (f_{yk} / \gamma_S) \times [\pi (\phi / 2)^2]$, with α proportion of force left to anchor from the start of the bend (the proportion (1 - α) of the force is already anchored before the start of the bend)

 a_b half of the centre-to-centre distance between the bars (or group) perpendicular to the plane of the bend

(if the bar (o group of bars) is adjacent to the face of the member, take the cover plus $\phi/2$)

 f_{cd} not greater than that for concrete class C 55/67.



Attention is drawn to the fact that in general for a tensioned bar a part of the force is already anchored at the start of the bend. Taking into account this aspect allows to reduce appreciably the value of the permissible mandrel diameter in relation to the no concrete failure criterion.



Fig./Tab.II.(5): Start of bend relative to start of anchorage for a tensioned bar

Longueur d'ancrage de calcul	Design anchorage length			
Origine de la partie courbe	Start of bend			

• Application to anchorages of bent bars

◦ For anchorages of bent, tensioned bars, the no concrete failure criterion is generally satisfied with a mandrel diameter of $\phi_m \ge 10 \phi$.

Example, special classic case:

1

- centre-to-centre distance $\geq 5 \; \varphi \qquad$ - cover $\geq 2 \; \varphi \qquad$ - $f_{yk} \leq 500 \; MPa \qquad$ - $f_{ck} \geq 35 \; MPa$

- 25 % of the forces are already anchored at the start of the bend,

$$\phi_{\rm m} \ge F_{\rm bt} \left(\frac{1}{a_{\rm b}} + \frac{1}{2\phi} \right) \cdot \frac{1}{f_{\rm cd}} = \{ 0.75 \times (500 / 1.15) \times [\pi (\phi/2)^2] \} \times \{ [2/(5\phi) + 1/(2\phi)] \times 1.5/35 \} \approx 10 \phi$$

The calculation was done in the case of a non-accidental situation with $\gamma_c = 1.5$ and $\gamma_s=1.15$, the accidental case here being less unfavorable due to the ratio γ_c/γ_s which is 1.2 in an accidental case, and 1.3 if not.

More precisely, for anchorage by hooks or bends, respect of the following prescriptions exempts from verification of the no concrete failure condition:

- Cover $\geq \phi$
- Steels of classification B500B or less
- Mandrel diameter: $\phi_m = 10 \phi$
- Anchorage length in straight part (before start of bend) greater than the values shown in the table below.

Centre-	Classification of concrete								
to- centre distance	C30/37	C35/45	C40/50	C45/55	C50/60	C55/67	C60/75		
2 φ	31 ø	25 φ	20 φ	16 φ	13φ	10 φ	10 φ		



3φ	25 ø	19 ø	14 ø	10 φ	б ф	3 φ	3 φ
4 φ	21 ø	15 φ	9φ	5φ	φ	0	0
5φ	18 φ	11 φ	6φ	φ	0	0	0
б ф	15 φ	8φ	3 φ	0	0	0	0

Fig./TabII.(6): Minimum anchorage length necessary in straight length

This anchorage length in straight part is a minimum length to be respected and does not exempt from calculation of the design anchorage length l_{bd} which may give an anchorage length in straight part greater than the minimum length shown in the table above, particularly when the calculation is done with the equivalent anchorage length.

Example, for a 90° standard bend, $\phi \le 32$ mm, concrete of classification C35/45, center-to-centre distance = 2 ϕ and $\phi_m = 10 \phi$ (figure a). With $\alpha_1 = 0.7$ and the other α_i equal to 1, calculation of the anchorage length l_{bd} [Chapter 8-III] gives:

 $l_{bd} = 0.7 \times l_{b,rqd} = 0.7 \times 46 \ \phi = 32.2 \ \phi$, that is an anchorage length in straight part of:

$$l_{bd} - [5 \phi + (\pi / 2) \times (\phi_m / 2 + \phi / 2)] = 18.6 \phi.$$

In this case, it is the minimum anchorage length in straight part 25 ϕ shown in the table [Fig./Tab.II.(6)], required by the no concrete failure condition and the choice $\phi_m = 10 \phi$, that prevails.

On the contrary, for an centre-to-centre distance of 3 ϕ and a concrete of classification C40/50 (figure b), the table [Fig./Tab.II.(6)] shows a minimum anchorage length in straight part of 14 ϕ . It is thus the calculation of the anchorage length l_{bd} , that gives an anchorage length in straight length of 15,8 ϕ , that prevails.



However, according to the conditions of centre-to-centre distance and the concrete classification, mandrel diameters less than 10ϕ may be obtained by verifying the no concrete failure criterion.

• Application to bends of continuous bars

For deviations by bends of continuous bars (bent-up bars and other bent bars), the no concrete failure condition is the most often satisfied (for concretes of classification C35/45 or higher) with a mandrel diameter $\phi_m \ge 15 \phi$.

Centre-	Classification of concrete											
to- centre distance	C35/45	C40/50	C45/55	C50/60	C55/67	C60/75						
2 φ	22 φ	19 ø	17 φ	15 φ	14 ø	13 φ						



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3φ	17 φ	15 φ	13 ø	12 φ	11 φ	10 φ
4 φ	15 φ	13 ø	11 φ	10 φ	9φ	9φ
5φ	13 ø	12 ø	10 φ	9φ	8φ	8φ
6 φ	12 φ	11 ø	9φ	9φ	8φ	7φ

Fig./Tab.II.(7): Minimum mandrel diameter , for steels of classification B500B or less , according to diameter ϕ of bar, for bends of continuous bars

The calculation is done here as in the previous example, but considering 100% of the force.

III.ANCHORAGE OF LONGITUDINAL REINFORCEMENT

III.1.<u>Definitions</u>

Clause 8.4 of Eurocode 2 part 1-1 brings in 5 types of anchorage length.

 l_b basic tension anchorage length

This is the length necessary to anchor a given force by straight anchorage for a single bar.

l_{b,rqd} basic required anchorage length

This is the length necessary to anchor the force $A_s \sigma_{sd}$ by straight anchorage of a single bar assuming that the bond stress is constant and equal to f_{bd} .

(with σ_{sd} design stress of the bar at the position from where anchorage is measured)

 l_{bd} design anchorage length

This is the anchorage length for a bar taking account of its shape and its environment

- $I_{b,min}$ minimum anchorage length if no other limitation is applied
- $\mathbf{l}_{b,eq}$ equivalent anchorage length

This is a simplified formula for l_{bd} in standard cases. In the case of curved anchorages it particularly allows calculation of the anchorage length without taking account of the mandrel diameter used.

III.2.<u>Principle</u>

The principle of determination of an anchorage length includes two steps:

1- determination of $l_{b,rqd}$ the basic required anchorage length (in the hypothesis of a straight anchorage) to anchor the calculated force. It should be noted that this calculated force is not necessarily equal to the maximum acceptable force for the bar for the corresponding situation.

2- determination of l_{bd} the design anchorage length that takes account of the bar's shape and its environment. It should be noted that this design length is less than $l_{b,rqd}$ and that it is deduced from this via the application of reducing factors.

• Case of compressed bars: bends and hooks do not participate to the anchorage of compressed bars. [EC2-1-1 8.4.1(3)]. The anchorage length may be measured as shown on the following sketch:





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• Case of tensioned bars:



Basic tension and design anchorage lengths l_b and l_{bd} are measured along the centreline whatever the shape. Fig./Tab.III.(2): l_b and l_{bd} in the case of a curved anchorage

To keep anchorage length calculations to a minimum, it is well to keep the same anchorage force for all steels of a given diameter and grade: the force giving the maximum anchorage length is $A_s \sigma_{sd} = A_s f_{yk} / \gamma_s$.

It is thus advisable to do the calculation for a non-accidental situation with $\gamma_c = 1.5$ and $\gamma_s = 1.15$, the accidental case being less unfavorable because of the ratio γ_c/γ_s that is 1.2 in the accidental case and 1.3 if not.

However an accurate calculation taking account of the exact value of the design force may be carried out for example to justify a particular steel or the steels of an existing structure.

A determination by a simplified approach (equivalent anchorage length) is also possible in some cases.

III.3. Determination of the basic required anchorage length

The corresponding clauses are in [EC2-1-1 8.4.3]

III.3.1.Ultimate bond stress

Determination of the basic required anchorage length necessitates the previous calculation of the "ultimate bond stress" [EC2-1-1 8.4.2]. This, noted f_{bd} , should be sufficient to prevent a bond failure.

 $f_{bd} = 2.25 \ \eta_1 \eta_2 \ f_{ctd}$, with:

 η_2 parameter linked to the bar diameter . $\eta_2 = (132-\phi) / 100$ for $\phi > 32$ mm, 1 if not

 f_{ctd} design value of concrete tensile strength $f_{ctd} = \alpha_{ct} f_{ctk,0,05} / \gamma_C$, limited to the value for C60/75. The coefficient α_{ct} is defined in 3.1.6 of Eurocode 2 part 1-1, $\alpha_{ct} = 1$.

Sétra



 η_1 parameter depending on bond conditions and the position of the bar during concreting. It has the following values:

- \circ $\eta_1 = 1.0$ when 'good' bond conditions are obtained, i.e.:
- for all the bars whose anchorage inclination with the horizontal is between 45° and 90°, during the concreting phase [Chapter 8-Fig./Tab.III(5)].
- For the bars whose anchorage inclination with the horizontal is between 0 and 45° and which during the concreting phase are at a distance less than 250mm from the bottom fibre of the section or, for slabs thicker than 600 mm, at a distance greater than 300 mm from the upper fibre of the section [Chapter 8-Fig./Tab.III(4)].
- $\circ \quad \eta_1 = 0.7 \text{ in all other cases.}$



Fig./Tab.III.(3): Bars inclined with horizontal at an angle between 45 and 90° - Good bond conditions

Direction du bétonnage	Concreting direction
	U



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Fig./Tab.III.(4): Bond conditions for bars inclined at an angle between 0 et 45° with the horizontal

Position de l'acier dans l'épaisseur de la dalle	steel position in slab thickness
Zone de conditions d'adhérence « médiocres »	'mediocre' bond conditions zone
Zone de conditions d'adhérence « bonnes »	'good' bond conditions zone
Hauteur de la section de béton	height of concrete section

	$f_{ck}\!/f_{ck,cube}$	12/15	16/20	20/25	25/30	30/37	35/45	40/50	45/55	50/60	55/67	60/75	70/85
'Good'	<i>φ</i> ≤ 32mm	1,65	2,00	2,32	2,69	3,04	3,37	3,68	3,99	4,28	4,43	4,57	4,57
conditions	ø=40 mm	1,52	1,84	2,14	2,48	2,80	3,10	3,39	3,67	3,93	4,07	4,21	4,21
'Mediocre'	$\phi \leq 32$ mm	1,16	1,4	1,62	1,89	2,13	2,36	2,58	2,79	2,99	3,1	3,2	3,2
conditions	<i>φ</i> =40 mm	1,06	1,29	1,49	1,73	1,96	2,17	2,37	2,57	2,75	2,85	2,94	2,94

Fig./Tab.III.(5): Values of ultimate bond stress (in MPa), according to concrete strength (in MPa), bar diameter and bond conditions

As shown above, the table was established for a non-accidental situation with $\gamma_{\rm C} = 1.5$ et $\gamma_{\rm S} = 1.15$ (remember: $f_{\rm ctd} = \alpha_{\rm ct} f_{\rm ctk,0,05} / \gamma_{\rm C}$)

Example: $f_{ck} = 35$ MPa, good bond conditions, $\phi \le 32$ mm, $f_{ctk,0,05} = 2.247$ MPa $f_{bd} = 2.25 \ \eta_1 \eta_2 \ f_{ctd} = 2.25 \times 1 \times 1 \times 1 \times 2.247 \ / \ 1,5 = 3.37 MPa$

III.3.2.Basic required anchorage length

The ultimate bond stress having thus been determined, the basic required reference anchorage length may then

be calculated by the formula: $l_{b,rqd} = \left(\frac{\phi}{4}\right) \cdot \left(\frac{\sigma_{sd}}{f_{bd}}\right)$

Sétra

et autorou



(with σ_{sd} design stress of the bar at the position from where the anchorage is measured)

The table below gives the basic anchorage length necessary to anchor all the design force the bar can take and not the particular force to which it is subjected. As previously explained, the design stress value used is $\sigma_{sd} = f_{yk} / \gamma_s$.

f _{ck} / f _{ck cube}	12/15	16 / 20	20 / 25	25 / 30	30 / 37	35 / 45	40 / 50	45 / 55	50 / 60	55 / 67	60 / 75	70 / 85
φ bar (mm)												
φ ≤ 32 mm	66 ø	54 ø	47 ø	40 q	36 ø	32 φ	30 ø	27 ø	25 ¢	25 ø	24 ø	24 ø
φ =40 mm	72 φ	59 φ	51 ø	44 ø	39 ø	35 ¢	32 ø	30 φ	28 φ	27 φ	26 ø	26 ¢

Fig./Tab.III.(6): Values of basic anchorage length necessary to anchor f_{yk} / γ_s , expressed according to bar diameter, for steels B500B and in 'good' bond conditions

Example: $f_{ck} = 35$ MPa, $f_{yk} = 500$ MPa, good bond conditions, $\phi = 20$ mm, $f_{bd} = 3.37$ MPa

$$l_{b,rqd} = \left(\frac{\phi}{4}\right) \cdot \left(\frac{\sigma_{sd}}{f_{bd}}\right) = (500 / 1.15) / (3.37 \times 4) \phi = 32.25 \phi = 645 \text{ mm}$$

For 'mediocre' bond conditions, the value in the table is divided by 0.7. Thus one obtains the basic anchorage length necessary according to bar diameter:

f _{ck} / f _{ck cube}	12 / 15	16 / 20	20 / 25	25 / 30	30 / 37	35 / 45	40 / 50	45 / 55	50 / 60	55 / 67	60 / 75	70/85
φ bar (mm)												
φ ≤ 32 mm	94 ø	78 φ	67 φ	58 φ	51 ø	46 ø	42 ø	39 ø	36 ø	35 ø	34 ø	34 ø
φ=40 mm	102 ø	84 ø	73 ø	63 ø	55 φ	50 φ	46 ø	42 ø	39 ø	38 ø	37 φ	37 ¢

Fig./Tab.III.(7): Values of basic anchorage length necessary to anchor f_{yk} / γ_S, expressed according to bar diameter, for B500B steels and in 'mediocre' bond conditions.

III.4. Design anchorage strength

The corresponding clauses are in [EC2-1-1 8.4.4]

The design anchorage length is defined by: $l_{bd} = max \{ \alpha_1 \alpha_2 \alpha_3 \alpha_4 \alpha_5 l_{b,rqd}; l_{b,min} \}$

 α_{l} is for the effect of the form of the bars, assuming adequate cover

 α_{2} is for the effect concrete minimum cover

 $\alpha_{3:}$ is for the effect of confinement by transverse reinforcement

 $\alpha_{4:}$ is for the influence of one or more welded transverse bars along the design anchorage length

 $\alpha_{5:}$ is for the effect of the pressure transverse to the plane of splitting along the design anchorage The product $\alpha_2 \alpha_3 \alpha_5$ has a lower limit of 0.7.

For tensioned bars: $l_{b,min} > max \{ 0.3 \ l_{b,rqd}; 10 \ \varphi; 100 \ mm \}$ For compressed bars: $l_{b,min} > max \{ 0.6 \ l_{b,rqd}; 10 \ \varphi; 100 \ mm \}$

By taking the minimum values $\alpha_1 = \alpha_4 = (\alpha_2 \alpha_3 \alpha_5) = 0.7$, it follows that $l_{bd} = 0.34 l_{b,rqd}$. It is thus impossible to have $l_{bd} < 0.3 l_{b,rqd}$ for tensioned bars. Similarly for compressed bars, with $\alpha_4 = 0.7$ comes $l_{bd} = 0.7 l_{b,rqd}$; it is thus impossible to have $l_{bd} < 0.6 l_{b,rqd}$.





All these coefficients α being at the most equal to 1, it results from the above table giving the basic anchorage lengths necessary to anchor f_{yk} / γ_S that:

for the qualities of concrete generally used for engineering structures, the design anchorage length of a tensioned bar in B500B steel by straight anchorage is at the most equal to 50 \$\phi\$.

This value of 50 ϕ is valid for mediocre bond conditions [Fig./Tab.III.(7)]. If this value is used as an anchorage length, it is then not necessary to verify the bar's bond conditions.

Similarly:

for the qualities of concrete generally used for engineering structures, the design anchorage length of a tensioned bar in B400B steel by straight anchorage is at the most equal to 40 ϕ .

For certain configurations, a simplification consists of considering that the anchorage of tensioned bars is ensured by taking account of equivalent anchorage lengths.

For "standard" bends, hooks and loops, the equivalent anchorage lengths $l_{b,eq}$ are defined as shown below.



pour un coude normal

crochet normal

boucle normale

Fig./Tab.III.(8): Equivalent anchorage lengths for standard bends, hooks and loops

Longueur d'ancrage équivalente pour un coude normal	Equivalent anchorage length for a standard bend
Longueur d'ancrage équivalente pour un crochet normal	Equivalent anchorage length for a standard hook
Longueur d'ancrage équivalente pour une boucle normale	Equivalent anchorage length for a standard loop

The drawings also define the ideas of "standard" bends, hooks and standard loops for which the forces are identical for the two ends.

 $\mathbf{l}_{b,eq} = \boldsymbol{\alpha}_1 \mathbf{l}_{b,rqd}$

- $l_{\rm hea} = 0,7 l_{\rm hrad}$ For tensioned reinforcement with $c_d > 3 \phi$:
- $l_{b.eq} = l_{b.rad}$ For tensioned reinforcement, with $c_d \le 3 \phi$:







The simplified method proposed by Eurocode 2 part 1-1 has no significance for the bends and hooks. It increases the length of steel in comparison with an instantaneous pessimistic calculation where 1 is used for coefficients α_2 to α_5 and where thus we have: $l_{bd} = \alpha_1 l_{b,rqd}$.

For example, for a high bond bar with with $\phi = 16$ mm and $R = 5 \phi$, the additional steel length is:

- for a standard 90° bend: 0.57 R + 4.8 φ ,or approx 12 cm

-for a standard 180° hook: 2.14 R + 5.6 ϕ , or approx. 26 cm

For standard loops, the difference between the two methods is minimal.



Fig./Tab.III.(10): Comparison between simplified formula and pessimistic application of the general formula

III.5.<u>Special case</u>

It is possible to use several types of anchorage (straight and curved). This arrangement is classic however in footing foundations.



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Fig./Tab.III.(11): Special case

IV.ANCHORAGE OF LINKS AND SHEAR REINFORCEMENT

The corresponding clauses are in [EC2-1-1 8.5].

The anchorages of links and shear reinforcement (rectangular hoops, stirrups and links) are done by means of bends or hooks (or by transverse welded reinforcement) by providing a bar inside the hook or bend and respecting the arrangements in the following figure. It should be noted that these arrangements are different from those with 'standard' hooks or bends of longitudinal reinforcement.





anchorage by bend (remember: for bends $90^{\circ} \le \alpha < 150^{\circ}$)

by bend anchorage by hook $a 90^\circ \le \alpha < 150^\circ$) (remember: for hooks $\alpha \ge 150^\circ$) Fig./Tab.IV.(1): Anchorages of links by bend or hook

V.LAPS

V.1.<u>General</u>

The corresponding clauses are in [EC2-1-1 8.7.1]

Forces are transmitted from one bar to another by:

- •
- Lapping of bars





- Welding
- Mechanical devices (sleeve)
- This paragraph concerns only transmission of forces by lapping.

V.2. Definitions

Eurocode 2 part 1-1 uses, according to the case, the terms 'neighboring' and 'adjacent'. In fact, the analysis of the English version shows that these two terms should be considered here as strictly synonymous.

- "adjacent laps": laps of neighboring bars. Note that the laps may be situated in different sections.
- "adjacent laps for a given section ": here the lapped bars can not be neighbours.



The overlaps B and C are neighbours. The overlaps B and D are neighbors for the section A considered. Fig./Tab.V.(1): Adjacent overlaps for a given section A

It is considered that a lap concerns a given section if its axis is at a distance less than 0.65 l_0 of this section, where l_0 is the lap length.



The laps of bars C and D whose axes are more than 0.65 l₀ from the section A, do not concern this section. Fig./Tab.V.(2): Laps to take into account for a given section

V.3. Longitudinal spacing between two adjacent laps

In practice, there are two cases for longitudinal spacing of adjacent laps:

• The laps concern the same section: in this case the longitudinal spacing between their axes and the considered section is less than $0,65 l_0$.



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Fig./Tab.V.(3): Adjacent laps for the same section

• The laps do not concern the same section: in this case it is advisable to space them at least 0.3 l₀. With this arrangement, no section will be concerned with the two laps at the same time.



Fig./Tab.V.(4): Adjacent laps in different sections

V.4. Clear distance between adjacent laps

The corresponding clauses are in [EC2-1-1 8.7.2]

The clear distance in the transverse direction between adjacent bars of two adjacent laps should be greater than 2ϕ and 20 mm.



Fig./Tab.V.(5): Clear distance between adjacent laps

Application of this rule is well limited to each layer considered separately, when the laps of the neighboring layers are displaced.

V.5.<u>Lap length l₀</u>

The corresponding clauses are in [EC2-1-1 8.7.3]

0

• <u>Case of straight laps</u>

$$\begin{split} & \text{if } c \leq sup \ \{ \ 4\phi \ , \ 50mm \ \} \ l_0 = sup \ \{ \alpha_6 \ l_{bd} \ ; \ l_{0,min} \ \} \\ & \text{if } c > sup \ \{ \ 4\phi \ , \ 50mm \ \} \ l_0 = sup \ \{ \alpha_6 \ l_{bd} \ ; \ l_{0,min} \ \} \ + \ c \end{split}$$





c: clear distance between the two bars of the lap



Fig./Tab.V.(6): Clear distance between bars of a lap.

 $l_{0,min} = max \{ 0, 3 \alpha_6 l_{b,rqd}; 15\phi; 200mm \}$

It is impossible that 0.3 $\alpha_6 l_{b,rqd}$ be greater than $\alpha_6 l_{bd}$ [Chapter 8-III.4].

 α_6 is a safety coefficient that takes account of the simultaneity of several laps concerning a given section.

 $\alpha_6 = (\rho_1/25)^{0.5}$ with ρ_1 proportion of bars with lap whose axis is at less than 0.65 l_0 from the axis of the considered lap. α_6 varies from 1 to 1.5.

ρ_1	< 25%	33%	50%	> 50%
α_6	1	1.15	1.4	1.5

• case of laps with curves

Eurocode 2 part 1-1 does not specify the arrangements to use in the case of laps with curves. In the absence of more accurate justifications, it is good to use the following arrangement: the length l_0 is measured in the straight part in compliance with the drawing below, and not from the start of the bar.



For a straight lap, there would be $l_0 = \alpha_6 l_{b,rqd}$; for an lap with curves, the simplified formula gives $l_0 = \alpha_6.\alpha_1.l_{b,rqd}$. Fig./Tab.V.(7): Length l_0 for laps with curves



Fig./Tab.V.(8): Length l₀ for an lap with standard hooks

Example of calculation of lap length – case of a standard 180° hook with $\alpha_1=0.7 \alpha_2.\alpha_3.\alpha_5=0.7$ and $\alpha_6=1.4$ (50% of lap for the considered section)

As previously, the design stress taken is $\sigma_{sd} = f_{yk} / \gamma_s$, and the persistent situation is considered.





$$l_{b,rqd} = \frac{\phi}{4} \frac{\sigma_{sd}}{f_{bd}} = \frac{\phi}{4} \frac{\frac{f_{yk}}{\gamma_s}}{2,25\eta_1\eta_2\alpha_{ct}} \frac{f_{ctk;0,05}}{\gamma_c} = \frac{\phi}{4} \frac{f_{yk}}{f_{ctk;0,05}} \frac{1,5}{2,25 \times 1,15 \times 1 \times 1 \times 1} = 0,58 \frac{\phi}{4} \frac{f_{yk}}{f_{ctk;0,05}}$$

Or the following special case: $\alpha_1 = 0.7$ (hook with $c_d > 3\phi$), $\alpha_2 \alpha_3 \alpha_5 = 0.7$, and $\alpha_6 = 1.4$ (50% of laps in the considered section).

$$l_0 = 0,7 \times 0,7 \times 1,4 \times l_{b,rqd} = 0,7 \times 0,7 \times 1,4 \times 0,58 \frac{\varphi}{4} \frac{f_{yk}}{f_{ctk;0,05}} = 0,4 \frac{\varphi}{4} \frac{f_{yk}}{f_{ctk;0,05}}$$

- Standard loop:
- The 'standard loops' are not used for laps. Particularly, the loops at the ends of prefabricated elements are not 'standard loops'.
- <u>Case of laps of compressed bars</u>

In the case of laps for compressed bars, the coefficient α_1 being equal to 1 regardless of the form of the anchorage the lapped bars, the lap length is equal to that of a straight lap. The use of curved anchorages thus requires more material (for anchorages [Fig./Tab.V.(7)]) with an lap length equivalent to that of a straight lap.

The joining of compressed bars by straight bars is advised. Further, for bent bars of medium or large diameter, there may be a large unbalanced radial force at the end of the bend, that justifies not taking account of the curved part for the lap.

Where straight bars are used, it should be remembered that it is advisable to make provisions to ensure safety of personnel.

V.6. Case with 100 % of laps in the same section

- This case can only apply in the following conditions:
- Compressed bars
- Secondary reinforcement (for distribution)
- Tensioned bars and arranged on one layer only
- •
- •

This is consistent, for example, with the case in paragraph 4.7 "longitudinal reinforcement" of the Sétra guide of June 2003 " prestressed bridges built with balanced cantilever method"

We then have $\alpha_6 = 1,5$.

V.7. <u>Transverse reinforcement in an lap zone in the case of tensioned bars</u>

The corresponding clauses are in [EC2-1-1 8.7.4.1]

Three levels of increasing exigence are used for transverse reinforcement according to the aggressiveness of the considered laps.

 $\circ \quad 1^{st} \text{ case } \phi < 20 \text{ mm or } \rho_1 < 25\%$





• If $\phi \ge 20 \text{ mm and } \rho_1 \ge 25\%$, the condition to verify is: $\sum A_{st} \ge A_s$

with A_{st} area of transverse reinforcement and A_s area of one of the lapped bars. ρ_1 see definition above [Chapter 8-V.5]

Thus two new cases present themselves:

 2^{nd} case: $\rho_1 \le 50\%$ or $a > 10 \phi$ thus it is advisable to place the transverse bars in the first layer and perpendicular to the lap direction.

 3^{rd} case: $\rho_1 > 50\%$ and $a \le 10 \phi$, thus the transverse reinforcement used should be rectangular hoop, stirrups or links, and perpendicular to the lap direction.



Fig./Tab.V.(9): Definition of "a" = distance between adjacent laps in a given section

These arrangements may be relaxed if the laps are in the zones where the bars are not too stressed.

In the last two cases, the reinforcement should be distributed in accordance with the drawing below.



Fig./Tab.V.(10): Arrangements of transverse reinforcement in an lap zone of tensioned bars

It is also conceivable to distribute uniformly the transverse reinforcement along the lap as per the drawing below.

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Fig./Tab.V.(11): Other possible arrangement of transverse reinforcement in an lap zone of tensioned bars

V.8. Transverse reinforcement in an lap zone of compressed bars

The corresponding clauses are in [EC2-1-1 8.7.4.2]

The same rules apply as for the tensioned bars, with an additional arrangement: to each side of the lap a transverse bar is placed at less than 4ϕ from the end of the lap.



Fig./Tab.V.(12): Transverse reinforcement in an lap zone of compressed bars

As for tensioned bars, it is possible to distribute uniformly transverse reinforcement along the lap, but always with the two additional bars placed at either side of the lap.

The aim of these two additional bars is to prevent the splitting of compressed concrete on contact with the end section of the bar.

VI.LARGE DIAMETER BARS

The additional rules of clause 8.8 of Eurocode 2 part 1-1 relative to large diameter bars apply to bars of diameter $\phi > \phi_{large}$.

The value to use is that shown in the national annex of Eurocode 2 part 1-1, that is $\phi_{\text{large}} = 40$ mm.





VII.BUNDLED BARS

VII.1.<u>General</u>

The bars may be placed together in bundles. The reinforcement should be of the same type and the same grade, possibly of different diameters subject to the ratio of the diameters not exceeding 1.7.

Unless otherwise stated, the rules for individual bars also apply for bundles of bars.

The number of bars is limited to 3 per bundle, except in the case of vertical bars in compression and bars in a lapped joint for which 4 bars maximum may be bundled together.

The reinforcement should be compactly arranged to prevent hindrance to the placing of the concrete.

In the special case of a bundle of two touching bars positioned one above the other and when the bond conditions are good, the bundle is treated as an individual bar.



Fig./Tab.VII.(1): Correct arrangement in the case of a bundle of three identical bars.

VII.2. Equivalent diameter of bundle

The corresponding clauses are in [EC2-1-1 8.9.1]

The equivalent diameter ϕ_n of the notional bar (same sectional area and same center of gravity as the bundle) is:

- Case where all bars have same diameter: $\phi_n = \phi \sqrt{n_b} \le 55 \text{mm}$
- Case of several bar diameters: $\phi_n = \sqrt{\phi_1^2 + ... + \phi_{n_b}^2} \le 55 \text{mm}$

where n_b the number of bars per bundle, limited to 3 or 4 according to the case.

VII.3.

VII.4. Clear distance between bundles of bars

The minimum clear distance between bundles of bars should be measured from the actual external contour of the bundle of bars.









Fig./Tab.VII.(2): Clear distances between bundles of bars

These distances should verify:

 $e_h, e_v \ge e_{mini} = \sup \{ \phi_n; (d_g + 5mm); 20mm) \}$

 d_g dimension of largest aggregate and ϕ_n equivalent diameter of bundle of bars. o with

In the particular case of a bundle of two bars positioned one over the other and when bond conditions are good, the bundle is treated as an individual bar, whence the following spacing:



Fig./Tab.VII.(3): Clear distance between bundles of two bars and individual bars

These clear distances should verify: $e_{h}, e_{v} \ge e_{mini} = \sup \{ \phi; (d_{g} + 5mm); 20mm) \}$

VII.5. Anchorage of bundles of bars

The corresponding clauses are in [EC2-1-1 8.9.2]

- Anchorage of bundles of tensioned bars 0
- 0
- 1st case: if the bars show large longitudinal staggered distance as indicated on the following sketches • (where $l_{b,rqd}$ is calculated with ϕ), the diameter of an individual bar ϕ is used to calculate l_{bd} .



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Fig./Tab.VII.(4): Anchorage of a bundle of two tensioned bars with a wide staggered distance.



Fig./Tab.VII.(5): Anchorage of a bundle of three tensioned bars with a wide staggered distance.

Décalages	Staggered distances
Ancrages	anchorages

Special case of a straight anchorage of a bundle of bars with, for a single bar , $l_{bd}(\phi) = l_{b,rqd}(\phi)$

- bundle of two bars: the anchorage length of the bundle is: $l_{paquet} \ge 2.3 l_{bd} (\phi)$

- bundle of three bars: $l_{paquet} \ge 3.6 \ l_{bd} (\phi)$

• **2nd case**: the equivalent diameter ϕ_n is used to calculate l_{bd} : $l_{bd}(\phi_n) = \sqrt{n} l_{bd}(\phi)$.

The corresponding paragraph of EN1992-1-1 is incomplete. It is well to keep a minimum staggered distance of the bars as shown on the drawings below.



with ϕ_2 equivalent diameter of the bundle of two bars, $\phi_2 = \sqrt{2} \phi$ Fig./Tab.VII.(6): Anchorage of a bundle of two tensioned bars







with ϕ_3 equivalent diameter of bundle of three bars, $\phi_3 = \sqrt{3} \phi$ Fig./Tab.VII.(7): Anchorage of a bundle of three tensioned bars

 $Special case of a straight anchorage of a bundle of bars with, for a single bar, l_{bd} (\phi) = l_{b,rqd} \\ - bundle of two bars: \quad l_{paquet} = 1.3 \ l_{bd} (\phi_2) = 1.3 \times 1.41 \times l_{bd} (\phi) = 1.83 \ l_{bd} \\ - bundle of three bars: \quad l_{paquet} = 1.6 \ l_{bd} (\phi_3) = 1.6 \times 1.73 \times l_{bd} (\phi) = 2.77 \ l_{bd}$

• Special case: $\phi_n < 32$ mm and bars near a support: no need to stagger the bars.



Fig./Tab.VII.(8): Anchorage of bundle of tensioned bars near a support for $\phi_n < 32 mm$

- Anchorage of bundles of compressed bars
- To stagger bar ends is not necessary
- if $\phi_n \ge 32$ mm, transverse reinforcement are arranged in compliance with the drawing below:



Fig./Tab.VII.(9): Anchorage of a bundle of compressed bars of equivalent diameter ≥ 32mm

VII.6.<u>Lapping of bundles of bars</u>

The corresponding clauses are in [EC2-1-1 8.9.3]

c is the clear distance between the two bundles of the lap measured from the actual external contour of the bundle:

```
• If c \le \sup \{ 4\varphi_n , 50mm \} l_0 = \sup \{ \alpha_6 l_{bd}; l_{0,min} \}
•
```





- - if > sup { $4\phi_n$, 50mm } $l_0 = sup {\alpha_6 l_{bd}; l_{0,min} } + c$

with l_{bd} calculated with the equivalent diameter ϕ_n .

 \circ <u>Case of bundle of two bars, with $\phi_n < 32 \text{ mm}$ </u>: it is not necessary to stagger the bars for the lap. The lap length l_0 is calculated with the equivalent diameter ϕ_n .



Fig./Tab.VII.(10): Lap of a bundle of 2 bars of equivalent diameter $\phi_n < 32$ mm

Calculé avec	Calculated with

The same arrangements of transverse reinforcement are applied in lap zones as those defined for individual bars ([Chapter 8-V.7], by replacing ϕ by ϕ_n).

• Case of a bundle of two bars with $\phi_n \ge 32 \text{ mm or of a bundle of three bars}$: the ends of bars should be staggered longitudinally by at least 1.3 l_0 , where l_0 is calculated with ϕ . It is possible to use an additional lap bar without however exceeding 4 bars in an lap cross section.



Fig./Tab.VII.(11): Lap of a bundle of 3 tensioned bars, with a 4th lapping bar



Detailing Chapter 9 - Detailing rules relative to members



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The détailing of members concerns the minimum reinforcement and the provisions to take for the reinforcement [EC2-1-1 Sect.9].

I.MINIMUM BENDING REINFORCEMENT

The minimum value of reinforcement to place is defined in [EC2-1-1 9.2.1.1] for reinforced concrete beams and in [EC2-1-1 9.3.1.1] for reinforced concrete slabs. For prestressed structures, the minimum reinforcement is given by the national annex that requires arrangement of the minimum reinforcement required by [EC2-1-1 7.3.2].

An example is given in [Appendix VII-2.5].

II.SURFACE REINFORCEMENT

Surface reinforcement is dealt with in two places in Eurocode 2: [EC2-1-1 7.3.3(3)] and [EC2-2/AN 9.1(103)].

Clause [EC2-1-1 7.3.3(3)] allows calculation of surface reinforcement for the sides of very high beams (h > 1.0m), **in tensioned zones only**, when the major reinforcement is concentrated on a part of the height only. For a rectangular beam in simple bending, of width b and tensioned height h_t , the value of the reinforcement is given by:

 $A_{s,peau} = k_c \times k \times f_{ct,eff} \times A_{ct} / f_{yk} = 0.4 \times 0.5 \times f_{ctm} \times b \times h_t / f_{yk}$

Clause [EC2-2/AN 9.1(103)] deals in a more general way with surface reinforcement for beam sides, at the same time in tensioned and compressed zones, parallel and perpendicular to the transverse section.

This surface reinforcement is not cumulative with the other steels calculated.

Application to a rectangular beam with height greater than 1.0m, a web thickness of 30cm, a concrete C30/37 and reinforcing steels $f_{yk} = 500$ MPa:

According to clause [EC2-1-1 7.3.3(3)], the amount of surface reinforcement to place, on the beam sides in a tensioned zone is equal to:

 $A_{s,peau} = 0.4 \times 0.5 \times 2.9 \times 0.30 \times 1.0 / 500 = 3.50 \text{ cm}^2$

Or 1,75 cm^2/m of surface.

Expressed as a percentage of the tensioned area, this value is equal to:

 $\rho = k_{\rm c} \times k \times f_{\rm ct,eff} \ / \ f_{\rm yk} = 0.4 \times 0.5 \times 2.9 \ / \ 500 = 0.12\%$

According to [EC2-2/AN 9.1(103)], the surface reinforcement to place on all the beam's perimeter is equal to 3cm²/m for the standard exposure classifications (XC4), or 5cm²/m in an aggressive environment (XD, XS).



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On the application it is stated that the calculation according to $[EC2-1-1 \ 7.3.3(3)]$ gives a value lower than the minimum of $3cm^2/m$ given in $[EC2-2/AN \ 9.1(103)]$. In standard cases it is possible to be exempt from the calculation of $[EC2-1-1 \ 7.3.3(3)]$ and to use directly the fixed values in section 9.

III.SHEAR REINFORCEMENT

III.1.Minimum section of shear reinforcement

The corresponding clauses are in [EC2-1-1 9.2.2]

III.1.1.Beams

A minimum transverse reinforcement will be required even in the case of elements not needing shear stress reinforcement.

The rate of shear stress reinforcement is given by the expressions:

$$\rho_{\rm w} = A_{\rm sw} / (s \times b_{\rm w} \times \sin \alpha)$$

[EC2-1-1 Expr.(9.4)]

[EC2-1-1 Expr.(9.5N)]

with a minimum recommended value confirmed by the national annex:

$$\rho_{\rm w,min} \, = (0,\!08 \sqrt{f_{ck}}\,) \, / f_{yk}$$

- $\circ \rho_w$ is the rate of shear reinforcement
- \circ A_{sw} is the area of the shear reinforcement section along the length s
- o s is the spacing of the shear reinforcement measured along the longitudinal axis of the element
- \circ b_w is the web width of the element
- $\circ \alpha$ is the angle between the shear reinforcement and the longitudinal axis [EC2-1-1 9.2.2(1)].

Compared to previous practices the expression (9.5N) leads to less steel for concretes with low characteristic strength and to more steel for higher characteristic strengths..





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Fig./Tab.III.(1): Variation of minimum ratio of shear reinforcement with characteristic strength of concrete

III.1.2.Slabs

The minimum rate of transverse reinforcement and its minimum value seen above for beams applies equally for slabs. However the slabs that benefit from a transverse load redistribution are exempt from it [EC2-1-1 6.2.1(4) et 9.3.2].

If shear reinforcement are necessary the slab should be at least 200mm thick.

Bridge slabs come under the category of slabs allowing a transverse load redistribution. As such, they may not include shear reinforcement unless the stresses applied to them require it. [Chapter 6-II.2.1]

Eurocode 2 [*EC2-1-1 9.3.1.4*], however, requires that longitudinal and transverse construction reinforcement be planned for along the free edges as shown in the figure below

For standard-thickness slabs (between 20 and 35cm) and concretes of $f_{ck} \leq 50$ MPa, a section of $2cm^2/m$ will cover the minimum reinforcement condition near the edges.





III.2.<u>Arrangements for shear reinforcement</u>

The major detailing arrangements of shear stress reinforcement are represented in the figure below [EC2-1-1 9.2.2].

Eurocode 2 part 2 recommends not using open frameworks and open stirrups.

A combination of frameworks and bent-up bars is acceptable.

The inclination of the shear reinforcement should respect the following condition: $45^{\circ} \le \alpha \le 90^{\circ}$.

However, at least 50% of the shear reinforcement should be in the form of frameworks and stirrups (this is a recommended value taken up by the national annex).





- A cadres, épingles et étriers intérieurs
- B cadre extérieur

Fig/Tab.III.(3): Example of shear reinforcement

Cadres, épingles et étriers intérieurs	Inner link alternatives
Cadre extérieur	Enclosing link

For beams of effective depth d, the maximum spacing of shear reinforcement courses is limited to:

Longitudinal direction	$s_{l,max} = 0.75 d (1+cot\alpha)$	[EC2-1-1 Expr.(9.6N)]
Transverse direction	$s_{t,max} = 0.75 \text{ d} \le 600 \text{mm}$	[EC2-1-1 Expr.(9.8N)]

For slabs, the maximum spacing in the longitudinal direction uses the same expression and the maximum transverse spacing is modified and limited to:

Transverse direction $s_{t,max} \le 1.5 d$ ($\alpha = \pi/2$)

With α = angle of inclination of reinforcement on the centroidal axis.

In the case of bent-up bars which are used less in practice, reference should be made to the text of *Eurocode 2*.

IV.TORSIONAL REINFORCEMENT

The detailing arrangements of torsional reinforcement are given in [EC2-1-1 9.2.3] and are represented in the figure below.

The frameworks should be anchored by overlaps and/or hangers and be perpendicular to the beam axis.

The maximum spacing of the transverse torsional reinforcement courses is limited to:

Longitudinal direction $s_{l,max} = min (0.75 \text{ d}; \text{U/8}; \text{smallest dimension of section})$

The maximum spacings of the longitudinal torsional reinforcement are limited to 350mm with at least one bar at each corner.

Previous regulations did not include particular specifications except for longitudinal reinforcement that had to be grouped together in the corners.





a) recommended configurations b) non-recommended configuration Fig/Tab.IV.(1): Examples of torsional reinforcement





CHAPTER 10 - SPECIAL PRESCRIPTIONS AND JUSTIFICATIONS



Special prescriptions and justifications Chapter 10 - Special prescriptions and justifications



I.SHEAR STRESS

I.1.<u>Résal effect</u>

In variable-height T or box girder sections with chords, the variation in shear force due to the Résal effect must be considered. It is calculated from:

- V_{ccd} : design value of the shear force component of the compressive force, in the case of a compressed inclined chord
- V_{td}: design value of the shear force component of the tensile force in the tensioned reinforcement, in the case of a tensioned inclined chord

The figure below shows the sign conventions [EC2-1-1 Fig.6.2]:



Fig./Tab.I(1): Shear stress components for variable-height elements

The shear force to take account of [EC2-1-1 6.2.1(6)], with taking into account of the variation due to the Résal effect is: V_{Ed} - V_{ccd} - V_{td} .

Eurocode 2 puts the Résal effect here on the side of the acting forces, but puts it also on the side of the resisting forces as in the following expression : $V_{Ed} < V_{Rd} = V_{Rd,s} + V_{ccd} + V_{td}$ [EC2-1-1 6.2.1(2) et (5)]

The two methods are equivalent. The previous regulations favored the second.

A more usual presentation of the rule applied to box girders is as follows:

Near an intermediate support, the section is subjected to a moment M and a shear force V_{Ed} , both negative. The lower slab is highly compressed whereas the upper slab is lightly compressed, even tensioned.

Where the upper slab remains compressed, the shear force variation of the Résal effect may be calculated as shown in the following diagram.



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Fig./Tab.I.(2): Résal effect – upper slab compressed

 N_s is the result of the axial stresses of the upper slab ($N_s > 0$ for a compression). In the case of tension it is the sum of the tensile stresses of the slab steels.

 α_s is the angle of inclination of the centroidal axis in relation to the upper slab.

 N_i and α_i are the corresponding values for the lower slab.

The variation due to the Résal effect is equal to: $\Delta V_{Résal} = -V_{cc,s} - V_{cc,i} = -N_s.sin(\alpha_s) + N_i.sin(\alpha_i)$

With $N_i > N_s$ and $\alpha_s \approx \alpha_i$, the variation ΔV_{Resal} is positive.

The force V_{Ed} being negative, the Résal effect is favorable in this case since it reduces the shear force (in absolute value).



Fig./Tab.I.(3): Résal effect- zones of members to consider

The taking into account of the Résal effect is a priori favorable because it is likely to reduce the general shear force for sections near the intermediate supports.

Conversely, when the mid-bay is approached, the upper slab becomes highly compressed, the lower slab tensioned and the Résal effect increases the general shear force.

The only difference with the previous practices comes from the fact that the web zones common with the slabs were not taken into account.

Another method of taking into account the Résal effect is also developed in the article by D. Le Faucheur, "Cumul des aciers de cisaillement et de flexion", in the Sétra "Ouvrages d'Art" magazine, n° 41 to which the designer might refer for more details. This article is based upon an



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analytical expression which lends itself more easily to a software programming. It gives however an increasing value of the Résal effect because the whole of the section is taken into account.

I.2. Shear between web and chords of T sections.

The corresponding clauses are in [EC2-1-1 6.2.4]

The strut and tie model was adapted to this configuration to be used in determination of reinforcement.

I.2.1.Minimum reinforcement of chords

The minimum reinforcement is no longer that specifically recommended relative to the shear stress [EC2-1-1 9.2.2], but that relative to slab bending reinforcement given by clause [EC2-1-1 9.3.1]. The following relationship will thus be applied:

$$A_{s,min} = 0.26 \frac{f_{ctm}}{f_{vk}} b_t d \qquad \text{with} \qquad A_{s,min} \ge 0.0013 \times b_t \times d \qquad [EC2-1-1 \text{ Expr.}(9.1N)]$$

In this case, d is the effective depth of the chord and b_t its length, as applied to the figure below $d = h_f$ et $b_t = \Delta x$.



A - compression struts B - longitudinal bar anchored beyond point obtained by construction with θ f (see 6.2.4 (7)) Fig./Tab.I.(4): Junction web-chord [EC2-1-1 Fig.6.7]

I.2.2.Acting forces

The longitudinal shear stress v_{Ed} , developed at the junction between a side of the chord and the web is determined by the variation of axial stress (longitudinal) applied to the part of the chord considered:

$$v_{Ed} = \Delta F_d / (h_f \times \Delta x)$$

[EC2-1-1 Expr.(6.20)]





In the case of distributed loads, a maximum value may be acceptable for Δx equal to half the distance between the nil moment section and the maximum moment section. In an isostatic beam, there may also be sections of one quarter of a bay on which the transverse shear reinforcement is constant.

In the case of concentrated loads the value Δx is at a maximum at the distance between loads. However in bridges this recommendation does not concern the action of concentrated loads from vehicles.

An alternative to the calculation of this stress is possible [EC2-2 6.2.4(103)]:

"The shear force transmitted between the web and the chord is equal to $V_{Ed} \Delta x / z$ and is broken down into three parts: one part exerting itself in the beam section limited to the width of the web, and the other two acting on the chords' flanges. Generally the part of the force the web stays subjected to is the fraction b_w / b_f of the total force. A greater force may be considered if the total effective width of the chord is not essential to resist the bending moment. In this case, it may be necessary to verify the crack openings at the service limit state"

This alternative comes back to the previous ways that applied to a length Δx , express the slipping force per unit of length:

 $v_{Ed} = (V_{Ed}/z)(b_1/b_{eff})$ where b_1 is the width of the chord of one side of the web.

As such, it remains as the force in the web $v_{Ed,w} = (V_{Ed}/z)(b_w/b_{eff})$

The expressions above are more specifically adapted to reinforced concrete T sections.

In the case of prestressed concrete, and box girders, in the absence of concentrated forces causing abrupt variations in axial stress in the slabs, it is better advised to use the classical expressions and to calculate:

 $v_{Ed} = \frac{V_{Ed} \times S}{I \times h_f}$ where S is the statical moment of the part of the chord concerned.

I.2.3. Verification of resistance

In the truss model used, the inclination of the struts in the chords is different than those of the webs.

The recommended values are:

 $1.0 \leq cot \theta_{\rm f} {\leq} 2.0$ for compressed chords

 $1.0 \leq cot \theta_{\rm f} \leq 1.25$ for tensioned chords.

The compression of the struts in the chords must be verified:

 $v_{Ed} \le v f_{cd} \sin \theta_f \cos \theta_f$ [EC2-1-1 Expr.(6.22)]

with $v = 0.6 (1 - f_{ck}/250)$

and the reinforcement calculated

 $(A_{sf} f_{yd}/s_f) \ge v_{Ed} h_f / cot\theta_f \qquad [EC2-1-1 \ Expr.(6.21)]$

These expressions are an adaptation of those relative to the verification of standard, more general sections.

If $v_{Ed} \le k \times f_{ctd}$, no shear reinforcement is necessary in addition to those required for bending.

Eurocode 2 part 1-1 recommends k = 0.4

The national annex recommends:

k = 0.5 in case of a vertical, construction-joint surface that is rough in the member



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k = 1.0 when there is no vertical, construction joint surface

(the verticality evoked here relates to the schematic diagram given above)

For information,
$$f_{ctd} = 1.50$$
 MPa for a C35 and 2.03 Mpa for a C60.

I.2.4. Combination with local bending

The combination bending/shear is dealt with in [Chapter 10-IV.3.3] of this guide.

I.3. Shear in construction-joint surfaces

The corresponding clauses are in [EC2-1-1 6.2.5]

I.3.1.Principle

In addition to the requirements stated to resist shear exerted between webs and chords or in webs, the following must be verified in the construction-joint surfaces:

 $v_{Edi} \le v_{Rdi}$ [EC2-1-1 Expr.(6.23)]

with

 $v_{Edi} = \beta V_{Ed}/(z.b_i)$ [EC2-1-1 Expr.(6.24)]

where β is the ratio of the axial longitudinal force in the construction-joint and the total longitudinal force in the compressed (or tensioned) zone.

This is also expressed as follows, assuming that we are still in an elastic domain:

 $v_{Edi} = V_{Ed}S/(I \times b_i)$

where S is the statical moment at the level of construction-joint

I is the section inertia.

The stress limit is given by the following expression

 $v_{Rdi} = c f_{ctd} + \mu \sigma_n + \rho f_{vd} (\mu \sin \alpha + \cos \alpha) \le 0.5 v f_{cd}$

[EC2-1-1 Expr.(6.25)]

 σ_n = axial stress at interface

c and μ coefficients depending upon the roughness of the interface

 α = inclination of the steels on the construction-joint

 $\rho = A_s / A_i$ = ratio of the steels crossing the construction-joint, brought to its surface.







Fig./Tab.I(5): Example of construction-joint surfaces [EC2-1-1 Fig.6.8]



A - re-concrete B - old concrete C - anchorage Fig./Tab.I(6): construction-joint with indentations [EC2-1-1 Fig.6.9]

The construction-joint surfaces are classified as very smooth, smooth, rough and indented:

Type of construction-joint	Coefficients	
Very smooth: surface cast in contact with molds of steel, plastic or specially-treated wood:	$c = 0.25$ et $\mu = 0.5$	
smooth: surface prepared by slip formworks, extruded surface or non-formworked surface, with no further treatment after vibration:	$c = 0.35$ et $\mu = 0.6$	
rough: surface has asperities at least 3 mm high, spaced at about 40 mm, obtained by ridging, direct washing or any other method giving equivalent results:	$c = 0.45$ et $\mu = 0.7$	
indented: surface has keys as shown in Figure 6.9:	$c = 0.50$ et $\mu = 0.9$	

Eurocode 2 part 2 recommends taking c = 0 for verifications in fatigue or dynamics. The stresses from traffic loads (UDL and TS tandem for example) are not to be considered as dynamic actions. The seismic actions are however dynamic actions.



Eurocode 2 part 1-1requires solely division by two of the values of c in the case of verification of fatigue or dynamic.

When the stress perpendicular to the construction-joint is tensile, c should be taken as 0.

I.4. <u>Members in composite bending, non-cracked at ULS and requiring no shear reinforcement</u>

For members requiring no shear reinforcement, subjected to composite bending and not cracked at ULS, clause [EC2-1-1 6.2.2(4)] leads to [EC2-1-1 12.6.3], which deals with resistance to shear stress of non-reinforced-concrete elements.

Clause [EC2-1-1 12.6.3(3)] recommends the sense of "non-cracked at ULS" for an element: it stays completely compressed or the absolute value of the major tensile stress in the concrete is less than f_{ctd} .

This clause gives calibrated calculation formulae for rectangular sections and for the case where the axial transverse stress is nil. The case of complex sections is not considered because the value k = 1.5 [EC2-1-1 Expr.(12.4)] adopted corresponds well to the maximum shear stress in a rectangular section.

Clause [EC2-1-1 12.1(2)] adds that section 12 applies to the elements where the effect of dynamic actions may be ignored. It **does not apply** when the effects are those caused by rotating machines and **traffic loads**.

It does not concern bridge decks but may be applied to other elements of structures such as foundation footings and piles of diameter greater than 600 mm for which $N_{Ed}/A_c \leq 0.3 f_{ck}$

Justification consists of verifying that the shear stress τ_{cp} in a section subjected to a shear force and a axial force is less than a design shear and compression resistance of concrete f_{cvd} . The designer is referred to the text of Eurocode 2 for detailed use of expressions (12.5), (12.6), (12.7) [EC2-1-1 12.6.3(2) and (3)]. They allow a curve to be drawn of the acceptable ULS shear stresses for lowly-reinforced parts, given below. In the same figure, the curve giving the stress limit given by annex QQ at SLS has been added for comparison.

The shear stress limit thus obtained is less than the stress limit calculated according to the recommendations of annex QQ of Eurocode2 part 2. This result is logical: the criterion of annex QQ expresses the non-cracking without safety factor, whereas the expressions [EC2-1-1 12.6.3 Expr.(12.5) to (12.7)] translate the resistance to the shear stress of non-reinforced parts, with great safety because the corresponding breakage is brittle. Conversely, it allows, without reinforcement, shear stresses higher than the expressions [EC2-1-1 Expr.(6.2a) and (6.2b)] as shown in the straight line representing $1,5V_{Rd,c}/b_wd$ resulting from these formulae and calculated with the maximum ratio of reinforcing steels.



Fig./Tab.I.(7): Shear stress acceptable at ULS according to [EC2-1-1 12.6.3] compared with stresses acceptable at SLS from annex QQ

Taux admissible MPa	Acceptable rate MPa
Contrainte normale sigma X MPa	Axial stress sigma X MPa
Cisaillement admissible béton non armé	Acceptable shear stress of non-reinforced concrete
selon 6.2a et 6.2b avec	According to 6.2a and 6.2b with

I.5. Prestressed members with one span only requiring no shear reinforcement

This case is dealt with in [EC2-1-1 6.2.2(2)] and particularly allows justification of prefabricated beams prestressed by pretension and used in building structures. The application of the given recommendations poses no particular difficulty and the designer is referred to the Eurocode text.

I.6. Special cases allowing an increase in resistance of concrete struts

I.6.1.Shear reinforcement stressed at less than 80% of f_{yk}

Note 2 of clause [EC2-1-1 6.2.3(3)] shows that in the case where the shear reinforcement are subjected to a calculated stress less than 80% of their yield strength f_{yk} the following values of v_1 may be adopted for calculation of the resistance of the struts:

$v_1 = 0.6$	if $f_{ck} \leq 60MPa$
$v_1 = 0.9 - f_{ck}/200 > 0.5$	if $f_{ck} > 60MPa$

This allows an increase in the resistance of the struts if the steels are less stressed.

Generally the design stress used for the dimensioning of the reinforcement is $f_{yd} = f_{yk}/1.15 = 0.87 f_{yk}$.

If the steels stress is $\sigma_s = 0.80 f_{yk}$ the over-consumption of steel is approximately 10% but the increase in resistance of the struts is in the ratio of v_1/v



The table below shows that the increase in resistance of the struts may be quite a lot greater than the loss of resistance of the steels.

f _{ck}	25	30	35	40	50	60	70	80	90
v expression (6.6)	0.540	0.528	0.516	0.504	0.480	0.456	0.432	0.408	0.384
v_1 expression (6.10)	0.600	0.600	0.600	0.600	0.600	0.600	0.550	0.500	0.450
ratio v_1 / v	1.111	1.136	1.163	1.190	1.250	1.316	1.273	1.225	1.172

It also shows that this rule is more favorable for the concretes with fairly high resistance.

I.6.2.Straight prestressing steels with high prestressing level

Eurocode reduces the value of the resisting shear stress for the members where the concrete is subjected to an mean compression greater than $0.5f_{cd}$ [EC2-2 (6.11.cN)]. Part 2 of Eurocode 2 seeks on the other hand to not penalize beams subjected to a straight prestress, giving increased compression if certain conditions are met [EC2-2 6.2.3(103) Note 4]: if the chords are capable of supporting the total prestressing force, and if blocks are used at the ends of the beams, the prestressing force may be assumed to be distributed between the chords. In this case $\alpha_{cw}=1$ may be used instead of the reduced value used for compression greater than $0.5f_{cd}$.



Fig./Tab.I.(8): Distribution of prestressing force in chords by end blocks [EC2-2 Fig.6.101]

II.STRUT AND TIE METHOD FOR ZONE OF DISCONTINUITY

This method was originally designed to adapt and expand the analysis method using the truss model to the treatment of areas called type D (D as in disturbance) that may consist of geometric discontinuities or disturbances due to the application of concentrated loads. The method was developed by the German school, particularly by Jörg Schlaich, and has been used and presented by the Comité Européen de Béton (CEB) in its Model Code. Eurocode 2 presents only a summary of it through its clauses "Analysis with strut and tie model" [EC2-1-1 5.6.4] and "Dimensioning using strut/tie models" [EC2-1-1 6.5]. For a correct usage of the method the designer is urged to consult, for example, the excellent report published in English by the PCI JOURNAL of May-June 1987 on the subject, entitled "Toward a Consistent Design of Structural Concrete" by Schlaich et al.

This guide mentions only a few essential rules.

II.1.<u>Principle</u>

The principle is as follows:





- determination of statically balanced forces in the struts and ties
- and finally, dimensioning and verification of struts, ties and nodes.





II.2. Dimensioning of struts, ties and nodes

II.2.1.Resistance of struts

The corresponding clauses are in [EC2-1-1 6.5.2].

Two cases are highlighted: the one where the strut is transversely compressed or not and the one where it is subjected to transverse tension, which leads to a reduction in its resistance.





In the figure on the right the vertical arrows represent a transverse compressive stress possibly nil. The resistances to compression of the struts are thus:

$$\begin{split} \sigma_{Rd,max} &= f_{cd} & \text{for transverse compression.} \quad [EC2-1-1 \text{ Expr.}(6.55)] \\ \sigma_{Rd,max} &= 0.6 \text{ v'} f_{cd} \text{for transverse tension.} \quad [EC2-1-1 \text{ Expr.}(6.56)] \\ & \text{with } v' &= 1-f_{ck}/250 \quad [EC2-1-1 \text{ Expr.}(6.57\text{N})] \end{split}$$

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This value of 0.6 v' may be brought closer to that of $v = 0.6(1-f_{ck}/250)$ where v is the reduction factor of the resistance of the cracked concrete to the shear stress.

II.2.2.Resistance of ties

The corresponding clauses are in [EC2-1-1 6.5.3].

The design resistances of the transverse ties and the tendons meet the same limits as given by the general rules [EC2-1-1 3.2 et 3.3].

In particular, for reinforcement of reinforced concrete it is advisable on the one hand to anchor them according to the principles of section [EC2-1-1 Sect.8] and on the other hand to limit their stress to:

$$f_{yd} = f_{yk}/\gamma_s$$

Eurocode 2 part 1-1 recommends not to concentrate the ties in their theoretical model position but to distribute them on the zone where stresses increase [EC2-1-1 6.5.3(3)].

Eurocode 2 part 1-1 then mentions the expressions of the tensile forces for two simple cases:

a) for the case of regions of partial discontinuity ($b \le H/2$), [EC2-1-1 Fig.6.25 a)],

$$T = \frac{1}{4} \frac{b-a}{b} F$$
 [EC2-1-1 Expr.(6.58)]

b) for the case of regions of total discontinuity (b > H/2), [EC2-1-1 Fig.6.25 b)],

$$T = \frac{1}{4} \left(1 - 0.7 \frac{a}{h} \right) F$$
 [EC2-1-1 Expr.(6.59)]







B Region without discontinuity D Region of discontinuity a) partial discontinuity b) total discontinuity Fig./Tab.II.(3): Transverse tensile forces in a field of compressive stresses with distributed reinforcement [EC2 -1-1 Fig.6.25]

II.2.3.Resistance of nodes

The corresponding clauses are in [EC2-1-1 6.5.4].

2.3.0.a)Compression limit in typical nodes

The compressive stress limits to meet for the three types of joint most commonly found are mentioned here:

(1) nodes subjected to compression with no tie anchored in them



Maximum acceptable compressive stress:

$$\sigma_{\rm Rd,max} = k_1 \,\nu' f_{\rm cd}$$

with k_1 = 1.0 value recommended by Eurocode 2 part 1-1 and the national annex

 $v' = 1 - f_{ck}/250$ (remember)

The national annex authorizes, on special justification, taking k_1 to k_1 = 1/ $\nu\text{'}$

Fig./Tab.II.(4): Node subjected to compression with no tie [EC2-1 - 1 Fig 6.26]

(2) nodes subjected to tension and compression with tie s in one direction







Maximum acceptable compressive stress:

 $\sigma_{Rd,max} = k_2 \nu' f_{cd}$

with k_2 = 0.85 value recommended by Eurocode 2 part 1-1 and the national annex

The national annex authorizes, on special justification, taking k_2 to $k_2 = 1.0$

Fig./Tab.II.(5): Nodes subjected to compression and tension with ties in one direction [EC2-1-1 Fig.6.27]

(3) Nodes subjected to tension and compression with ties in two directions



Maximum acceptable compressive stress:

 $\sigma_{Rd,max} = k_3 \nu' f_{cd}$

with $k_3 = 0.75$ value recommended by EC2-1-1 and the national annex

The national annex authorizes, on special justification, taking k_3 to $k_3 = 0.9$



2.3.0.b)Increase of compression limit for special conditions

For certain nodes the acceptable compressive stress values may be increased by 10% if at least one of the following conditions is verified:

- a tri-axial compression is assured,
- all the angles between struts and ties are $\geq 55^{\circ}$,
- the stresses at the level of the supports or the points loads are uniform, and the node is bordered by transverse reinforcement,
- the reinforcement are arranged on several courses,
- the node is securely confined by a special support arrangement or by friction.



2.3.0.c) Case of nodes subjected to tri-axial compression

The special case of nodes subjected to tri-axial compression may be dealt with as that of confined concrete [EC2 1-1 3.1.9] whose characteristic resistance may be defined by the expressions:

$f_{ck,c} = f_{ck} (1.000 + 5.0 \sigma_2 / f_{ck})$	pour $\sigma_2 \leq 0.05 f_{ck}$	[EC2-1-1 Expr.(3.24)]
$f_{ck,c} = f_{ck} (1.125 + 2.50 \sigma_2 / f_{ck})$	pour $\sigma_2 > 0.05 f_{ck}$	[EC2-1-1 Expr.(3.25)]

where σ_2 is the lateral compressive stress due to confinement.

The compression limit is however maximized at the value $\sigma_{Rd,max} = k_4 \nu' f_{cd,c}$

with $f_{cd,c}=f_{ck,c}/\gamma_c$. et $k_4=3.0$ value recommended and accepted by the national annex which, moreover, authorizes on special justification an increase in k_4 to the value of $k_4 = 3/v^2$.

III.DIFFUSION OF PRESTRESSING FORCES

III.1.<u>Rules proposed by Eurocode 2</u>

The major rules concerning the anchorage zones are found in [EC2 8.10.2] for pre-tensioning and in [EC2 8.10.3] for post-tensioning. These rules are completed by other rules spread around different chapters, which doesn't really help their application.

This guide is more especially concerned with justification of anchorage zones for post-tensioning. It mentions and presents in a more methodical way for the designer the essential rules to use for study of prestressing forces distribution.

III.1.1.Prestress force

Firstly the prestressing force to use for the study, applied at the active end of tensioning P_{max} , is found in [EC2-2 Anx.J.104.2]. It is defined in [EC2-1-1 5.10.2.1(1)P] with recommendations for use to prevent crushing or spilling of concrete.

For study of distribution, to be dealt with as a verification of local effect [EC2-1-1 8.10.3(2)], this force must be balanced by a factor $\gamma_{P,unfav}=1,2$ [EC2-1-1 2.4.2.2(3)]. The angle of distribution of the prestressing force, that

takes effect at the end of the anchorage device, may be simplified to equal 2β , with $\beta = a \tan(\frac{2}{3})$ [EC2-1-18.10.3(5)].

III.1.2.Calculation method

The suggested method is in [EC2-1-1 8.10.3(4)]: use of a strut and tie model or of other appropriate methods of representation to evaluate the tensile stresses due to concentrated forces. The reinforcement are to be arranged according to their design resistance. If the stress in the reinforcement is limited to 300Mpa for their design, the verification of crack opening is not necessary [EC2-1-1 8.10.3(4)].

Eurocode 2 part 2 proposes a value of 250 MPa as being even safer [EC2-2 8.10.3(104)]. Nonetheless, there is here a strange mix between control of cracking to be done at SLS and design





of the reinforcement to be done at ULS. So as not to adopt a too-penalizing rule, it is suggested that the value of 300 Mpa be used for the design and that it be considered as obtained from the value 250 MPa at SLS and increased by the coefficient 1.2.

III.1.3.Special rules

In [EC2-1-1 6.7] is found a rule that sets the stress limit to be met in the case of a uniformly distributed load applied to the concrete surface. This rule is not foreseen for large concentrated stresses like those developed by prestressing anchorages; on the other hand it may be used for zones under bearings.

Clause [EC2-2 Anx.J.104.2] is entitled "zones d'ancrage des éléments précontraints par post-tension"; "Anchorage zones of prestressed elements by post-tension". It is there that the use of P_{max} explicitly appears. This clause is in fact an addition to recommend what is to be done, just behind the anchorage in the zone called that of first regularisation. It is not intended to treat the distribution problem as a whole.

III.2. <u>Application of Eurocode 2 rules in the simple case of a single anchorage</u>

This is the application of the strut and tie method whose principles have already been shown in the preceding paragraph [Chap.10 II].

III.2.1.Data

A concrete beam of $0.5m \times 1m$ to which a prestressing force of $P_{max} = 1MN$ at tensioning is applied.

Characteristic of concrete at t_0 , date of tensioning: $f_{ck}(t_0) = 30$ MPa

Characteristic of reinforcement, $f_{yk} = 500 \text{MPa}$ or $f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{500}{1,15} = 435 \text{MPa}$

Taking account of partial factor $\gamma_{P,unfav} = 1,2$, a design force results from it:

 $P_d = \gamma_{P,unfav} \times P_{max} = 1,2MN$.

III.2.2.Choice of a strut and tie model

In the simple case, the choice of a model is easy since it has served for all presentations of the method. It is illustrated in the following figure [Fig./Tab.III.(1)]:





Fig./Tab.III.(1): Strut-tie method

Prisme de première régularisation	Primary regularisation prism
Longueur de régularisation	Regularisation length
Bielle de compression	Compression strut
Tirant	Tie

The first nodes are placed directly behind the anchorage plate. The stress is normally divided in two behind the anchorage and is distributed by two struts inclined at 33.7° $(a \tan \left(\frac{2}{3}\right))$. Once the distribution is effected, the load is distributed in the section with a uniform stress modeled by two horizontal struts situated at the lower and upper quarters of the section. It is generally accepted that this distribution is carried out completely on a length of the order of the element's transverse dimension. This length is determined by the choice of the angle of distribution that gives, in its turn, the necessary reinforcement section.

It may be noted that in the standard model used for a concentrated stress, the angle of diffusion is also often adopted as corresponding to 2 for 1 (2:1).

III.2.3. Verification of nodes

2.3.0.a)Nodes behind the anchorage plate

Behind the anchorage plate the mean compressive stress is:

 $\sigma = P_d/a \times a' = 1.2 / (0.150 \times 0.150) = 53 \text{ MPa}$

It is the stress that exerts itself on the nodes in this place. These nodes are submitted to compressive forces only. The maximum stress to verify is thus:

$$\sigma_{\text{Rd,max}} = k_1 \nu' f_{\text{cd}}$$
 with $\nu' = \left(1 - \frac{f_{\text{ck}}}{250}\right)$ and $k_1 = 1$ (recommended value) [EC 2-1-1 6.5.4], or 17.6 MPa.

It is obvious that this criterion can never be verified. Only a confinement of the concrete under anchorage, obtained by closed frameworks or adequately arranged reinforcement, may allow an increase in resistance to compression of the concrete to resist the applied stress [EC2-1-1 3.1.9(2)].

In the example studied the resistance to compression must be increased to a value such as





 $f_{ck,c}/1.5 \ge 53$ MPa, with $f_{ck,c} = f_{ck} \times (1.125 + 2.5 \sigma_2 / f_{ck})$ [E2-1-1 Expr.(3.25)] with however a maximum value of $\sigma_{Rd,max} = k_4 \nu' f_{cd}$ and $k_4 = 3$ [E2-1-1 6.5.4(6)]; for that the necessary hooping must be arranged to obtain a value of σ_2 allowing the condition to be satisfied.

In practice the reinforcement under anchorage foreseen in the ETA of the various prestressing procedures allows this objective to be reached in principle.

Indeed the post-tensioning kits are the object of a European Technical Agreement (ETA). They are subjected to a certain number of tests to demonstrate their ability to take the force exerted by the tendon and to transfer it to the concrete structure (particularly transfer tests on end blocks). Finally, the ETA of a post-tensioning kit recommends the major characteristics of the anchorages and the possible uses, and gives the reinforcement to be arranged behind the anchorage as well as the interaxials and distances to the edge to be respected to ensure good load transfer to the concrete. It also recommends the minimum strength of concrete to reach before tensioning.

Eurocode 2 recommends of course respect for the ETA recommendations.

Eurocode 2 part 1-1 gives insufficient detail to the designer to do the practical design of this hooping. It is to compensate for this lack of practical calculation method of the reinforcement that the clause [EC2-2 Anx.J.104.2] was added in part 2. It allows to foresee, in the volume defined as being the primary regularization prism, the minimum reinforcement necessary that thus allows passage from very high stresses to stresses of the order of $0.6 f_{ck}(t_0)$.

The designer may also find a more detailed method in the Model Code 90 [MC90 3.5] for determination of this reinforcement.

2.3.0.b)Nodes after diffusion of the prestress stress

In the nodes after distribution of the prestressing force the mean compressive stress is:

 $\sigma = P_d / b \times h = 1.2 / 0.5 \times 1 = 2.4 \text{ MPa}$

Since the nodes are subject to two compressive forces and one tensile force, the stress not to be exceeded is:

$$\sigma_{\text{Rd,max}} = k_2 \,\nu' f_{\text{cd}}$$
 with $\nu' = \left(1 - \frac{f_{\text{ck}}}{250}\right)$ and $k_2 = 0.85$ (recommended value)

 $\sigma_{\text{Rd,max}} = k_2 \text{ v' } f_{\text{cd}} = 0.85 \times \left(1 - \frac{30}{250}\right) \times \frac{30}{1.5} = 15 \text{ MPa}$ or a value much greater then the applied stress.

After distribution the nodes may be called "smeared nodes" in the strut-tie method; they are generally exempt from verification of the criterion of concrete compression limit.

Verification of all nodes is thus assured.

III.2.4. Verification of struts

The stress in each strut just after the first regularisation prism is of the order of $0.6 f_{ck}(t_0)$ if the necessary has been done for it to be thus: respect of the distances of the anchorage plates at the free edges of the parts, design of primary regularisation prism, design and arrangement of reinforcement at level of this prism (see below [Chapter 10-II.2.5.a)]). This stress will even diminish as one gets nearer from the other end of the strut whose section approaches half the section of the beam in the example considered.

According to clause [EC2-1-1 6.5.2], in the case of the concrete struts in an area where there are no transverse tensile stresses, the acceptable stress is given by:





 $\sigma_{Rd,max} = f_{cd}(t_0) = \frac{f_{ck}(t_0)}{1.5} = 0,66 f_{ck}(t_0)$. The criterion on the struts is thus well verified.

III.2.5.Design of reinforcement

2.5.0.a)Reinforcement in primary regularisation prism

Primary regularisation prism

The primary regularisation prism is defined by its rectangular section $c \times c'$ and its depth $\delta = 1,2 \max(c;c')$ [EC2-2 Anx.J.104.2 (102)].

The dimensions c and c' should be such that:

$$c \times c' = \frac{P_{max}}{0.6 f_{ck}(t_0)}$$
, to respect, on the one hand a reasonable stress limit

and on the other hand, the criteria of geometry

 $\frac{c}{a} \leq 1,25 \sqrt{\frac{c \times c'}{a \times a'}}$ and $\frac{c'}{a'} \leq 1,25 \sqrt{\frac{c \times c'}{a \times a'}}$

where a and a' are the dimensions of the smallest rectangle including the anchorage plate.

The idea is to go from the anchorage plate to an associated rectangle whose area is chosen to obtain a mean stress equal to $0.6f_{ck}(t_0)$. The rectangle's dimensions may then be adapted, for geometrical reason, but in keeping a constant area. This is expressed by the geometric conditions previously seen.

The formulae in Eurocode 2 allow a variation of up to $\pm 25\%$ of the rectangle's dimensions whereas the ETAs allow only $\pm 15\%$. It is thus recommended that the factor 1.25 be replaced by 1.15 or 1/0.85.

It will be noted that the mean stress of $0.6f_{ck}(t_0)$ leads most of the time to safer dimensions than those proposed in the ETA.

In taking the simple case of a square prestressing anchorage with a = a', in choosing c = c' and in not using the allowed dimensional variations, c is thus defined by the only first condition (the last two inequalities are automatically verified).

This gives
$$c = \sqrt{\frac{P_{max}}{0.6 f_{ck}(t_0)}}$$
, or in the example $c = 0.236 \text{ m}$ and $\delta = 0.283 \text{ m}$.

Reinforcement

As already stated above, the reinforcement to arrange in the primary regularisation prism just determined are in fact given by the ETA of the post-tensioning system. Its minimum section may be deduced from the simple dimensioning given in [EC2-2 Anx.J.104.2 (103)]:

$$A_s = 0.15 \frac{P_{max}}{f_{yd}} \gamma_{P,unfav}$$
, or in this example $A_s = 4 \text{ cm}^2$.

This transverse reinforcement must cross the first regularisation prism in two orthogonal directions and have in each of these directions the minimum preceding section. The reinforcement will moreover be distributed over the whole prism length.





The reinforcement given by the ATE is often more than the minimum required. It is however defined for a single anchorage. It is advisable to adapt its fabrication to the actual dimensions of the elements particularly in the case of several juxtaposed anchorages. It is also advisable to ensure a good attachment of the prism to the concrete surrounding the element.

2.5.0.b) Reinforcement corresponding to the tie of the strut and tie method

Considering the struts inclined at $a \tan\left(\frac{2}{3}\right)$, this gives a force in the tie of $\frac{P_d}{2} \times \frac{2}{3} = 0.4$ MN.

This gives a diffusion reinforcement $A_{sd} = \frac{0.4}{300} = 13.3 \text{ cm}^2$ to place in addition to the steels A_s previously defined along the length of the primary regularisation prism [Fig./Tab.III.(1)].

defined along the length of the primary regularisation prism [Fig./ 1 ao.1.

2.5.0.c)Positioning of previous reinforcement

This information is not given in Eurocode 2. In principle the documents dealing in detail with the strut and tie method should be consulted. It will nonetheless be appreciated that these reinforcement should be distributed and have as an average position that of the tie of the model, while respecting the rules of good construction.

III.2.6. Conclusions on Eurocode 2 rules

The rules proposed by Eurocode 2 to deal with distribution of prestressing forces by post-tension are obviously neither sufficiently detailed nor complete. The serious problem can be cited of the combination of the transverse diffusion reinforcement with the shear stress and torsion reinforcement that is not mentioned at all.

Further, the strut and tie method, if it has the advantage of making the designer reflect on the way the stresses are distributed in the regularisation zone, on the other hand it puts the designer in a delicate situation faced with the method's disadvantages and deficiencies. Already he has to keep diagrams that are too complicated (a good model of struts and ties is that which minimizes the number of ties), but in any event the method may rapidly become inextricable in more complex cases (particularly 3D). And there is its major disadvantage: the slightly more complicated cases shown in the example require a lot of experience and a high level of expertise on the part of the designer to be correctly dealt with. The advice given to designers is often cited: this involves making a model of the finished elements to obtain the inclination of the struts and to verify that the proposed diagram is compatible with the elastic forces.

III.3. <u>Sétra guide method [Distribution of concentrated loads]</u>

III.3.1.Principle

The distribution is verified according to two orthogonal directions corresponding in general to the major axes of inertia of the section studied (often horizontal and vertical).

The calculation of distribution of the prestressing force is done in two zones:

- one zone including the immediate vicinity of the anchorage called local prism;

- one zone called regularisation including all the section along the length necessary for regularisation of stresses.

If D is a given corresponding to the first direction, the corresponding given in the second direction will be D'.

This method is based upon verification of the shear stresses at the level of cut offs made in the part considered.





III.3.2.Justifications in local prism



Fig./Tab.III.(2): Local prism

It is a prism of section $c \times c'$ centered around the anchorage and of a depth δ .

The intersection of the local prism with the section is called the impact rectangle. This impact rectangle should not overlap the impact rectangles associated with the neighboring anchorages nor come out of the concrete.

Area A of the impact rectangle verifies:

$$A = \max\left(\frac{F_{max}}{0.6f_{ck}(t_0)}; 4b_0b_0'\right)$$

 $2b_0$ and $2b'_0$ are the dimensions of the end block tested to obtain the ETA.

 F_{max} is the prestressing force at tensioning.

Furthermore, c and c' are limited in such a way that the proportions of the rectangle stay near to those of the end block.

For this, the rectangle homothetic to the section of the end block of area A is defined by its dimensions $c_0 \times c'_0$:

$$\mathbf{c}_0 = \sqrt{\mathbf{A}} \sqrt{\frac{\mathbf{b}_0}{\mathbf{b}_0'}}$$
 and $\mathbf{c}_0' = \sqrt{\mathbf{A}} \sqrt{\frac{\mathbf{b}_0'}{\mathbf{b}_0}}$

Then a variation of $\pm 15\%$ on its dimensions is permitted to define the impact rectangle.

Thus, c and c' must meet the conditions:

$$0,85c_0 \le c \le \frac{1}{0,85}c_0$$
$$c \times c' = A$$

The depth δ is defined by the following formula: $\delta = 1,2 \max(c;c')$





The writing of annex J of Eurocode 2 part 2 was inspired by the justifications in the local prism of the Sétra guide.

The case of multiple anchorages is dealt with in the Sétra guide.

Taking the example previously developed, the result is:

c = c' = 0,236m and $\delta = 0,283m$. (the conditions on b_0 and b'_0 are not taken into account since they depend specifically on the anchorage considered)

3.2.0.a) Reinforcement in the local prism

The local prism should be made up of the reinforcement recommended by the ETA of the post-tensioning kit.

Furthermore, the local prism should be stitched to the remainder of the cross-section of the member by <u>local</u> <u>zone reinforcement</u>. For an adaptation of the outline of the reinforcement of the ETA (extension on all the dimension of the panel), the reinforcement of the ETA may serve as attachment reinforcement. If not, they come as an addition.

3.2.0.b)Local zone reinforcement

Their dimensioning takes account of the inclination of the force and the eccentricity in relation to the axis of the panel.

$$A_{se} = \frac{\left[0,15\,\xi + \sin\left(\xi - 1\right)\right] \times F_{d}}{f_{vd}}$$

where F_d design prestress force $F_d = 1,2 F_{max}$

$$f_{yd}$$
 design stress of steel sections; $f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{f_{yk}}{1,15}$ (f_{yk} limited to 500MPa)

 α angle of inclination of tendon

$$\xi = \sqrt{\frac{2}{d \times \left(\frac{3}{h} + \frac{1}{c}\right)}}$$

see figures [Fig./Tab.III.(3)] and [Fig./Tab.III.(4)] for definitions of d, c and h.



Fig./Tab.III.(3): First zone – offset inclined tendon



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Fig./Tab.III.(4): First zone – definition of local height

hauteur locale	Local height
hauteur totale	Total height
In the example:	

d = 0,5 m;c = 0.236 mh = 1 m $\alpha = 0$ rad

hence: $\xi = 0,743$

$$A_{se} = \frac{[0,15 \times \xi + \sin \alpha(\xi - 1)] \times F_d}{f_{vd}} = \frac{[0,15 \times 0,743] \times 1,2}{500/1,15} = 3cm^2$$

III.3.3. Justifications in the regularization zone



Fig./Tab. III.(5): Definition of regularization zone

The regularization zone extends from the section where the tendons are anchored (Anchorage section S_A) to the regularization section (S_R) where it may be considered that the stresses from the anchorage forces are linearly distributed on the whole section, according to Bernoulli's principle.



Fig/Tab.III.(6): Calculation of regularization length





III.3.4. Justifications in the regularization zone

3.4.0.a) Length of regularization

The regularization length is calculated in the following manner in a given direction:

 $L_{R} = \max \left(H - d, \frac{H}{2} \right)$ In the example, $L_{R} = \max \left(1 - 0.5; \frac{1}{2} \right) = 0.5 \text{ m}$

3.4.0.b) Verification of stresses and calculation of reinforcement to be arranged on the regularization zone

On each cut of the section situated at a distance greater than c/2 or c'/2 of an anchorage, the axial stress N* and the shear force V* being applied to this cut (figure opposite) are calculated by taking account of all the stresses exerted on the block isolated by the cut (prestressing forces, support reaction, self weight, stresses at sections situated at both sides of S_A and S_R, etc).



Fig./Tab.III.(7): Stresses on cut

Plan de coupure	Cut plan

The prestressing forces are the design forces F_d (=1.2 F_{max})

The other stresses taken into account are the ULS stresses concomitant with the lowest support reaction (possible).

If σ^* and τ^* are the mean stresses corresponding to N* and V*, they should verify:

$$\left|\boldsymbol{\tau}^{*}\right|\!-\!\boldsymbol{\sigma}^{*}\!\leq\boldsymbol{f}_{csd}\!\left(\boldsymbol{t}_{0}\right)$$

where $f_{csd}(t_0)$ is the concrete shear stress limit at tensioning: $f_{csd}(t_0) = 1,2 \times f_{ctk\ 0,05}(t_0)$.

In the example:

On a cut plan situated at $\frac{c}{2}$ of the center of the section, the result is:

$$N^* = 0$$

$$V^* = F_d \times \frac{\frac{H_2 - c_2}{H}}{H} = 1.2 \times \frac{0.5 - \frac{0.236}{2}}{1} = 0.46MN$$

whence


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$$\sigma^* = 0$$

$$\tau^* = \frac{V^*}{L_R \times 0.5} = 1.83 \text{ MN}$$

Knowing that $f_{ctk 0.05}(t_0) = 2MPa$, $f_{csd}(t_0) = 2.4MPa$, the criterion $|\tau^*| - \sigma^* \le f_{csd}(t_0)$ is verified

3.4.0.c) General equilibrium reinforcement

$$\mathbf{A}_{sc} = \frac{(|\mathbf{V}^*| - \mathbf{N}^*)}{f_{yd}} \times \left(0, 2 + 0, 8 \times \sqrt{\frac{|\tau^*| - \sigma^*}{f_{csd}}}\right)$$

In the example, the result is:

$$A_{sc} = \frac{0.46}{435} \times \left(0.2 + 0.8 \times \sqrt{\frac{1.83}{2.4}}\right) = 10 \text{ cm}^2$$

3.4.0.d)Positioning of reinforcement

In the local prism, the ETA reinforcement and the local zone reinforcement A_e that should cover the whole height of the part (the ETA reinforcement may be adapted here) are placed.

This reinforcement is installed to have at least $\frac{2}{3} A_{sc}$ on the first third of $L_R A_{sc}$ on the second third of L_R .





Where the section is not the definitive end section, reference should be made to the guide edited by Sétra.

III.4.Summary of reinforcement obtained

The following table summarizes the major results obtained by the two methods:

Method EC2 strut and tie type		Method Sétra guide type		
Reinforcement ETA	see ETA	Reinforcement ETA	see ETA	



Reinforcement regularization prism	in 1	first	4 cm ²	Local zone reinf	orcement	3 cm ²
Reinforcement obta and tie method	ined by	y strut	13.3 cm ²	General reinforcement	equilibrium	10 cm ²

III.5. Conclusions

In the example chosen the two methods give quasi-similar results. However, one must not lose sight of the fact that use of the first method, the strut and tie method, was largely facilitated by the simplicity of the example, chosen for its didactic ends. As already mentioned, the method gives insufficient recommendations about where and how reinforcement should be placed, and it quickly becomes very involved for complex structures. On the other hand, the second method, called "analytical method", that has served as a basis for previous practices, has the great advantage of dealing with the problem of distribution from a global approach. As such it is not only complete but allows a simple adaptation to much more complex cases than the example chosen.

In conclusion the designer is advised to use the guide "Diffusion des efforts concentrés – Efforts de précontrainte et des appareils d'appui" edited by Sétra, that has summarized the expertise acquired on the subject and corrected a number of imperfections of previous practices. Further, the regulations in this guide were adapted before its publication to put them in compliance with the spirit and expressions of the Eurocodes.

IV.PLATE DESIGN

The design of plates (particularly bridge slabs and webs) brings up a number of recurring questions: which stress values to consider? How to combine them? How to combine bending, shear stress, diffusion steels? Is there an overall performance model for plates, in reinforced concrete, subject to bending and shear forces?

The paragraphs detail Eurocode 2's answers to these questions, together with useful additional information on those questions not dealt with in Eurocode 2.

IV.1.Stress calculations

IV.1.1.Analysis method

For bridges, calculation of stresses in the plates is generally done by linear elastic analysis. Eurocode 2 part 1-1 permits other methods (elastic analysis with fixed moment redistribution, plastic analysis, non-linear analysis). These methods are essentially used in buildings, where the risks from cracking are low. In civil engineering structures the use of plastic analysis is not recommended except in special cases. Analyses with fixed redistribution are possible subject to effective crack control at SLS.

IV.1.2. Calculation value used for design

Eurocode does not specify the moment value to use for plate design, particularly where concentrated loads are applied: is it the bending moment resulting from the analysis, or could a mean value be used? In compliance with past French practice, an average moment value might be used on a width equal to twice the thickness of the slab (average taken transversely in the direction of the stress considered).





In practice, for thicknesses and loads currently used in civil engineering structures, there is very little difference between the two calculations. The bending moments of the abacuses in the bridge slabs published by Sétra have taken into consideration the point bending moments.

IV.1.3. Stress combinations

Model LM1 of Eurocode 1 part 2 leads to a combination of overall and local effects. For example, in an upper slab of a box girder bridge or in a composite bridge slab, at an intermediate support, an overall tension due to the negative moment created by the UDL loads distributed along the bay, and local bending of the slab created by a TS tandem situated near the section, may exist simultaneously.

There are two possible solutions:

- Deal in an exact manner with the load concomitants (particularly when it is not possible to dissociate overall and local effects, for example for slab bridges), or
- Take account of concomitances in a simplified way, from extreme stresses.

Eurocode 2 has no information regarding the combination of overall and local stresses. On the other hand, annex E of Eurocode 3 part 2 gives a combination factor ψ , a function of the span length. The combination rules proposed in this annex may be used to treat the examples cited above.



Fig./Tab.1.1): Combination factor depending on span length [EC3-2 Fig.E.2]

IV.2. General problematic of design of concrete plates

The figure below, from annex LL of Eurocode 2 part 2, represents the different stresses that might act on a slab element:







Fig./Tab.1.1): Different stresses that might act on a slab element

- 3 plate components n_{Edx} , n_{Edy} , $n_{Edxy} = n_{Edyx}$
- 3 slab components m_{Edx} , m_{Edy} , $m_{Edxy} = m_{Edyx}$
- 2 transverse shear forces v_{Edx} , v_{Edy}

In the case of a bridge slab, there are typically:

- a longitudinal stress n_{Edx} , created by the global bending in the longitudinal direction,
- a stress n_{Edy} , created by a possible transverse prestress,
- bending moments m_{Edx} and m_{Edy} created by TS loads applied on the slab,
- an in-plane shear flux n_{Edxy} , created by the global torsion,
- transverse shear v_{Edx} and v_{Edy} , created by the TS loads applied on the slab.

In the case of a web of a box bridge, there are typically:

- a longitudinal stress n_{Edx} , created by the longitudinal prestress,
- a bending moment m_{Edy} created by transverse bending (effect of TS loads applied on the slab),
- an in-plane shear flux n_{Edxy} , due to the beam shear/torsion combination.

The other components are generally small.

Eurocode 2 allows dimensioning of the necessary reinforcement for each of these stresses taken individually, according to the rules presented in the preceding chapters of this guide.

There is thus the question of the reinforcement combination obtained. Eurocode 2 deals with this problem in several ways:

• fixed combination rules for some special cases (particularly combination of in-plane bending and shear),





• a general method of justification of a plate under a unity of concomitant stresses.

These combination rules are all formulated at ULS.

IV.3. <u>Fixed combination rules for reinforcement</u>

IV.3.1.Combination of shear stress steels/torsional stress/prestressing dispersion steels

The combination of shear stress and torsional stress steels is covered in chapter 6.

Eurocode 2 gives no rules concerning the combination rules of prestressing dispersion steels. Previous practices might for example be used (see the Sétra design guide on prestressed concrete bridges built using balanced cantilever method):

$$A_{cis} = \max \begin{cases} A_{dispersion} \\ \\ min \begin{cases} 1,5 \ A_{shear/torsion} \\ A_{dispersion} + A_{shear/torsion} \end{cases}$$

IV.3.2. Combination bending/transverse shear

Applications: slab subjected to a combination of bending/transverse shear stress.

This combination rule was developed in the chapter relative to the justifications of the shear stress. It is the well-known 'shift rule', where the calculation of longitudinal reinforcement is based on an offset moment of length a_l in the unfavorable direction:

- $a_1 = d$ if the slab requires no shear stress reinforcement
- $a_1 = z \times \cot\theta / 2$ if not, θ being the inclination of the shear stress struts.

IV.3.3.Combination bending / in-plane shear

These rules are given in Eurocode 2 for the connection of a slab to a concrete web. It is however quite acceptable to apply them in a more general way to webs and slabs.

Applications:

- connection of a slab to a concrete web or a steel web
- web subjected to a shear stress/transverse bending combination.

3.3.0.a)Rule for reinforcement

The corresponding combination rules are given in [EC2-1-1 6.2.4]. The combination rule may be summarized as follows: $A = Max \{ A_{cis}; \frac{1}{2} A_{cis} + A_{flex} \}$, where

 A_{cis} is the reinforcing steel section required to balance the maximum shear/torsion and prestressing dispersion stresses [chapter 10–IV.3.1]

A_{flex} is the reinforcing steel section necessary to balance the maximum bending moment

This formula is written to illustrate the case of the figure (6.7) of Eurocode 2 part 1-1 (connection of a slab to a concrete web), where a single bending moment is considered to bend the upper fiber.



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In the more general case where there are two bending moments of opposite signs, the distribution between the two layers of reinforcement must respect the following rules:

 $A_{\text{sup}} + A_{\text{inf}} \geq Max \ \left\{ \ A_{\text{cis}} \text{; } \frac{1}{2} \ A_{\text{cis}} + A_{\text{flex,sup}} \text{; } \frac{1}{2} \ A_{\text{cis}} + A_{\text{flex,inf}} \ \right\}$

where

A_{flex,sup} is the upper layer reinforcement section necessary to balance the corresponding bending moment

Aflex,inf is the lower layer reinforcement section necessary to balance the corresponding bending moment

By proposing this combination rule, Eurocode considers there to be no simultaneous instance of maximum bending and maximum shear. However, Eurocode ignores the contribution of compressed concrete to shear resistance, which is on the safe side.

Eurocode does not specify how to distribute the steels between the two layers. The following distribution may, for example, be adopted:

$$\begin{split} A_{sup} &\geq A_{cis} \; / \; 4 + A_{flex,sup} \\ A_{inf} &\geq A_{cis} \; / \; 4 + A_{flex,inf} \\ A_{sup} + A_{inf} &\geq A_{cis} \end{split}$$

Other distribution is always possible, particularly for the evaluation of existing structures.

Example 1:

 $A_{\text{flex,sup}} = 20 \text{cm}^2 \qquad A_{\text{flex,inf}} = 0 \text{cm}^2 \qquad A_{\text{cis}} = 12 \text{cm}^2$

The total area of reinforcement should exceed max $(12; 20+12/2) = 26.0 \text{ cm}^2$, to divide between the two layers. By adopting the recommended distribution, 23 cm^2 will be placed on the upper layer and 3 cm^2 on the lower layer.

Example 2:

 $\begin{array}{ll} A_{\rm flex,sup} = 5 {\rm cm}^2 & A_{\rm flex,inf} = 5 {\rm cm}^2 & A_{\rm cis} = 36 {\rm cm}^2 \\ \mbox{The total area of reinforcement should exceed max (36; 5+36/2) = 36 cm^2, to divide between the two layers while respecting a minimum of 14 {\rm cm}^2 on the upper layer and 14 {\rm cm}^2 on the lower layer. There will thus be 18 {\rm cm}^2 {\rm per layer.} \end{array}$

3.3.0.b) Case of exemption from combination

When shear is low, that is when $v_{Ed} < 0.4f_{ctd}$ [EC2-1-1 6.2.4(6)], only the steels necessary relative to bending need to be put in place.

3.3.0.c)Combination rules relative to compression in concrete

Eurocode 2 part 2 imposes in [EC2-2 6.2.4(105)] a combination rule for compression in concrete. According to this rule, when the concrete is raised to its plastic resistance to balance bending, it cannot also contribute to the resistance of the shear stress struts. It is advisable to verify the compression in the struts based on a reduced height of concrete, namely $h_{f,red}$, where:

 $h_{f,red}$ = slab height – compressed height used in ULS bending

IV.4. General method of plate verification

Annexes F and LL of Eurocode 2 part 1-1 and Eurocode 2 part 2 present a method that may be used to verify in a general way the resistance at ULS s of a plate subjected to a given combination of concomitant stresses.



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The method is directly applicable to a plate with an orthogonal reinforcement, and it may be used with certain adaptations in the case of a plate with skew reinforcement.

The general principle of this verification consists in splitting the plate into three layers:

- 2 outer layers that balance the bending stresses and in-plane shear,
- 1 intermediate layer that balances the transverse shear

The plate stresses are broken down into the three layers. The 2 outer layers thus work as a membrane, which means they are subjected only to in-plane stresses, with no bending moments.

The articulation of the various parts of the Eurocode 2 is as follows:

- Annex F Eurocode 2 part 1-1 (modified by Eurocode 2 part 2, and completed by clause 6.109) shows the justification of a concrete membrane
- Annex LL of Eurocode 2 part 2 describes the way the stresses are broken down into the three layers, and the way the plate is justified
- Annex MM of Eurocode 2 part 2 is a simplified application of annex LL for webs

IV.4.1.General verification principle

The proposed method is iterative. It is much simpler to explain and to apply for a verification, but it may also be used in dimensioning. The approach to verification is described below.

The first step is to know if the plate is cracked or not. For this, the stresses are calculated on the whole thickness of the plate, on the upper face, in the center and on the bottom face of the plate, assuming it not to be cracked. Equation (LL101) then gives a criterion that determines if the plate is cracked or not. This criterion is expressed according to the principal stresses, which generalize in 3 dimensions the criteria of cracking by shear/tension.

If the plate is completely uncracked, it is sufficient to simply verify that the maximum principal compressive stress is less than f_{cd} .

The plate will now be assumed to be cracked by application of the preceding criterion. If the thickness of the layers is known, the verification principle is as follows:

- The plate stresses are broken down into membranous stresses
- It is verified that each layer may balance the stresses to which it is subjected, by applying the rules of annex F for the external layers, and the rules of annex LL for the intermediate layer.

If there is a combination of layer thicknesses such that verification of each of them is satisfied, the plate is capable of resisting the applied stresses.

The justification models of the various layers are shown below:

- Intermediate layer (annex LL)
- External layers (membrane model of annex F)

The order of this presentation stems from the fact that the stresses in the external layers depend upon the choices made for justification of the intermediate layer.

IV.4.2.Verification of intermediate layer





The intermediate layer is a plate subjected to two transverse shear stresses v_{Edx} and v_{Edy} . These two shear stresses combine vectorially into one principal shear stress v_{Ed0} in direction ϕ_0 [EC2-2 Anx.LL.(109), Expr.(LL.121) and (LL122)].

The justification is then brought back to that of a slab subjected to a shear stress v_{Ed0} . The applicable rules are those of clause 6.2 of Eurocode 2, modified by the national annex for slabs.

Firstly the necessity of shear stress reinforcement is evaluated, by applying the formula (6.2a). This formula brings in the rate of longitudinal reinforcement in the slab direction. Here the longitudinal reinforcement make an angle φ_0 with the direction of the principal shear stress, and it is possible to demonstrate that the rate of reinforcement to use is given by the formula (LL.123):

 $\rho_{l} = \rho_{lx} \times \cos^{2} \phi_{0} + \rho_{ly} \times \sin^{2} \phi_{0} = (A_{slx}/h) \times \cos^{2} \phi_{0} + (A_{sly}/h) \times \sin^{2} \phi_{0}$

where A_{slx} is the area of reinforcement per linear meter in direction x,

A_{sly} is the area of reinforcement per linear meter in direction y.

If the formula (6.2a) is verified, it is not necessary to use shear stress reinforcement.

If not, the truss model developed in 6.2.3 to dimension the shear stress steels is applied and the direction of the struts θ is chosen.

In the formula (6.2a), the longitudinal reinforcement in the outer layers should be taken into account.

There results from this an additional longitudinal stress $v_{Ed0} \times \cot\theta$ in the plate that will be equally distributed on the two outer layers. This stress is oriented toward the direction ϕ_0 . Its projection according to directions x and y gives the equations (LL.124 to LL.127). For example, for the additional stress in direction x:

 $n_{Edxc} = (v_{Ed0} \times \cot\theta) \times \cos^2 \varphi_0 = (v_{Ed0} \times \cot\theta) \times (v_{Edx} / v_{Ed0})^2 = v_{Edx}^2 / v_{Ed0} \times \cot\theta$ (LL.126)

IV.4.3.Membrane model

The external layers are verified using the membrane model developed in annex F, completed by clause [EC2-2 6.8.7 Expr.(6.109)].

The application of this annex poses no particular difficulties. The designer is referred to the text. The justification principle consists of superposing two models independently:

- The membranous tensions are taken up by the reinforcement
- The shear stresses are balanced by a system of struts and ties; the compression in the struts is verified by taking account of the general compression using an interaction criterion (6.109), and the tensions developed by the ties are added to the stresses in the reinforcement.

The figure below shows the interaction criterion (6.109) for a bi-compressed membrane, and compares it with the simpler criterion in annex F of Eurocode 2 part 1-1 ($\sigma < f_{cd}$).







Fig./Tab.1.1): Domain of resistance for a bi-compressed membrane

The application of this model shows that even a slab tensioned in the two directions can balance the shear stresses. In other words, there is no interaction between the resistance to shear at the level of the slab and the traction.

The model proposed allows in theory the choice of a completely free orientation of the struts, which would assume a perfectly plastic concrete. In practice, the struts orientation is conditioned by the behaviour of the membrane at SLS. A large rotation may lead unacceptable cracking. For this, annex F has a limitation given at the end of clause [EC2-2 Anx.F.1(104)].

IV.4.4.Breakdown of stresses on membranes

In the previous paragraphs it was seen how to justify the different layers. It remains to describe the way the plate stresses are broken down into membranous stresses. The formulae are given by equations (LL137) to (LL148).

The general principle consists in writing the equilibrium in axial force and in bending moment between the plate and membrane forces.

For example, with the sign notations and conventions of annex LL, for the breakdown of stresses n_{Edx} and m_{Edx} applied on the face with perpendicular axis x in two membranous stresses n_{Edxs} and n_{Edxi} , the equilibrium equations are:

 $n_{Edxs} \!\!+\! n_{Edxi} \!= n_{Edx}$

 $y_s \times n_{Edxs}$ - $y_i \times n_{Edxi} = m_{Edx}$

The equations (LL137) and (LL138) are simply deduced from them.

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If the plate is cracked by transverse stresses, the additional longitudinal stresses given by (LL126) should be added. The expressions (LL143) and (LL144) are deduced from them.

Finally, a special treatment should be given to the case (frequent) where the layers of steel are not centered in the outer membranes. This is the subject of the equations (LL149) and (LL150), which allow correction of the membranous stresses to take account of this eccentricity.

IV.4.5.Summary

In summary, the complete justification process of a cracked plate is as follows:

- choice of layer thicknesses (2 parameters)

- possible choice of θ if the plate is cracked under the transverse shear stress (1 parameter)

- for each membrane, choice of the inclination of the longitudinal shear stress struts (2 parameters)

If there is a combination of these 5 parameters such that the stresses in the concrete and the steels in the three layers are satisfied, then the plate is justified.

IV.5. Conclusion

The method called "sandwich" is a general method that appears satisfactory from a theoretical point of view, but whose implementation requires many calculations. In current cases, when it is possible, the simplified combination rules as described above will be favored. In more complex cases, for plates subjected to multiple stresses, the method in annex LL gives an overall, consistent answer to the question of dimensioning of the concrete plates and can help to define optimum quantities of reinforcement.

V.FOUNDATIONS

Eurocode 2 deals with foundations in certain of its clauses in an essentially different way according to the theme dealt with, but also in an incomplete way because it refers all problems of structure/ground interaction to Eurocode 7 "Geotechnical design" which includes two parts: EN1997-1 "General rules" and EN1997-2 "Ground interpretation and testing"

It has however been accepted by CEN that Eurocode 7 be mainly devoted to the fundamental requirements of geotechnical design and that it necessitates additions by other national standards. This is why the national annex to Eurocode 7 part 1 specifies that its application on French territory seek help from additional national standards, the major ones being, for engineering structure foundations:

- Standard NF P 94 261 for "Shallow foundations";
- And standard NF P 94 262 for "Deep foundations".

These standards are however still in development at the date of writing this guide, and while awaiting their publication the designer could usefully refer to the provisions of the Fascicle 62 title V of the CCTG, for what particularly concerns the conditions of consideration of the ground-structure interaction (particularly useful for the STR justifications of structural resistance and the geotechnical justification of the foundations GEO [EC0 6.4.1]). These provisions, in effect, compatible with the national annex to Eurocode 7 part 1, will be taken up by the additional national standards in preparation.





It thus appears useful to specify what Eurocode 2 brings to the designer for the foundations of his bridge projects.

Starting from the general principle where verification of a concrete structure comes under Eurocode 2 (STR), the problem is clearly illustrated by saying that a foundation footing, even a pile, will normally be designed using Eurocode 2 methods and rules once the effects of the ground-structure interaction have been taken into account.

It is initially important to find the factors linked to the method of execution of the geotechnical structures in the appropriate documents. This is the case for example of the compression limit value in the concrete of a bored pile, defined via a partial specific factor [EC2-1-1 2.4.2.5]. The national annex of Eurocode 2 part 1-1 then makes clear to the designer that he should refer to standard NF EN 1536 on bored piles [EC2-1-1/AN 3.1.2(2)P].

This is a general application principle concerning foundations to prevent all risk of uncertainty; moreover, this risk is low since in principle the development of additional specific standards takes account of the existence of Eurocodes.





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APPENDICES

Guide EC2 - Version V7





ANNEXE I.STRUCTURES USED FOR DIGITAL APPLICATIONS

The digital applications in this guide are based upon two calculation examples.

Common data

The structures proposed are assumed to support 2×2 lanes of normal traffic and are dual superstructures. Each deck has a 12.30m slab with the following:

- A crash barrier, type GS2 0.50m wide
- A left, lowered lane, 1.00m wide
- 2 lanes 3.50m wide
- An emergency stopping lane, 3.00m wide
- A type BN4 barrier, 0.80m wide.

Example of bridge built by balanced cantilever method

This is a single box structure in prestressed concrete built by balanced cantilever method with 3 spans at 65m - 100m - 65m, made up of a box girder with inclined webs of variable height of 2.75m to 5.90m (along 45m) and 12.30m wide.



Structures used for digital applications





: Example of bridge built by balanced cantilever method – transverse and longitudinal sections

Cutting into segments leads to a VSP 8.0m long, a connecting segment 2.0m long or 13 segments approximately 3.46m long and a curved part 16,0m wide in end spans.

The major characteristics of the sections on piers and connecting segment are:

- On piers $A_c = 9.50m^2$; $I_c = 52.73m^4$; v = 2.60m
- On connecting segment $A_c = 6.34m^2$; $I_c = 6.93m^4$; v = 0.95m

The concrete for the deck is C60/75 whose major characteristics are:

- $f_{ck} = 60 \text{ MPa}; f_{cm} = 68 \text{ MPa}$
- $E_{cm} = 39 \text{ GPa}$
- •





The prestress is made up of:

- beam tendons, 12T15S one pair per segment, or 13 pairs of tendons,
- fixed on two beds (9 + 4 pairs of tendons whose axis is at 0.13m and 0.22m from the upper fiber) and anchored on the segment section at 0.40m from the upper fiber,
- the major characteristics are:

 $E_p = 195 \text{ GPa}; f_{pk} = 1860 \text{ MPa}; f_{p0,1k} = 1637 \text{ MPa};$

 $\phi_{gaine} = 0.09m$; $\rho_{1000} = 25$; wedge draw-in 6mm; coefficient of friction $\mu = 0,19$; angular displacement k = 0.01.

- o internal continuity tendons 12T15S at 2 pairs on end spans and 3 on center span
- fixed in one bed (whose axis is at 0.13m from the lower fiber); from the V8 and V9 segment abutment for the end span and between the V7, V8 and V9 segments for the center span (anchored in a boss at 0.43m from the lower fiber),
- the major characteristics are identical to those of the beam tendons,
- external continuity tendons 19T15S on 2 spans at 2 pairs on end spans and thus 4 on center span.
- Fixed on one bed (whose axis is at 0.32m from the lower midspan fiber and at 0.28m from the upper fiber on pier) and anchored at the level of the beams of the segments on pier,
- The major characteristics are identical of those of the beam tendons.

The reinforcement are of type B500B for all steels.

The supports of this structure are made up of two piers, 32m and 21m high, on 6 piles of 1.60m diameter. The pier shafts are rectangular, 2.30m longitudinally by 4.60m transversely, and in C30/35 concrete.

Example of PSIDP

This is a structure of the PSIDP type, with 3 spans of 17.50m - 27.00m - 17.50m, made up of a slab with wide cantilever, 0.90m high and 12.30m wide.





Structures used for digital applications



: Example of PSIDP – transverse and longitudinal sections

The deck concrete is C35/45.

The prestress is made up of 20 12T15S tendons on 3 spans. These tendons have the same characteristics as those of the bridge constructed by balanced cantilever method. The layout is as per the following diagram:



: Example of PSIDP: tendon layout

For the requirements of some calculations (fatigue, control of cracking particularly), the same structure is dimensioned with a partial prestress. Its general characteristics are retained but the number of tendons is reduced to 15 type 12T15S. The eccentricities of the tendons on pier and keyed are then modified to be in compliance with the following diagram:



Structures used for digital applications





: Example of PSIDP: tendons profile of partial prestressing





ANNEXE II.GEOMETRIC IMPERFECTIONS

Example of articulated arc [EC2-2 5.2 (105) (106)]

Initial data

Major characteristics of the arc

C = 201,0m Length of cord F = 33,5m Deflection for a ratio $\frac{C}{F} = 6$ R = $\frac{C^2}{8F} + \frac{F}{2} = \frac{201^2}{8 \times 33,5} + \frac{33,5}{2} = 167,5m$ Radius of arc $\alpha = 2 \arccos\left(1 - \frac{F}{R}\right) = 2 \arccos\left(1 - \frac{33,50}{167,50}\right) = 1,287$ rd Angle of arc A = R × α = 167,5×1,287 = 215,57 m length of arc

Characteristics of constant section

- b = 8.00m Width
- h = 3.00m Height
- f_{ck} = 30MPa Characteristic resistance of concrete

Shapes of geometric imperfection:



: First modes of buckling of arc (Results of Sétra PCP software)



Appendix II - Geometric imperfectionsGeometric imperfections





: Shapes of geometric imperfections of the arc in the vertical and horizontal plans

Values of geometric imperfections in the vertical plan

$$\theta_{1} = \theta_{0} \times \alpha_{h} = \frac{1}{200} \min\left(\frac{2}{\sqrt{L}}\right) = \frac{1}{200} \min\left(\frac{2}{\sqrt{215,573/2}}\right) = 0,0009632 \text{ rd}$$

$$a = \theta_{1} \frac{L}{2} = 0,0009632 \times \frac{\frac{215,573}{2}}{2} = 0,0512 \text{ maximum amplitude of sinusoidal curve}$$

$$a_{i} = a \sin\left(\frac{\pi s_{i}}{l_{0}}\right) = 0,0512 \times \sin\left(\frac{\pi s_{i}}{215,573/2}\right) \text{ amplitude of two waves at curvilinear abscissa } s_{i}$$

Values of geometric imperfections in the horizontal plan

$$\theta_{1} = \theta_{0} \ \alpha_{h} = \frac{1}{200} \min\left(\frac{2}{\sqrt{L}}\right) = \frac{1}{200} \min\left(\frac{2}{\sqrt{215,573}}\right) = 0,00068109 \text{ rd}$$

$$a = \theta_{1} \frac{L}{2} = 0,00068109 \times \frac{215,573}{2} = 0,074 \text{ m} \text{ maximum amplitude of sinusoidal curve}$$

$$a_{i} = a \sin\left(\frac{\pi s_{i}}{l_{0}}\right) = 0,074 * \sin\left(\frac{\pi s_{i}}{215,573}\right) \text{ amplitude of wave at curvilinear abscissa } s_{i}$$

Example of a pier



Overall inclinations for geometric imperfections $\theta_i = \theta_0 \alpha_h = \frac{1}{200} \min\left(\frac{2}{\sqrt{L}}; 1\right)$ and derived eccentricities at top of piers:

- For L=25m $\theta_i = 0,002 \ 00rd$ or $e_i = 25,0 \times 0,002 = 0,05m (>e_0 = 0,02m)$ [EC2-1-1 6.1(4)]
- For L=45m $\theta_i = 0,00149 \text{ rd}$ or $e_i = 45 \times 0,00149 = 0,067 \text{ m} (> e_0 = 0,02 \text{ m})$ [EC2-1-1 6.1(4)]



Appendix III - Shear/torsion justifications - Digital applications.Justifications at ULS relative to fatigue



ANNEXE III.SHEAR/TORSION JUSTIFICATIONS - DIGITAL APPLICATIONS

The digital applications described below are relative and restricted to a single transverse section situated practically at the midspan of the center span of the bridge constructed by balanced cantilever method, called Section S(1/2) and whose transverse section is represented below:

SEES LARENEG SETN ARTNOCED SNO CES



: Transverse section of box girder

SECTIONS DE CONTRAINTES GENERALISEES	GENERALIZED SECTION STRESSES	

In the scope of an execution study, the justifications relative to shear should be established in numerous transverse sections. In each transverse section the two webs should be studied if the stresses are not symmetrical. Different local sections should also be considered such as the sections at the level of the gusset (for example A2, S1 and S2 in the figure).

To illustrate the following digital applications the only sections considered are A1, A3 and I1, justified in relation to shear and torsion due to general forces, taking no account of transverse bending effects or possible diffusion that normally should be analyzed elsewhere..

Characteristics of the box girder section at mid-span

Their geometric and mechanical characteristics are given in the following table:

GEOMETRIC AND MECHANICAL CHARACTERISTICS OF S(1/2)		Section S(l/2)
height of box girder	m	2.665
width of upper slab	m	12.30
exterior width of lower slab (mid-thickness)	m	5.564
thickness of upper slab	m	0.250
thickness of lower slab	m	0.222
height at upper mid-slab	m	2.540
area of <mark>bar</mark> Sx	m ²	6.299
reduced area of axis z Sz	m ²	1.725
torsion inertia of axis x Ix	m ⁴	11.82
inertia of axis y Iy	m^4	6.477



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GEOMETRIC AND MECHANICAL CHARACTERISTICS OF S(1/2)		Section S(l/2)
inertia of axis z Iz	m ⁴	60.19
distance from centroid to edge of box girder (y>0 side)	m	3.132
distance from centroid to edge of box girder (y<0 side)	m	3.132
distance from centroid to upper fiber	m	0.925
distance from centroid to lower fiber	m	1.739
radius of curvature in elevation of lower slab	m	368.18
coefficient of shear for web = $S/(I.b_w)$	m ⁻²	0.700
coefficient of shear relative to torsion Mx	m ⁻² /m	0.115
area enclosed by the centre-lines of the connecting walls A_k	m ²	13.579
effective depth according to web inclination (taken as 0.9h /cos8.6°) d=	m	2.398/cos8.6°
length of web (calculation of shear stress due to torsion) = $(h - half sum of slabs)/(cost angle inclination of web) = z_i$	m	2.457
inclination of web to vertical	degré	8.600
straight width of a web t=	m	0.320
horizontal thickness of a web b _w =	m	0.324

: Characteristics of S(1/2)

This table brings up the following comments:

- the area A_k is calculated by taking the trapezoid section passing by the mid-thickness of the slabs and the mid-thickness of the webs. The influence of the cantilevers in the resistance to torsion is thus ignored.
- the length of the torsion wall considered is that of the webs. It is the offset length taking account of the inclination of the webs between the average fibers of the slabs.

<u>Material characteristics</u>

They are detailed in the following table:

MATERIAL CHARACTERISTICS		
inclination of shear reinforcement in relation to average fiber α =	degree	90
characteristic cylinder strength of concrete f_{ck} =	MPa	60
characteristic strength of shear reinforcement f _{ykw} =	MPa	500
characteristic strength of longitudinal reinforcement f_{yk} =	MPa	500
partial factor on strength of concrete $\gamma_{\rm C}$ =		1.5
partial factor on strength of steels $\gamma_s =$		1.15
coeff α_{cc} recommended value = 1 [EC2-1-1 et EC2-1-1/AN 3.1.6]		1.0
coeff α_{ct} recommended value = 1 [EC2-1-1 et EC2-1-1/AN 3.1.6]		1.0
design compressive strength of concrete $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$	MPa	40.0
mean tensile strength of concrete f _{ctm} [EC2-1-1 Tab.3.1]	MPa	4.35
design tensile strength of concrete $f_{ctd} = \alpha_{ct} \times f_{ctk \ 0.05} / \gamma_{C} =$		2.03
design yield strength of shear reinforcement $f_{ywd} = f_{ykw}/\gamma_S$	MPa	434.78
design yield strength of longitudinal reinforcement $f_{yd}=f_{yk}/\gamma_S$	MPa	434.78



Appendix III - Shear/torsion justifications - Digital applications. Justifications at ULS relative to fatigue



: Material characteristics

The shear reinforcement are perpendicular to the average fiber of the structure due to the necessity of taking up the torsional shear stresses [EC2-1-1 9.2.3].

Results of calculation of general bending

Generalities

The overall design of the structure was led by the use of Sétra's ST1 software, taking account of the construction stages and the loads of Eurocode 1.

Extracts from the results useful for the different justifications and mentioned below stem from the envelopes of the SLS and ULS actions effects and correspond to the situation of the structure in service, to time infinity after all prestress losses undergone.

The results are determined for two cases of concomitance of stresses:

- Bending moments M_y extreme and other concomitant stresses
- Extreme shear stresses at the level of the center of gravity (section A1) and other concomitant stresses

In the results, the shear stresses take account of:

- the Résal effect due to inclination of the slabs
- the prestressing effects
- the torsion effect

Results at characteristic SLS

Sections	Section S(l/2)	Section S(1/2)
Extreme force and moment considered	Maximum shear	Maximum moment
Axial stress (MN)	42.701	40.200
Reduced shear stress T _z (MN)	-2.116	-1.128
Reduced shear stress with Résal effect (MN) = V_{Ed}	-2.175	-1.205
Torsion moment M_x (MN.m) = T_{Ed}	-1.306	0.280
Bending moment M _y (MN.m)	18.543	32.443
Axial stress upper fiber (MPa)	9.429	11.018
Axial stress lower fiber (MPa)	1.800	-2.330
Axial stress at center of gravity (MPa)	6.779	6.382
Shear stress at CDG (A1) (MPa)	-1.672	-0.811
Shear stress at base of web (A3) (MPa)	-1.377	-0.648
Axial stress at bottom of web (MPa)	3.007	-0.217
Shear stress in bottom slab (I1) (MPa)	-1.288	-0.547

Results at ULS

Sections	Section S(1/2)	Section S(1/2)
Extreme stress considered	Maximum shear	Maximum moment
Axial stress (MN)	42.917	42.914
Shear stress T _z (MN)	-2.856	-1.383



Appendix III - Shear/torsion justifications - Digital applications.Justifications at ULS relative to fatigue

Sections	Section S(1/2)	Section S(l/2)
Shear stress with Résal effect (MN) = V_{Ed}	-2.942	-1.493
Torsion moment M_x (MN.m) = T_{Ed}	-1.763	0.202
Bending moment M _y (MN.m)	36.647	53.081
Axial stress upper fiber (MPa)	12.050	14.397
Axial stress lower fiber (MPa)	-3.028	-7.441
Axial stress at center of gravity (MPa)	6.813	6.813
Shear stress at CDG (A1) (MPa)	-2.261	-1.021
Shear stress at base of web (A3) (MPa)	-1.862	-0.819
Axial stress at base of web (MPa)	-0.641	-3.985
Shear stress in bottom slab (I1) (MPa)	-1.741	-0.702
Mean axial stress in bottom slab (MPa)	-2.400	-6.532

It may be stated that the Résal effect does not reduce the value of the shear in this case. This is due to the fact that the very wide upper slab is highly compressed whereas the lower is little, even tensioned. The taking into account of the Résal effect is not thus very favorable if it is to be carefully calculated, taking account of the inclination of the two slabs on the neutral fiber.

Justifications relative to shear at ULS resistance

These justifications dealt with in [Chapter 6-II] and [Chapter 6-III] are illustrated here by the verification of webs of section S(1/2), completed by the study of the two sections I1 et A3.

In the example studied, there is a box girder with inclined webs. Determination of the shear due to torsion that act upon the webs is done by taking account of the offset lengths of the webs. The resisting shear force of the webs is determined with the lever arm of the elastic couple of the web according to the offset. The taking into account of the shear, to be consistent with that from torsion, should consider the shear redesigned according to the web.

Resistance of webs without shear reinforcement other than the minimum section

It should be verified [Chapter 6-II.2.1] if the resistant shear force of webs without transverse reinforcement is sufficient, and since a box girder is under consideration to assume the ratio ρ nil and to apply the expression:

$$V_{Rd,c} \ge (v_{min} + k_1 \sigma_{cp}) b_w d$$
 [EC2-1-1 Expr.(6.2b)].

 v_{min} being defined by $0.053/\gamma_C~k^{3/2}\times f_{ck}{}^{1/2}$

 $k_1 = 0.15$ (recommended value, adopted by the national annex)

 $f_{ck} = 60 \text{ MPa}$

$$\sigma_{cp} = 6.813$$
 MPa = mean axial stress considering the prestress

$$v_{min} = 0.053/\gamma_C k^{3/2} \times f_{ck}^{1/2} = 0.053/1.5 \times 1.289^{3/2} \times f_{ck}^{1/2} = 0.4004$$

(these values are those for beams, from the national annex)

$$V_{Rd,c} = (v_{min} + k_1.\sigma_{cp}) b_w d = (0.4004 + 0.15 \times 6.813) \times 0.32 \times 2.398 / \cos 8.6^\circ = 1.10 \text{ MN per web}$$
d taken according to the web offset

The resisting force per web is 1.10 MN for an acting force per web of $V_{Ed} = 2.942/2/\cos 8.6^\circ = 1.49$ MN.

The section S(1/2) can thus not be justified with the minimum reinforcement section. It is thus necessary to foresee and dimension shear reinforcement and justify the section accordingly.



Appendix III - Shear/torsion justifications - Digital applications. Justifications at ULS relative to fatigue



For information, if the resistance of the section without reinforcement had been sufficient justification of the section would have ended by determination of minimum web reinforcement sections, as follows:

This minimum section is given by the expressions [EC2-1-1 Expr.(9.4) et (9.5)]

 $\rho_{\rm w} = A_{\rm sw} \, / \, (s \times b_{\rm w} \times sin\alpha \,)$

and $\rho_{\rm w} > \rho_{\rm w,min} \ with \ \rho_{\rm w,min} \ = (0.08 \sqrt{f_{ck}} \) / f_{yk}$

 $f_{ck} = 60$ MPa et $f_{yk} = 500$ MPa, $\rho_{w,min} = 0.00124$, $b_w = 0.32$ m et sin $\alpha = 1$ (vertical steels)

 $A_{\mbox{\tiny sw}} \:/ \: s = 4 \mbox{ cm}^2/m$ and per web

Verification of webs requiring shear reinforcement

Section S(1/2) should be verified under the combination of shear and torsion.

Verification is done by applying expressions [EC2-1-1 Expr.(6.13), (6.14), (6.15)] for the shear and [EC2-2 6.3.2(104)] for torsion in the case of the box girder.

Verification of strut compression.

(i) <u>Shear force</u>

• $V_{Ed,i(V)} = 2,942/2/\cos 8.6^\circ = 1.49 \text{ MN}$

(ii) Torsional shear force

The torsional shear force in a web is given by:

$$V_{Ed,i(T)} = T_{Ed} \times z_i / 2A_k$$

with $A_k = 13.579 \text{ m}^2$, $T_{Ed} = 1.763 \text{ MN.m}$ and $z_i = 2.457 \text{ m}$ (offset length of web)
 $V_{Ed,i(T)} = 0.159 \text{ MN}$

(iii) <u>Resistant shear force</u>

 $V_{Rd,max} = \alpha_{cw} b_w z v_1 f_{cd} (\cot\theta + \cot\alpha)/(1 + \cot^2\theta)$

[EC2-1-1 Expr.(6.14)]

The recommended value of α_{cw} , validated by the national annex is as follows:

1	for non prestressed structures	
$(1+\sigma_{cp}\!/f_{cd})$	for $0 < \sigma_{cp} \leq 0.25~f_{cd}$	[EC2-1-1 Expr.(6.11.aN)]
1,25	for 0.25 $f_{cd} < \sigma_{cp} \leq 0.5 ~f_{cd}$	[EC2-1-1 Expr.(6.11.bN)]
$2,5 (1 - \sigma_{cp}/f_{cd})$	for 0.5 $f_{cd} < \sigma_{cp} < 1.0 f_{cd}$ [EC2-1	-1 Expr.(6.11.cN)]

With $\sigma_{cp} = 6.813$ MPa and $f_{cd} = 40$ MPa, $\sigma_{cp}/f_{cd} = 0.170$ whence $\alpha_{cw} = 1.170$

A maximum value of $\cot\theta$ leads to minimizing the section of vertical reinforcement and to increasing the compressive stress of the struts. $\cot\theta = 2.5$ is chosen as a starting point for the verification.

The webs are simultaneously subjected to shear and transverse bending. In this case the verification of the noncrushing condition of the concrete should be done by reducing the thickness of concrete in the compressive bending zone [Chapter 10-IV.3.3].



Appendix III - Shear/torsion justifications - Digital applications. Justifications at ULS relative to fatigue

A parallel bending calculation led to evaluation of the compressed bending zone (zone strictly necessary) to 2cm, a value to deduct from the web width.

Whence $b_w = 0.32-0.02$ m = 0.30 m net web width

z = lever arm of elastic couple of web, here taken as equal to the offset length = 2.457 m

$$v_1 = v = 0.6 \left[1 - \frac{f_{ck}}{250} \right] = 0.6 \times (1-60/250) = 0.456$$

 $\cot \alpha = 1$ since the steels are inclined at 90° to the neutral fiber (in fact the steels are vertical, very slightly inclined to the neutral fiber)

whence $V_{Rd,max} = 5.42$ MN for a web

(iv) Verification under combined shear and torsion

Must verify: $V_{Ed,i(V)} + V_{Ed,i(T)} \leq V_{Rd,max}$

It comes:

$$1.49 + 0.159 = 1.65 < 5.42$$

Compressive strength of struts is thus assured.

Calculation of shear reinforcement section perpendicular to the medium fiber

(i) Section for a web

According to clause [EC2-1-1 6.3.2(102)] shear and torsion are combined and the calculation principles defined for the shear force are applied.

The stresses are given in the preceding table "Results at ULS".

The shear force for a web is: $V_{Ed,i(V)} = 1.49$ MN

The torsional shear force in a web is: $V_{Ed,i(T)} = 0.159$ MN

The reinforcement section is obtained by equaling the total shear force to V_{Rd,s} (resisting force supplied by reinforcement)

whence

 $A_{sw}/s = (V_{Ed,i(V)} + V_{Ed,i(T)})/(z f_{vwd} \cot\theta)$

z = lever arm of elastic couple of web, here taken as equal to the offset length = 2.457 m

 $f_{ywd} = 434.78 \text{ MPa}$

 $\cot\theta = 2.5$

 $A_{sw}/s = (1.49+0.159)/2.457/434.78/2.5 = 6.2.10^{-4} \text{ m}^2/\text{m} = 6.2 \text{ cm}^2/\text{m}$

Maximum section for a web. (ii)

It may be verified that the necessary section does not exceed the maximum useful section given by:

$$\frac{A_{sw, max} f_{ywd}}{b_w s} \leq \frac{1}{2} \alpha_{cw} v_1 f_{cd}$$

$$\alpha_{cw} = 1.17$$
[EC2-1-1 Expr.(6.15)]

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 $v_1 = 0.456$

From this:

 $A_{\text{sw,max}}/s = 0.5\alpha_{\text{cw}} \nu_1 \ f_{\text{cd}} \ b_{\text{w}} \ / f_{\text{ywd}} = 0.5 \times 1.17 \times 0.456 \times 40 \times 0.32 \ / \ 434.78 = 78.5 \text{.}10^{-4} \ m^2/m = 78.5 \ cm^2/m = 78.5 \ c$

Longitudinal force due to shear and torsion

The calculation is done with the case of the load giving the maximum shear stress.

Section S(1/2) has as geometric characteristics

width of lower slab l = 5.564 m

thickness of lower slab e = 0.222 m

torsion area $A_k = 13.579 \text{ m}^2$

gives stresses at mid-thickness of lower slab: $\sigma = -2.400$ MPa

and is subjected to the following actions effects:

 $V_{Ed} = 2.94 \text{ MN}$ $M_{Ed} = 36.65 \text{ MN.m}$ et $T_{Ed} = 1.76 \text{ MN.m}$

Longitudinal forces due to shear

The inclination of the truss struts produces an additional tensile force in the longitudinal reinforcement of the lower fibers:

$$\Delta F_{td,V} = 0.5 V_{Ed} (\cot\theta - \cot\alpha) \qquad [EC2-1-1 Expr.(6.18)]$$

As the shear reinforcement are perpendicular to the medium fiber

 $\alpha = 90^{\circ}$ and $\cot \alpha = 0$

As chosen $\cot\theta = 2.5$ for the dimensioning of the shear reinforcement

$$\Delta F_{td,V} = 1.25 ~V_{Ed}$$

 V_{Ed} = 2.942 MN whence $\Delta F_{td,V}$ = 3.68 MN for the lower slab unit.

The additional longitudinal tensile force due to the shear force, to the linear meter, is thus:

 $\Delta F_{td,V}$ = (3.68/5.50) = 0.661 MN/ml

Longitudinal forces due to torsion

Torsion necessitates longitudinal reinforcement whose section may be calculated according to:

$$\frac{\sum A_{sl} f_{yd}}{u_k} = \frac{T_{Ed}}{2A_k} \cot \theta \qquad [EC2-1-1 \text{ Expr.}(6.28)]$$

where u_k is the perimeter of the area A_k

Each member of this identity is equivalent to a force per linear meter of wall, which gives:

 $\Delta F_{td,T} = T_{Ed} \cot \theta / 2A_k$

The additional longitudinal tensile force due to torsion is:



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 $\Delta F_{td,T} = 1.763 \times 2.5/2/13.579 = 0.162 \text{ MN/ml}$

Upper limit of additional longitudinal force

Section S(l/2) is near mid-span where the maximum moment $M_{Ed,max}$ is produced. The upper limit of the longitudinal tensile force due to shear alone may thus intervene. It is given by [EC2-1-1 Expr.(6.18)].

$$\begin{split} M_{Ed}/z + \Delta F_{td,V} &\leq M_{Ed,max}/z \\ \text{with} \qquad M_{Ed,max} = 53.08 \text{ MN.m} \text{, the value of the maximum moment in the span.} \end{split}$$

or $\Delta F_{td,V} \le (M_{Ed,max} - M_{Ed})/z = (53.08-36.65)/2.234 = 7.36$ MN or 7.36/5.564 = 1.32 MN/ml

The whole of the longitudinal tensile force due to shear 0,661 MN/ml is to be taken into account.

Determination of longitudinal reinforcement sections in the lower slab

The section of reinforcement in the lower slab is made up of reinforcement to resist the whole of the longitudinal bending, completed where applicable by reinforcement to take up the additional longitudinal forces from the shear forces determined above.

Reinforcement necessary to take up additional horizontal forces

The additional longitudinal forces from the tangential forces may be taken up by additional steels and/or by the stress increase of bonded prestressing steels [Chapter 6-II.2.3].

(i) Force able to be taken up by stress increase of prestressing tendons

In the section considered, the lower slab has four joined 12T15S tendons injected with cement grout, hence bonded, whose total force calculated concomitantly with the dimensioning forces factored in, is:

 $F_p = 8.54$ MN. (permanent state)

The tension in the strands is thus $\sigma_p=8.54/4/0.0018 = 1186$ MPa

With strands of characteristic tensile strength $f_{pk} = 1860$ MPa, $f_{p0,1k} = 0.9 f_{pk}$, the possibility of stress increase is:

 $\Delta \sigma_{\rm p} = 0.9 \times 1860/1.15 - 1186 = 270$ MPa (value largely below stress increase limit of 500 MPa)

and the force that might be taken up by stress increase of the 4 tendons is thus:

 $\delta F_p = (270 \times 4 \times 12 \times 150 \times 10^{-6}) = 1.94$ MN or 0.35 MN/ml of lower slab.

(ii) Calculation of additional reinforcement

Additional reinforcement are necessary because the additional longitudinal tensile force (0,661+0.162=0.823 MN/ml) cannot be totally taken up by stress increase of the bonded tendons (force of 0,35 MN/ml).

Dimensioning of longitudinal reinforcement

Annexe IV.

The summary of forces is as follows:

additional longitudinal tensile force:

longitudinal tensile force from axial forces:

 $\Delta F_{td,V+T} = -0.823 \text{ MN/ml}$ $F_{h} = -0.533 \text{ MN/ml}$



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force taken up by stress increase of bonded tend	ons:	$\delta F_p = 0.35 \text{ MN/ml}$
resulting tensile force:	F= -0.8	23-0.533+0.35= -1.006 MN/ml

The section of longitudinal reinforcement is:

 $A_s = F/f_{yd} = (F_h + \Delta F_{td,V+T} - \delta F_p)/f_{yd} = 1.01/434.78 = 23.14.10^{-4} \text{ m}^2/\text{m} = 23.14 \text{ cm}^2/\text{ml of slab}$

(or $11.6 \text{ cm}^2/\text{m per face}$)

or a total area for the slab of 128.75 cm^2 .

With the case of load giving the maximum moment, the same calculation of this section would lead to a resulting tensile force of -1,12 MN/m, or a section of longitudinal reinforcement of:

 $25,74 \text{ cm}^2/\text{m}$ or 143 cm^2 for the whole of the slab.

This reinforcement section should be retained .

Dimensioning of longitudinal reinforcement using Sétra's CDS software

Sétra's CDS software allows direct calculation of strains and stresses in a section of prestressed concrete by taking account of the stress increases of the prestressing tendons between the state under permanent load and the state under ULS loads. The bi-linear stress-strain diagrams with inclined upper branch [EC2-1-1 Fig.3.8 and Fig.3.10] for the reinforcement and prestressing steels may be and are used for this application.

The method applied is as follows:

The stresses and strains of the section considered are calculated under the actions effects (N, M) ULS with minimum reinforcement. In this case that taken into account is the minimum-skin reinforcement given by [EC2-2/AN 9.1(103)] and explained in [Chapter 9-II] of this guide. The resistance of the section is verified and where applicable the amount of additional reinforcement to ensure this resistance.

Under theses ULS actions effects (N, M), is obtained:

 σ_{pE} stress in interior joined tendons in concrete

 ϵ_{pE} elongation in these tendons

 $\sigma_{sE} \, stress$ in reinforcement in lower slab

 ϵ_{sE} elongation in this reinforcement

For prestressing steel, the elongation remaining is deducted before reaching the maximum design elongation ϵ_{ud} . It is the difference between the calculated elongation and the maximum design elongation ϵ_{ud} (see [EC2-1-1 Fig.3.10]): $\delta\epsilon_{pE} = \epsilon_{ud} - \epsilon_{pE}$.

The strain ε_{ud} is associated with the stress σ_{ud} . The increase in stress possible in the tendons is: = $\sigma_{ud} - \sigma_{pE}$

In the bottom slab the center of gravity of the reinforcement is at the same level as that of the internal tendons. They thus undergo the same strains. The additional strain for the reinforcement is thus: $\delta \varepsilon_{sE} = \delta \varepsilon_{pE}$

The maximum possible strain for the reinforcement is then: $\epsilon_s = \epsilon_{sE} + \delta \epsilon_{sE}$

This is subtracted from the maximum possible stress σ_s for the reinforcement according to the stress-strain curve (see [EC2-1-1 Fig.3.8]) and the variation in stress possible $\delta\sigma_s = \sigma_s - \sigma_{sE}$.

The area of the bottom slab tendons is A_p , that of the reinforcement A_s .



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The "reserve" force of the section is thus:

 $\Delta F_{\text{R}} = \delta \sigma_{\text{p}}.A_{\text{p}} + \delta \sigma_{\text{s}}.A_{\text{s}}$

The horizontal force from the shear and the torsion is $\Delta F_{td,V^+T}$.

If $\Delta F_R > \Delta F_{td,V+T}$ there is no need for additional reinforcement to take up the horizontal force from the shear force. In the opposite case its section is calculated with the stress σ_s determined previously.

 $\Delta A_{s} = (\Delta F_{td,V+T} - \Delta F_{R}) / \sigma_{s}$

	Section S(1/2)	
Form	Maximum	Maximum
Force	shear	moment
Axial SLS force QP (MN)	42.913	42.913
Bending moment M _y SLS QP(MN.m)	0.043	0.043
Force of tendons of bottom slab (MN)	2.154	2.154
Force of beam tendon (MN)	1.937	1.937
axial ULS force (MN)	42.917	42.914
Bending moment M _y ULS (MN.m)	36.647	53.081
Shear force reduced with Résal effect (MN) = V_{Ed}	-2.942	-1.493
Longitudinal tensile force MN	3.69	0.0
Longitudinal tensile force due to torsion MN	0.901	0.02
Total longitudinal force MN	4.59	0.02
ULS results		
Sections of steels in bottom slab in cm ²		
BA steels necessary in bending	0.00	86.73
Additional shear and torsion steels	70.97	0.12
Total steels in bottom slab	70.97	86.85

Nota: In the case of maximum shear all the internal forces from bending and part of the horizontal forces from shear and torsion are taken up by the stress increase of the tendons. The more accurate CDS calculations allow savings of steel.

Verification of shear between a web and a flange (example of the section at the junction of the bottom slab with the gusset section II)

The justification is done in compliance with [EC2-1-1 6.2.4] and is dealt with in this guide in [Chap 10.I.2].

The bottom slab is under tension even before taking into account the forces from the shear and torsion. In this case the compressive struts' inclination θ_f may be chosen such as $\cot\theta_f = 1.25$ whence $\theta_f = 38.6^{\circ}$.

The table in paragraph "results at ULS" above [Appendix III-4.3] gives $v_{Ed} = 1.74$ MPa.

This value does not take account of the reduction in thickness linked to a concomitant transverse bending moment.

 $v_{Ed} \le v f_{cd} \sin \theta_f \cos \theta_f$. Must be verified

 $v_{Ed} = 1.74 < 0.456 \times 40 \times sin(38.6) \times cos(38.6) = 8.9$ MPa.



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The gap between the effective stress and the acceptable stress shows that taking into account a possible transverse bending should not affect the shear resistance. For this section it may be assumed that the transverse bending is negligible.

The shear reinforcement are determined by:

$$A_{sf}/s_{f} \ge v_{Ed} h_{f}/cot\theta_{f/} f_{yd}$$

 $A_{\rm sf}\,/s_{\rm f} {\geq}\, 1.74 \times 0.222/1.25/434.78 = 0.00071 \ m^2/m = 7.1 \ cm^2/m.$

These reinforcement are to be distributed on the two faces of the slab.

Verification of a re-concreted section (example of the section at base of web – section A3)

The choice of this section brings in the previous determination of the suspension forces induced in the webs by the lower slab. These forces are to be taken into account in the study done of all horizontal sections at the web level.

Slab suspension forces

These forces are due to:

self weight of slab q_{cp} ,

the **bulge** created by the slab curvature,

the bulge created by the curvature of the joined tendons contained in the slab and at the web level.

If: R is the elevated radius of the concave side of the bridge,

F_h the force developed by longitudinal forces in the slab,

F_p the force of the prestressing tendons,

then:

- the bulging due to the curvature of the slab is q= F_h/R,
 (F_h is directed upwards if it is compression, downwards if it is tension),
- the bulging due to the tendons is $Q=F_p/R$. It is directed downwards.

The digital application gives:

Width of slab between webs: l'= $5.564-0.324 \times 2 = 4.916$ mLongitudinal tensile force of slab: $F_h = 6.532 \times 0.222 = 1.45$ MN/mlTotal force in 12T15S tendons $F_p = 4 \times 0.0018 \times 0.8 \times 1860 = 10.71$ MN (losses not deducted)Radius of curveR = 368.182 m

The summary for a web is thus the following (minus sign means upwards):

Slab weight $q_{cp} = 0.5 \times 4.916 \times 0.222 \times 0.025$ = 0.0136 MN/mlTensile bulging $q = 0.5 \times 4.916 \times 1.45 / 368.182 = 0.0097$ MN/mlTendon bulgingQ = 10.71 / 2 / 368.182= 0.0145 MN/ml



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[EC2-1-1 Expr.(6.25)]

Summary of suspension forces for a web

F = 0.0136 + 0.0097 + 0.0145 = 0.0378 MN/ml.

This force requires, per web, an addition of transverse steels of:

 $A_{sw} = 0.0378/434.78 = 0.87.10^4 \text{ m}^2/\text{ml} = 0.87 \text{ cm}^2/\text{ml}$

Calculation of shear stress (section A3)

The shear stress at ULS is obtained from the expression $v_{Edi} = V_{Ed} \times S / (I_y \times b_i)$ where

S is the static moment at the level of the re-concreting

 I_y is the inertia of the section

It is $v_{Edi} = 1.86$ MPa

Shear stress limit in the section-calculation of section of reinforcement necessary

The shear stress limit along the re-concreting is given by:

 $v_{Rdi} = c f_{ctd} + \mu \sigma_n + \rho f_{yd} (\mu \sin \alpha + \cos \alpha) \le 0.5 \nu f_{cd}$

with an upper value of 0.5 v $f_{cd} = 0.5 \times 0.456 \times 40 = 9.12$ MPa (this upper value cannot be reached).

 $c{=}~0.45$ and $\mu{=}~0.7$ considering a rough re-concreting showing asperities at least 3mm high, spaced at approximately 40 mm

 f_{ctd} = 2.03 MPa = acceptable tensile strength of concrete

 σ_n = axial stress at interface. As the web is subjected to vertical tension due to suspension of the slab of F = 0.0378 MN/ml, c×f_{ctd} = 0 must be used

 $\sigma_n = -0.0378/0.32 = -0.118$ Mpa.

 $\alpha = 90^\circ$ = inclination of steels to re-concreting

 $\rho = A_s/A_i =$ ratio of steels crossing the re-concreting added to the surface

Respect of the shear stress limit then allows calculation of the ratios of steels necessary to ensure equilibrium of the re-concreting in equalling v_{Edi} et v_{Rdi} :

 $v_{Edi} = 1.86 \le v_{Rdi} = 0 - 0.7 \times 0.118 + \rho \times 434.78 \times (0.7+0) = -0.0826 + 304.35 \times \rho$

Then $\rho \ge 0.0064$, or $A_s = 0.0064 \times 32 \times 100 = 20.5 \text{ cm}^2/\text{m}$.

The area of reinforcement necessary without re-concreting would be:

 $A_{sw}/s = 6.71 + 0.87 = 7.58 \text{ cm}^2/\text{m}$ (combination of area of shear and suspension steels).

Comparison of the two values shows the significance of avoiding re-concreting.

Verification of webs at characteristic SLS

Annexe V.

Verification of the shear resistance at SLS was shown in [Chapter 7-II.5.2II.5.1].

Although not specified in the Eurocodes, the verification should be done in all web sections: at the center of gravity, at the junction with the lower slab, at the junction with the upper slab, etc.

The verification deals here with the section at the center of gravity where shear is at a maximum, and the section at the base of the webs where the axial stress is minimal.



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The results of the calculations are in figure [Appendix III-Fig./Tab.(4)] below which also shows that:

- the conditions imposed by the Eurocodes are less restrictive than those of previous practices, since the acceptable shear stress is higher,
- the calculated stresses in this example [Appendix III-4.2] are clearly lower than the acceptable stresses.



: Acceptable shear stress at characteristic SLS

Taux admissible MPa	Acceptable rate MPa
Contrainte normale sigma x MPa	Axial stress sigma x MPa
tau adm pouret	Acceptable rate forand
Contraintes de calcul au centre de gravité	Design stresses at center of gravity
Contraintes de calcul en bas des âmes	Design stresses below the web

The sections thus show no insufficiency.

For the whole of the structure this verification would be determining only if the web thickness dropped to less than 0.17 m.



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JUSTIFICATIONS AT ULS RELATIVE TO FATIGUE

Location of fatigue loads

Trucks circulate in the axis of the slow lane. Location of slow lane is defined in accordance with Eurocode 1 part 2.

According to clause [EC1-2 4.6.1(4)]:

"all fatigue load models should be placed centrally on the notional lanes defined in accordance with the principles and rules given in 4.2.4(2) and (3). The slow lanes should be identified in the design."

Clause [EC1-2 4.2.4] stipulates:

"(2) For each individual verification, the number of lanes to be taken into account as loaded, their location on the carriageway and their numbering should be so chosen that the effects from the load models are the most adverse.

(3) For frequent and fatigue representative values and models, the location and the numbering of the lanes should be selected depending on the traffic to be expected in normal conditions."

It is thus up to the client to define the location of the slow lanes, "according to the normally foreseeable traffic". This choice is to be contemplated case by case for each structure, in anticipating future traffic possibly different from that foreseen at the design stage, perhaps leading to a different distribution of the lanes during the design life of the structure.

In the PSIDP study on longitudinal bending, the following transverse sections show two illustrated cases.

In the 1st case, the location of the slow lane on the right lane corresponds to initial marking of pavement at start of service.

However, to anticipate possible changes in motorway with three lanes per direction, a second case was envisaged, with a location of the slow lane on the notional lane beside support beam of safety barrier.





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: Location of slow lane and fatigue loads

In transverse bending, location of fatigue loads is very important. As in the previous example, two cases may be envisaged.

1st case:on actual right lane

2nd case: on notional lane beside safety device

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: Location of slow lane and fatigue loads

In the first case, the addition of a traffic lane by removal of the emergency slip road is considered as improbable. Passing of fatigue loads thus produces no stress and no damage by fatigue in the restraint section.

On the contrary, in the second case, which allows the possible change of the motorway with three lanes per direction, justification relative to fatigue becomes an important dimensioning criterion.

General method

This verification procedure for reinforcement is defined by [EC2-1-1 6.8.4]. It consists of calculation of damage from cycles of stress ranges and uses the reinforcement' S-N fatigue resistance curves.

Determination of cycles and stress ranges





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Fatigue study of a structure requires on the one hand knowledge of the heavy-vehicle traffic using it and on the other hand the effects of this traffic on the structure, translated in terms of tensile stresses in the reinforcement likely to be affected by fatigue. The traffic is known either as a result of records of actual traffic or from modeling of the traffic, based on other similar known traffic.

Determination of stress ranges is established from curves showing development of the tensile stress in the reinforcement studied during passage of trucks on the structure.

The passing of each truck on the structure gives rise to stress variations that may give different ranges. For each truck, the ranges $\Delta \sigma_{s1}$, $\Delta \sigma_{s2}$, ... $\Delta \sigma_{sj}$,...may be obtained by applying the "reservoir" method.



: Application of "reservoir" method

Effets	Effects
Essieu avant, essieu arrière	Front axle, rear axle
Robinet	faucet
Ouverture du robinet	Opening of tap
Hauteur d'eau	Water level

It is then necessary to analyze the various stress ranges by category of values $\Delta \sigma_{si}$ and determine their frequency of occurrence. The result leads to a spectrum of stress ranges $[(\Delta \sigma_{si}, n_i)_i]$, illustrated by the following graph. (n_i: number of occurrences in each category or number of cycles)


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Fig./Tab.(4): Spectrum of stress ranges

Failure number of cycles - Damage

The number of cycles $N_i(\Delta \sigma_{si})$ for a stress range $\Delta \sigma_{si}$ causing reinforcement failure by fatigue, is given by the S-N curves [EC2-1-1 6.8.4(1)]. They are the results of tests depending on a number of factors. Many cases are extensively studied to give the designers the practical information to be used for a justification.

On a logarithmic scale, the curve representing $\Delta \sigma_s$ function of N is bi-linear. $1/k_1$ and $1/k_2$ are the slopes of lines. Intersection of lines is positionned with the resisting stress range $\Delta \sigma_{Rsk}$ at N^{*} cycles. Some curves useful for the projects, corresponding to different types of reinforcement and placing conditions, have thus been studied and their parameters are defined in the tables EC2-1-1 Tab.6.3N et 6.4N], and reproduced below.



: S-N curve of reinforcement

Type of reinforcement	\ ⊺*	Stress exp	$\Delta \sigma_{Rsk}$ (MPa)		
Type of remitorcement	IN ·	\mathbf{k}_1	\mathbf{k}_2	at N* cycles	
Straight bars	106	5	9	fixed by A.N ⁽¹⁾	
Welded bars and wire fabrics	107	3	5	58.5	



(1)

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Splicing devices	107	3	5	35
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The national annex of Eurocode 2 part 1-1 gives the following values:

- $\Delta \sigma_{\text{Rsk}} = 210 \text{MPa}$ for $\phi \leq 16 \text{mm}$

- $\Delta \sigma_{Rsk} = 160 MPa$ for $\phi = 40 mm$

- linear interpolation for diameters $16mm < \varphi < 40mm$

: Parameters of S-N curves for reinforcing steels

Prostrossing steel	N!*	Stress e	$\Delta \sigma_{\text{Rsk}}$ (MPa)	
Freshessing steel	IN ·	\mathbf{k}_1	\mathbf{k}_2	at N* cycles
pre-tensionning	10 ⁶	5	9	185
post-tensionning :				
- single strands in plastic ducts	10^{6}	3	9	185
 straight tendons or curved tendons in plastic ducts 	10^{6}	3	10	150
- curved tendons in steel ducts	10^{6}	3	7	120
 splicing devices 	10 ⁶	5	5	80

: Parameters of S-N curves for prestressing steel

Criteria of verification

By definition of the S-N curves, fatigue failure occurs for a given value of $\Delta \sigma_{si}$, if the number of stress-range cycles applied n_i reaches its associated value N_i . The state of fatigue of the reinforcement may thus be characterized by the idea of damage D = n/N which should be less than 1 if their breakage is to be avoided.

In the use of S-N curves, the resisting stress range $\Delta \sigma_{Rsk}$ should be divided by the safety factor $\gamma_{s,fat} = 1.15$ defined by clause [EC2-1-1 2.4.2.4].

In a simple case, with a constant stress range $\Delta \sigma_s$, it's simply a question of verifying: $n \leq N$, N being the resisting number of cycles for $\Delta \sigma_s$.

In a more general case, with variable amplitudes, overall damage is calculated by applying the Palmgren-Miner rule of damage accumulation, $D_{Ed,i}$ being the damage produced by the n_i cycles of each range $\Delta \sigma_{si}$: $D_{Ed,i} = n_i/N_i$.

The verification **criterion** in a general case is then:

$$D_{ed} = \sum D_{ed,i} = \sum \frac{n_i}{N_i} \le 1$$

For each $\Delta \sigma_{si}$ of the stress-range spectrum, taking account of $\gamma_{s,fat}$, the corresponding value N_i for calculation of the damage caused by n_i cycles of application of $\Delta \sigma_{si}$ must be calculated. The expressions of N as a function of $\Delta \sigma_s$ are easily obtained from the half-line equations:

$$if \ \Delta \sigma_{s} \geq \frac{\Delta \sigma_{Rsk}}{\gamma_{s,fat}} \qquad \log(\Delta \sigma_{s}) - \log\left(\frac{\Delta \sigma_{Rsk}}{\gamma_{s,fat}}\right) = \frac{1}{k_{1}} \left(\log N^{*} - \log N\right) \qquad N = N^{*} \left(\frac{1}{\gamma_{s,fat}} \frac{\Delta \sigma_{Rsk}}{\Delta \sigma_{s}}\right)^{k_{1}}$$

$$if \ \Delta \sigma_{s} \leq \frac{\Delta \sigma_{Rsk}}{\gamma_{s,fat}} \qquad \log\left(\frac{\Delta \sigma_{Rsk}}{\gamma_{s,fat}}\right) - \log(\Delta \sigma_{s}) = \frac{1}{k_{2}} \left(\log N - \log N^{*}\right) \qquad N = N^{*} \left(\frac{1}{\gamma_{s,fat}} \frac{\Delta \sigma_{Rsk}}{\Delta \sigma_{s}}\right)^{k_{2}}$$





Other formulation of verification criterion

If stress ranges are smaller than $\Delta \sigma_{Rsk} / \gamma_{s,fat}$, only the second expression of N is to be used. **The damage D**_{Ed} for reinforcement subjected to a spectrum of stress ranges $[(\Delta \sigma_{si}, n_i)_i]$ may then be calculated by the following explicit relationship:



Fatigue load models

For application of the general method clause [EC1-2 4.6] proposes the two fatigue load models FLM4 and FLM5.

Model FLM5, the more general, involves using the actual traffic data and requires the use of appropriate elaborate software and hence will not be dealt with here.

Model FLM4, made up of 5 standard trucks producing effects equivalent to those of traffic typical of European roads, is more suited to a standard project verification.

It is described by table [EC1-2 Tab.4.7]:

	Truck type n°i		Axle spacing (m)	Equivalent axle loads (kN)	Proportion of trucks p _i (in %) according to traffic type		
i		Load (en kN)			Long distance	Medium distance	Local traffic
1		200	4,50	70 130	20	40	80
2		310	4,20 1,30	70 120 – 120	5	10	5
3		490	3,20 – 5,20 1,30 – 1,30	70 – 150 90 – 90 – 90	50	30	5
4		390	3,40 – 6,00 1,80	70 – 140 90 – 90	15	15	5
5		450	4,80 - 3,60 4,40 - 1,30	70 – 130 90 – 80 – 80	10	5	5

: Trucks of FLM4 fatigue load model

"Each standard lorry is considered to cross the bridge in the absence of any othe vehicle ". [EC1-2 4.6.5(3)].

In reality, according to the structure's geometry, the simultaneous presence of several trucks on the structure is conceivable. To obtain a complete picture, it is necessary to consider convoys of several trucks. The stress ranges due to each convoy will then depend upon the different types of truck making up the convoy and upon the distance between vehicles.

Clause [EC1-2 4.6 (2) note 2] shows that when the simultaneous presence of several trucks on the bridge cannot be ignored, it is advisable to use the FLM4 model only when completed by additional data specified in the national annex. According to the national annex, these additional data (distance between vehicles in the same lane, density of traffic in the different slow lanes) are then to be specified in the contract, for each individual project.





In practice, the hypothesis that considers only one truck is valid for small or medium-sized structures or elements (30m).

For large-sized structures, the accumulation of trucks on the same span gives greater stress variations than does one truck. For medium-length spans (< 30m), with truck lengths of the FLM4 model varying from 5 to 15m, this difference is less marked. Furthermore, the probability of having several trucks on the same span is small.

Determination of numbers of cycles

The passing of each truck of type i gives the stress ranges: $\Delta \sigma_{si,1}, \Delta \sigma_{si,2}, \dots \Delta \sigma_{si,j}, \dots$

The number of cycles of each range $\Delta \sigma_{si,j}$ is the number \mathbf{n}_i of trucks of type i using the structure, during its design life.

This number is obtained by multiplying the following data:

- N_{obs}: annual heavy-vehicle traffic
- **p**_i: proportion of trucks of type i in the PL traffic
- N_{years}: design life

The total number of type i trucks and range cycles $\Delta \sigma_{si,j}$ is thus $n_i = p_i \times N_{obs} \times N_{years}$.

The spectrum of stress variation may thus be expressed as $[(\Delta \sigma_{si,j}; \mathbf{n}_i)_j]_{i=1 a 5}]$.

The **design life** N_{years} is defined by the client. It is normally taken as 70 to 120 years.

The **proportion** \mathbf{p}_i of different types of truck may be determined from the preceding table, according to table [EC1-2 Tab.4.7], from the type of traffic.

The number of vehicles per year and by slow lane N_{obs} may be determined from the following table, in compliance with table [EC1-2 Tab.4.5], according to the traffic category.

	Categories of traffic	N _{obs} per year and per slow lane
1	Roads and motorways with 2 or more lanes per direction with high flow rates of lorries	$2.0.10^{6}$
2	Roads and motorways with medium flow rates of lorries	$0.5.10^{6}$
3	Main roads with low flow rates of lorries	$0.125.10^{6}$
4	Local roads with low flow rates of lorries	0.05.106

: Number of heavy vehicles per year and per slow lane

The type of traffic(local, medium distance, long distance) and the category of traffic are not linked. For example, a main road of "long distance" type with low truck traffic is a category 3 of traffic. Conversely, an urban road with 2x2 lanes with high heavy-vehicle traffic, made up essentially of twin-axle trucks (20T) is a category 1 of traffic and "local traffic" type.

The traffic category should not also be confused with the loading classification of road bridges defined in clause [EC1-2 4.2.2]. In effect, regardless of the traffic category, the majority of structures will be dimensioned





with a second classification loading. The structures with category 4 traffic could possibly support a 3^{rd} classification loading.

Damage calculation

With the use of the FLM4 model and hence a limited spectrum of stress ranges $[[(\Delta \sigma_{si,j;} n_i)_j]_{i=1 a 5}]$, the damage calculation is summed up, if the stress ranges are low enough, by application of the following expression:

$$(\text{with } n_i = p_i \times N_{obs} \times N_{years}) \qquad \qquad D_{ed} = \frac{N_{obs} \cdot N_{years}}{N^*} \left(\frac{\gamma_{s, fat}}{\Delta \sigma_{Rsk}} \right)^{k_2} \cdot \sum_{i, j} p_i (\Delta \sigma_{si, j})^{k_2}$$

"Equivalent" method of reinforcement verification

Eurocode 2 also proposes an "equivalent" method, from the simplified fatigue load model FLM3 made up of a single truck, reproducing the effects of traffic representative of European roads. The method is described in clause [EC2-1-1 6.8.5] and [EC2-2 Anx.NN].

The FLM3 fatigue load model described in clause $[EC1-2 \ 4.6.4]$ is made up of a four-axle truck, each with a weight of 120 kN.



: Fatigue model load FLM3

The fatigue strength of the reinforcement should be directly verified from an equivalent stress range $\Delta \sigma_{s,equ} = \lambda_s$. $\Delta \sigma_{s,EC}$, with:

- $\Delta \sigma_{s,EC} = \sigma_{s,max} \sigma_{s,min}$ is the maximum stress range causing by the truck FLM3 model calculated by determining the unfavorable ($\sigma_{s,max}$) and favorable($\sigma_{s,min}$) positions of the truck on the structure;
- λ_s : equivalent damage factor to be determined from annex [EC2-1-1 Anx.NN]. Determination of this coefficient will be illustrated more simply in numerical applications.

The verification criterion relative to fatigue is then:

$\Delta \sigma_{_{s,equ}} \leq \frac{\Delta \sigma_{_{Rsk}}}{\gamma_{_{s,fat}}}$
--

Calibration of the method

Parameter λ_s was calibrated so as to obtain an equivalence, relative to damage, between the number of cycles N^* of a stress range $\Delta \sigma_{s,equ} = \lambda_s \Delta \sigma_{s,EC}$ and a spectrum $[(\Delta \sigma_{i;} n_i)_i]$ due to a typical road traffic.

The general expression of damage is given by the relationship:
$$D_{ed} = \frac{1}{N*} \left(\frac{\gamma_{s,fat}}{\Delta \sigma_{Rsk}} \right)^{k_2} \cdot \sum_{i} n_i (\Delta \sigma_{si})^{k_2}$$





In replacing the spectrum $[(\Delta \sigma_{i;} n_{i})_{i}]$ *by* $(\Delta \sigma_{s,equ;} N^{*})$ *, the relationship becomes:*

$$D_{ed} = \frac{1}{N^*} \left(\frac{\gamma_{s,fat}}{\Delta \sigma_{Rsk}} \right)^{k_2} N^* \left(\Delta \sigma_{s,equ} \right)^{k_2} = \left(\gamma_{s,fat} \frac{\Delta \sigma_{s,equ}}{\Delta \sigma_{Rsk}} \right)^{k_2}$$

Condition $\mathbf{D}_{ed} \leq \mathbf{1}$ *is then equivalent to:*

 $\Delta \sigma_{s,equ} \leq \frac{\Delta \sigma_{Rsk}}{\gamma_{s,fat}}$

The adjustment factor λ_s was calibrated from traffic measurements and calculations on different types of structure (longitudinal bending) or elements (transverse bending) and from the relationships:

$$\lambda_{s} = \frac{\Delta \sigma_{s,equ}}{\Delta \sigma_{s,EC}} and \qquad \Delta \sigma_{s,equ} = k_{v}^{2} \frac{1}{N^{*}} \sum_{i} n_{i} (\Delta \sigma_{si})^{k_{2}} \quad (equivalence \ of \ damages)$$

Calibration of factor λ_s was established from calculation of moment variation, by accepting the hypothesis of a <u>linear relationship between stress</u> variations and moment variations. Even when this hypothesis is not verified, the method turns out to be safe.

Alternative simplified method for reinforcement

Clause [EC2-1-1 6.8.6] proposes a simplified rule of verification of fatigue strength of reinforcement of reinforced concrete. It is to verify that the stress range under a frequent cyclic load associated with the basic combination is such that [EC2-1-1/AN 6.8.6(1)]:

• $\Delta \sigma_s \leq 100 \text{ MPa}$ for unwelded reinforcing bars

 $\Delta \sigma_{\rm s} \leq 35 \text{ MPa}$ for welded reinforcing bars

The combination of fatigue used can, in the majority of cases, be the frequent combination bringing in the main load model LM1: $C_{fat} = C_0 + Q_{LM1,Fréq}$.

The envelope of the moments due to the frequent load model LM1 gives extreme values of forces allowing calculation of the stress range $\Delta \sigma_s = \sigma_{s,max} - \sigma_{s,min}$

 $\sigma_{s,max} \left(N_o; M_o + M_{LM1,Fréq,déf} \right) \qquad \qquad \sigma_{s,min} \left(N_o; M_o + M_{LM1,Fréq,fav} \right)$

Application to the justification of a PSIDP in longitudinal bending

Data

Project

In theory, each PSIDP section is to be verified relative to fatigue. In the following example, the verification will be limited to reinforcement and prestress at mid-span and supports area section.

Longitudinal prestress is made up of 15 12T15S tendons.

The reinforcement is made up of 23HA20, or a section of 72.25 cm², on top of the support area section, and at bottom on mid-span section.



Hypothesis of traffic – Design life

The hypothesis of a 2 x 2 road with high heavy-vehicle traffic is adopted. This traffic is characterized by:

- Category 2 traffic: $N_{obs} = 2.0.10^6$ trucks per year in the slow lane
- traffic of "long distance" type with a distribution of different truck types in the following proportions: $p_1 = 20$ %; $p_2 = 5$ %; $p_3 = 50$ %; $p_4 = 15$ %; $p_5 = 10$ %

The `design life of the structure is $N_{years} = 100$ years.

Characteristics of materials

The values of the S-N curve parameters are given in tables [EC2-1-1 6.8.4 Tab.6.3N and 6.4N]:

- Steel safety : $\gamma_{s,fat} = 1.15$;
- Reinforcement : $k_2 = 9$ $N^* = 10^6$ $\Delta \sigma_{RSk} = = 202 \text{ MPa}$

Pour $\phi = 20 \text{ mm}$, $\Delta \sigma_{RSk} = 210 - 50 \times (\phi - 16)/(40 - 16) = 202 \text{ MPa}$





• prestress: $k_2 = 7$ $N^* = 10^6$ $\Delta \sigma_{RSk} = 120 \text{ MPa}$

Basic combination

This combination is: $C_0 = G_{max} + P_{k,inf} + 0.6.\Delta T_M$

For bonded post-tension prestressing : $P_{k,inf} = r_{inf}P_{m,\infty}$, with $r_{inf} = 0.90$.

The permanent actions are combined with the ferquent value of the thermal gradient, or 60 % of the characteristic value. For the mid-span section, it is the thermal gradient tensioning the lower fiber that is unfavorable, and for the support area section a thermal gradient tensioning the upper fiber .

Which gives the following results (tension on reinforcement is given in absolute value):

Section at mi	d-span:	Supported section:			
$C_0 =$	$G_{max} + 0.9 \cdot P_{m,\infty} + 0.6 \cdot \Delta T_{M,heat}$	$C_{0} = 0$	$G_{max} + 0.9 \cdot P_{m,\infty} + 0.6 \cdot \Delta T_{M,Cool}$		
Forces :		Forces :			
N_0	= 29.95 MN	N_0	= 29.95 MN		
M_0	= 3.54 MN.m	M_0	= -2.90 MN.m		
Stress level :		Stress level :			
$\sigma_{c,sup}$	= 6.30 MPa	$\sigma_{c,inf}$	= 6.53 MPa		
σ_{P}	= 1109 MPa	$\sigma_{ m P}$	= 1109 MPa		
σ	compressed reinforcement	σ_{s}	compressed reinforcement		

Application of general method

Application of fatigue loads

In this medium-sized structure, it is accepted that the simultaneous presence of several trucks may be ignored. The FLM4 model that assumes that each truck crosses the structure in the absence of all other vehicles [EC1-24.6.5(3)] is thus well adapted to evaluate the stress ranges.

The slow lane is laid out on the notional lane beside support beam of safety barrier.

Note [EC1-2 4.6.1 Note 1] proposes the addition of 10 % of Nobs for each fast lane.





: Location of fatigue loads on lanes

Voie rapide	fast lane
Voie lente	Slow lane

The following are transverse distribution coefficients:

	Span 1 or 3	Span 2
Truck in axis of slow lane:	1.42	1.24
Truck in axis of fast lane 1:	1.07	1.04
Truck in axis of fast lane 2:	1.13	1.09

To simplify the sequence of the numerical application, the heavy-vehicle traffic in the fast lanes is not taken into account. As regards the transverse coefficients, the moment due to a truck centered in the fast lanes represents about 80% of the moment due to a truck centered in the slow lane. In this case, the taking into account of heavy-vehicle traffic in the fast lanes has only little influence on the justification relative to fatigue.

The following figures show the variation of the bending moment according to the longitudinal position of the various trucks in the slow lane. (coefficient of transverse distribution included).





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: Mid-span section – variation of moment due to trucks in slow lane





: Support P1 area section - variation of moment due to trucks in slow lane

The stress variations due to translation of trucks are combined with the stresses corresponding to the reference state. The curves showing the development of the global moment M_{fat} during truck movement are effectively the same as the previous curves after translation (following a vertical axis); $M_{fat} = M_0 + M_{Qfat}$

The axial force is constant: $N = N_0 = 29.95$ MN.



The following table gives the extreme values of the moment M_{Qfat} due to the fatigue loads and the global moment M_{fat} .

M _{Qfat} (in MN.m)		Truck 1		Truck 2		Truck 3		Truck 4		Truck 5	
		M_{Qfat}	M_{fat}	M_{Qfat}	M_{fat}	M_{Qfat}	M_{fat}	M_{Qfat}	M_{fat}	M_{Qfat}	M_{fat}
Mid-span	Max	0.910	4.447	1.429	4.966	1.749	5.286	1.345	4.882	1.457	4.994
	Min	-0.135	3.402	-0.210	3.327	-0.278	3.259	-0.211	3.326	-0.221	3.316
Support area	Max	0.117	-2.786	0.183	-2.721	0.242	-2.662	0.184	-2.720	0.192	-2.711
	Min	-0.613	-3.517	-0.953	-3.856	-1.385	-4.289	-1.079	-3.983	-1.195	-4.099

: Extreme values of moments M_{Qfat} and M_{fat}

Annexe VI.Calculation of stresses – stress ranges

The evolution of stress in reinforcement is calculated with following forces :

- N₀
- $M_{fat} = M_0 + M_{Qfat}$

Calculation of stresses is done from the reference state of the structure with a minimal initial stress at the boundarie of the section. The stress variations produced by the fatigue load are then determined in compliance with [Chapter 3-III.1.4.b)].

The following diagrams show the relation between the bending moment M_Q due to overloads and tension in reinforcement, overstressing in prestressing tendons, concrete compression in upper fibre (mid-span) and lower fibre (support area), for a section subjected to the axial force N_0 and to the bending moment $M_0 + M_Q$.



: Section at mid-span





: Supported section

Contraintes soussur appui avec	Stresses underon support areawith
Contrainte en Mpa	Stress in Mpa
Acier; prec; béton	Reinforcement ; prestress; concrete

These diagrams show the importance of the definition of a basic state in the fatigue combination. The relationships between the stresses and the moment are not linear. The variation in moment due to movement of a truck gives a stress variation depending upon the "unloaded" state.

For the supported section, the higher tensile stresses in the reinforcement appear if moments M_Q is smaller than -3,20 MN.m.

The graph [Fig./Tab.(16)] shows that the moment M_{Qfat} is higher than this value. The supported section is thus totally compressed under the translation of fatigue loads of the FLM4 model. There is no fatigue in the reinforcement of support area section.

In the case of mid-span section, the curve [Fig./Tab.(18)] shows that tension in bottom reinforcement appears when moments M_Q is higher 0.910 MN.m.

σ (in M	Pa)	Truck 1	Truck 2	Truck 3	Truck 4	Truck 5	
σ_{s}	Max	Max 0.281 3.722		6.276	3.113	3.936	
	Min	compressed	compressed	compressed	compressed	compressed	
$\Delta \sigma_{P}^{*}$	Max	3.779	6.618	8.741	6.114	6.796	
	Min	0.000	0.000	0.000	0.000	0.000	

* stress increase of prestressing steel

: Section at mid-span – Extreme stresses

This table gives extreme stresses and thus the maximum stress range $\Delta \sigma_{max} = \sigma_{max} - \sigma_{min}$. The curves of figure [Fig./Tab.(15)], show that translation of each truck generates a single stress variation (one peak). In effect,



decompression of covering area occurs only for moments higher than 0.910 MN.m. Only parts of curves above this value create tension in reinforcement as shown in the following figure:



: M_Q > 0,910 MN.m

The translation of type i truck produces only one stress range:

- for reinforcement: $\Delta \sigma_{s,i} = \sigma_{s,max,i}$
- for prestressing steel: $\Delta(\Delta \sigma_{P,i}) = \Delta \sigma_{P,\max,i}$ (variation of overstressing)

According to clause [EC2-1-1 6.8.2(2)], 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 .

Calculation of η is done from the following values:

- $A_s = 72,26 \text{ cm}^2$ $\phi_s = 20 \text{ mm};$
- $A_P = 15 \times A_{P,c\hat{a}ble} = 270 \text{ cm}^2$ $\phi_P = 1.6 \sqrt{A_{P,c\hat{a}ble}} = 67.88 \text{ mm};$
- $\xi = 0.50$ (Table [EC2-1-1 Tab.6.2]: post-tension, strands, concrete \leq C50/60)

Thus
$$\eta = \frac{A_s + A_p}{A_s + A_p \sqrt{\xi(\phi_s/\phi_p)}} = 1,95$$

The stress ranges applied to reinforcement of mid-span section are calculated according to relation stressmoment represented in figure [Fig./Tab.(18)]:

Type of truck i	1	2	3	4	5
$\Delta \sigma_{s,i} = \eta \times \sigma_{s,max,i}$	0.546	7.242	12.213	6.058	7.659
$\Delta(\Delta\sigma_{P,i})$	3.779	6.618	8.741	6.114	6.796

: Stress ranges (in MPa)

Calculation of factor damage





Stress ranges are all smaller than $\Delta \sigma_{Rsk} / \gamma_{s,fat}$. So factor damage must be calculated with stress exponent k₂ of the S-N curve and the expression previously determined :



Type of truck i		1	2	3	4	5	$\Sigma p_i . (\Delta \sigma_{si})^{k2}$
$p_i (\Delta \sigma_{si})^{k_2}$	k2 / pi	20 %	5 %	50 %	15 %	10 %	
Reinforcement	9	8.69.10-4	$2.74.10^{6}$	3.02 . 10 ⁹	1.65.106	9.07 . 10 ⁶	3.04 . 10 ⁹
Prestressing steel	7	$2.20.10^3$	$2.78.10^4$	1.95 . 10 ⁶	4.79 . 10 ⁴	6.69 . 10 ⁴	$2.09.10^{6}$

: Detail of damage calculations

The results are :

- $D_{ed} = 3.82.10^{-9}$ For reinforcement : < 1
- $D_{ed} = 3.11.10^{-6}$ For prestressing steel: < 1

The fatigue strength of the reinforcement is thus verified.

Mean stress - Stress limit

With damage values almost nil, reinforcement do not seem to suffer from fatigue.

Damage however is an exponential function of the stress ranges $\Delta \sigma_i$. It is thus advised to calculate the parameter:

$$\sqrt[4]{D_{ed}} = \sqrt[4]{\frac{N_{obs} \times N_{years}}{N^*}} \times \frac{\gamma_{s,fat}}{\Delta \sigma_{Rsk}} \times \sqrt[4]{k_2} \sum_{i,j} p_i (\Delta \sigma_{si,j})^{k_2}$$

The expression $\sqrt[k_2]{\sum_{i,j} p_i (\Delta \sigma_{si,j})^{-k_2}}$ is similar to an "average" stress range

The condition $D_{ed} < l$ or $\sqrt[k_2]{D_{ed}} < l$ comes then to the verification

$$\Delta \sigma_{\text{moy}} < \frac{\Delta \sigma_{\text{Rsk}}}{\gamma_{\text{s,fat}} \times K} \qquad \text{with} \qquad K = \sqrt[k_2]{\frac{N_{\text{obs}} \times N_{\text{years}}}{N^*}} \quad and \quad \Delta \sigma_{\text{moy}} = \sqrt[k_2]{\sum_{i,j} p_i (\Delta \sigma_{\text{si},j})^{k_2}}$$

The parameter $\sqrt[k_2]{D_{ed}}$ may thus be considered as a ratio of average stress range to stress range limit. This parameter is thus easier to interpret than damage D_{ed}.

The results are :

For reinforcement: $\sqrt[9]{D_{ed}} = 0.116 < 1$

 $\sqrt[7]{D_{ed}} = 0.163 < 1$ *For prestressing steel:*

Safety in relation to fatigue strength is satisfactory. However the parameter $\sqrt[k_2]{D_{ed}}$ gives values more perceptible by the designer ($\approx 15\%$) than D_{ed} ($\approx 10^8$).

The following tables show the variation of $\sqrt[k_2]{D_{ed}}$ as a function of D_{ed} .



When the fatigue variation is satisfactory, the values of D_{ed} will often be very low. A damage of 1 % of the reinforcement corresponds to $\sqrt[9]{D_{ed}} = 60\%$ of the acceptable stress range. The section is thus nearer to the fatigue ultimate limit stress than it appears from the damage value.



Figure: $\sqrt[k_2]{D_{ed}}$ as a function of $D_{ed} < 1$

When the verification is unsatisfactory, values of damage factor are higher. A factor damage of 5.0 for reinforcement corresponds to $\sqrt[9]{D_{ed}} = 1.20$. It is then sufficient to increase the reinforcement section by about 20%, to obtain a decrease in stresses of the same order, and damage values near to 1.0.



Application of "equivalent" method

Application of fatigue loads

The fatigue load Q_{fat} is in this case the truck of the FLM3 model. According to clause [EC2-2 Anx.NN.2.1(101)], the axle loads of the model should be multiplied by the following factors:

- 1.75 for a verification at intermediate supports of continuous bridges,
- 1.40 for all other sections.





It is important that this coefficient be multiplied to the load, and thus to the forces in the section, because relation between stress variations and moment variations is not linear .The figures [Fig./Tab.(18) or Fig./Tab.(19)] show that the stress variation $\Delta\sigma_s(\gamma_Q \times \Delta M)$ is greater than $\gamma_Q \times \Delta\sigma_s(\Delta M)$. ($\gamma_Q = 1.40 \text{ or } 1.75$)

The following figure shows the curve of variations of moments $M_{Qfat} = \gamma_Q M_{FLM3}$, for the support area and midspan sections, during the translation of FLM3 truck on the axis of slow lane (transverse factor included):



: Variation of moment $M_{\mbox{\scriptsize Qfat}}$

Stress range $\Delta \sigma_{EC}$

Under fatigue combination, the following forces are thus applied to the sections:

- \circ N_o = 29.95 MN
- $\circ \quad M_{fat} = M_o + \gamma_Q.M_{FLM3}$
- Support area section: $M_o = -2.90 \text{ MN.m}; \quad \sigma_{s,0} = \text{compressed} \quad \gamma_Q = 1.75$
- Mid-span section: $M_o = 3.54$ MN.m; $\sigma_{s,0} = compressed$ $\gamma_Q = 1.40$

For the support area section, tensile stress in top reinforcement appears if moment M_{Qfat} is smaller than -3,20 MN.m (application of general method).

The previous figure shows that the moment M_{Qfat} is higher than this value. As in the previous method, the support area section is thus completely compressed during crossing of FLM3 truck.

In the case of the mid-span section, tensile stresses in bottom reinforcement appears if moment M_{Qfat} is higher than 0.910 MN.m.

The extreme stresses are calculated with the following forces :

• $N_o = 29.95 \text{ MN}$



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• $M_{Qfat,min} = -0.39 \text{ MN.m}$ \rightarrow $\sigma_{s,min} = \text{compressed};$ $\Delta \sigma_{P,min} \approx 0$ • $M_{Qfat,max} = 2.40 \text{ MN.m} \rightarrow M_{fat,max} = 5.94 \text{ MN.m}$ \rightarrow $\sigma_{s,max} = 12.72 \text{ MPa};$ $\Delta \sigma_{P,max} = 14.13 \text{ MPa}.$

The stress ranges are :

- Reinforcement: $\Delta \sigma_{s,EC} = \eta . (\sigma_{s,max} \sigma_{s,min}) = 24.80 \text{ MPa}; \text{ (with } \eta = 1.95\text{)}$
- Prestressing steel: $\Delta \sigma_{P,EC} = \Delta \sigma_{P,max} \Delta \sigma_{P,min} = 14.13 \text{ MPa};$

Correction factor λ_s

According to the annex [EC2-2 Anx.NN] the correction factor λ_s includes the influence of span, annual traffic volume, design life, multiple lanes, traffic type and surface roughness and can be calculated by:

 $\lambda_{\rm s} = \lambda_{\rm s,1} \!\! \times \lambda_{\rm s,2} \!\! \times \lambda_{\rm s,3} \!\! \times \lambda_{\rm s,4} \!\! \times \phi_{\rm fat}$

(i) Factor $\lambda_{s,1}$

 $\lambda_{s,1}$ is a factor accounting for element type (eg. continuous beam) and takes into account the damaging effect of traffic depending on the critical length of the influence line or area.

The values of $\lambda_{s,1}$ are obtained from the abacuses of figures NN.1 and NN.2 of annex NN.

For the support area section, the values should be read on the abacus NN.1. This section, however, as previously seen, undergoes no damage.

For the mid-span section, the abacus NN.2 gives the values $\lambda_{s,1}$.

The length of the influence line, in abacus abscissa, should be considered as the span length, in the case of a continuous deck with several identical spans.

The coefficient $\lambda_{s,1}$ was calibrated on several types of structure, particularly on continuous structures with 3 spans of length 1 varying from 10 to 90 m. This length 1 is defined as the "length of the influence line" in annex [EC2-2 Anx.NN].

Where the structure has spans of different lengths, the following rules are adopted:

- verification in span: length of span

- verification in support area: average of lengths $l = (l_{i-1} + l_i)/2$

For verification of mid-span section, the length is thus l = 27.00 m.

Reading of the $\lambda_{s,1}$ is done on the curves dealing with continuous beams (suffix a):

- 3a) reinforced concrete steels ($k_2 = 9$) $\lambda_{s,1} = 1.18$
- 2a) curved tendons in steel ducts ($k_2 = 7$) $\lambda_{s,1} = 1.35$

(ii) Factor $\lambda_{s,2}$

This factor takes account of the volume of traffic. In the example considered, the traffic hypotheses correspond to the basic hypotheses of the "equivalent" method, i.e. a category 2 traffic ($N_{obs} = 2.0.10^6$) and long distance type (Q = 1.0).

Thus:

 $\lambda_{s,2} = 1.00$





Sétra

In use of the expression {EC2-2 Anx.NN Expr.(NN.103)], the volume of traffic is expressed in millions of trucks. The calculation is done with $N_{obs} = 2.0$ millions.

(iii) Factor $\lambda_{s,3}$

This factor takes account of the bridge design life. In the example considered, $N_{years} = 100$ years , is the basic hypothesis of the "equivalent" method.

 $\lambda_{s,3} = 1.00$

(iv) Factor $\lambda_{s,4}$

This factor takes account of the possible influence of heavy-vehicle traffic on other lanes. In application of the general method, the impact of heavy-vehicle traffic on the first fast lane, with volume of 10% of N_{obs}, was ignored. Taking account of this traffic would give a factor $\lambda_{s,4} \approx 1.01$. Thus:

 $\lambda_{s,4} = 1.00$

 $K \times \Delta \sigma_{moy} \leq \frac{\Delta \sigma_{Rsk}}{\gamma_{s.fat}}$

 $\Delta \sigma_{s,equ} \leq \frac{\Delta \sigma_{Rsk}}{\gamma_{s \text{ fat}}}$

(v) Factor ϕ_{fat}

Clause [EC1-2 4.6.1(6)] shows that "the fatigue load models 1 to 4 take account of a dynamic increase corresponding to pavement of good quality".

One thus adopts:		$\phi_{\text{fat}} = 1.00$
In conclusion,	for reinforcement	$\lambda_s = 1.18$
	for prestressing steel	$\lambda_s = 1.35$

Verification criterion

~ ·

This is verification of condition $\Delta \sigma_{s,equ} = \lambda_s \cdot \Delta \sigma_{s,EC} \leq \frac{\Delta \sigma_{Rsk}}{\gamma_{s,fat}}$ • reinforcement: $\Delta \sigma_{s,equ} = 29.26 \text{ MPa} \leq 176.00 \text{ MPa} \quad (\Delta \sigma_{Rsk} = 202 \text{ MPa})$ • prestressing steel: $\Delta \sigma_{P,equ} = 19.08 \text{ MPa} \leq 104.30 \text{ MPa} \quad (\Delta \sigma_{Rsk} = 120 \text{ MPa})$

Annexe VII.5.4. Comparison of the two methods

To compare the two methods in the PSIDP example, in both cases the stress range limit $\Delta \sigma_{Rsk} / \gamma_{s,fat}$ may be used as reference.

In the general method, verification may be described:

For the "equivalent method":

Ratios of calculated stress range to stress range limit may be compared. In the first case, this is given by the parameter $\sqrt[k_2]{D_{ed}}$, whereas in the second case it is given by the expression: $\gamma_{s,fat} \times \Delta \sigma_{s,equ} / \Delta \sigma_{Rsk}$

$\sqrt[k_2]{D_{ed}}$ $\gamma_{s,fat} \times \Delta t$	$\sigma_{\rm s,equ}/\Delta\sigma_{\rm Rsk}$
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Reinforcement	11.6 %	16.7 %
Prestressing steel	16.3 %	18.3 %

: Comparison of methods

In this case the "equivalent method" is thus secure. The difference between the two methods is explained by the fact that for prestressed concrete sections, bending combined with axial forces is applied. The basic hypothesis of the "equivalent method" (the linearity of the relationship between stress variations and moment variations) is not true.

Application of "simplified" method

This involves verification of $\Delta \sigma_s < 100$ MPa , under the fatigue combination $C_{fat} = C_0 + Q_{LM1,Fréqu}$.

For calculation of effects of the load model LM1, the coefficients are as follows:

- Loads (2nd class): $\alpha_{Q1} = 0.9$; $\alpha_{Q2} = \alpha_{Q3} = 0.8$; $\alpha_{q1} = 0.7$; $\alpha_{q2} = \alpha_{q3} = \alpha_{qr} = 1.0$;
- Combination: $M_{LM1,Fréqu} = 0.75 \times M_{TS} + 0.40 \times M_{UDL}$

		$N_{o}(MN)$	M _o (MN.m)	$M_{\rm LM1,Fréqu}$	M_{fat}	σ _s (MPa)	$\Delta \sigma_{s} *$	
Support area	unf	29.95	2.00	-3.37	-6.27	0.966	1 99	
support area	fav	29.95	-2.90	0.53	2.37	0.000	1.00	
Mid spop	unf	29.95	2.54	4.77	8.31	54.76	106 79	
Mid-span	fav	29.95	5.54	-0.72	2.82	0.000	100.78	

* $\Delta \sigma_s = \eta.(\sigma_{s,max} - \sigma_{s,min})$ with $\eta = 1.95$

: Calculation of stress range

Reinforcement of mid-span section does not verify the required criterion. Even so the two other methods showed a large margin compared to the limit. Thus this simplified method appears very secure.

Application to justification of a corbel in reinforced concrete in transverse bending

Data

Project

In the following example, the restraint section of cantilever is considered. The fatigue verification concerns reinforcement of this section.



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3,00 2,00 M>0 1,05 2,35

: Transverse section

Traffic hypotheses

In the hypothesis of a 2 x 2 lanes motorway with high heavy-vehicle traffic:

- category 2 traffic: $N_{obs} = 2.0.10^6$ trucks per year in the slow lane
- "long distance" type traffic with a distribution of truck types in the following proportions: $p_1 = 20$ %; p_2 = 5 %; $p_3 = 50 \%$; $p_4 = 15 \%$; $p_5 = 10 \%$

The structure design life is $N_{years} = 100$ years.

Characteristics of materials

The parameters of the S-N curve of the reinforcement are:

- $k_2 = 9$; N^{*} = 10⁶; $\Delta \sigma_{Rsk} = 202$ MPa (reinforcement $\phi 20$);
- safety factor for steel: $\gamma_{s,fat} = 1.15$;

Combination of actions – stress calculation

All actions applied to the cantilever - self weight, weight of superstructure and equipments, fatigue loads cause a moment tensioning top fibre.

The basic combination for fatigue verification M₀ and fatigue loads M_{Q,fat}, always give same-sign moments.

The tensile stress in reinforcement is equal to $\sigma_s = M/(z.A_s)$, with z constant. There is thus a linear relationship between the stress variations in the reinforcement, and moment variations due to translation of fatigue loads:

$$\Delta \sigma_{s} = \frac{\left| M_{Q,fat} \right|}{z \times A_{s}}$$

with As steel section and z elastic lever arm





It is thus not necessary in this case to know the state of reference of the restraint section. It is enough to study the variations of the moment during translation of fatigue loads.

For the numerical application, the following hypotheses are adopted:

- Modular ratio steel-concrete: $\alpha = 15$
- Height, position of reinforcement: **h** = **0.32 m** et **d** =**0.28 m**
- Elastic lever arm $z = 0.90 \times d \approx 0.250$ m.

Application of general method

Application of fatigue load model FLM4

The following graph represents the variations of the moment during crossing of trucks of model FLM4. The moments are estimated from the abacus of Pücher (slab with constant thickness restraint on the end). They are expressed in kNm/ml.



:Variation of moment due to translation of model FLM4 trucks

The relationship between $\Delta \sigma_s$ and $|\Delta M|$ is linear. The "reservoir" method may thus be applied to the previous graph, to determine the ranges of moment variations, the stress ranges then being deducted from them.

Trucks of type i	1	2	3	4	5
$ \Delta M_{i,1} $ (en KN.m/m)	23.46	32.27	30.08	25.48	24.29
$\left \Delta M_{i,2} ight $ (en KN.m/m)			27.37-8.69= 18.68	20.23-5.11= 15.12	21.96-8.97 =12.99





: Ranges of moment variations

The ranges $\Delta M_{1,2}$, $\Delta M_{2,2}$, $\Delta M_{5,3}$ *and* $\Delta M_{5,4}$ *relatively low are ignored.*

Calculation of damage

The stress ranges are given by the relationship $\Delta \sigma_{si,j} = \frac{\left| \Delta M_{i,j} \right|}{z \times A_s}$.

The expression of factor damage is thus:

The following table gives the values of $\Delta M_{moy} = \sqrt[k_2]{\sum p_i \cdot (|\Delta M_{i,j}|)^{k_2}}$:

Ι	1	2	3	4	5	$k_2 \sqrt{\sum \mathbf{p} (\mathbf{M})^{k_2}}$
pi	20 %	5 %	50 %	15 %	10 %	$\sqrt[n]{2} \mathbf{p}_i \cdot (\mathbf{w}_{i,j})$
$p_i \times (\Delta M_{i,1})^{k_2}$	4.30 . 10 ¹¹	1.90 . 10 ¹²	1.01 . 10 ¹³	6.80 . 10 ¹¹	2.94.1011	28 76 kN m/m
$p_i \times (\Delta M_{i,2})^{k_2}$	3.44.106	3.75 . 10 ⁴	1.39 . 10 ¹¹	6.19 . 10 ⁹	1.05.109	20.70 RIV.III/III

This function may be defined as an average range of fatigue moment in reinforcement.

with ΔM expressed in kNm/ml for the calculation

: Calculation of average moment variation

The reinforcement area , $A_{s,fat,lim}$, giving a sufficient minimum resistance to fatigue is obtained by equaling the expression representing damage of a value of 1:

$$D_{ed} = 1.00 \quad \text{or} \quad \sqrt[k_2]{D_{ed}} = 1.00.$$
Or:
$$A_{s, fat, lim} = \sqrt[k_2]{\frac{N_{obs} \times N_{years}}{N^*}} \times \frac{\gamma_{s, fat}}{z \times \sigma_{Rsk}} \times \Delta M_{moy}$$

The numerical application gives:

 $A_{s,fat,lim} = 11.80 \text{ cm}^2/\text{m}$

Dimensioning at ULS, not considering fatigue, leads to a section of 16.50 cm²/m.

Simplification of calculation

In the previous example, the crossing of each type i of truck gives rise to several stress ranges (or moment ranges); a major range $\Delta \sigma_{i,1}$ ($\Delta M_{i,1}$) and secondary ranges $\Delta \sigma_{i,2}$, $\Delta \sigma_{i,3}$...





 $Ort \ \alpha_{i,j} \ the \ ratio \ \frac{\Delta \sigma_{i,j}}{\Delta \sigma_{i,1}} = \frac{\Delta M_{i,j}}{\Delta M_{i,1}} \ with \ j \ge 2 \qquad and \qquad \alpha_{max} = Max \ (\alpha_{i,j})$

In a first calculation only the major ranges $\Delta M_{i,1}$. are considered. Then the first two ranges are taken into account($\Delta M_{i,1}$ et $\Delta M_{i,2}$) assuming, $\Delta M_{i,2} = \alpha_{max} \cdot \Delta M_{i,1}$

In the first case:

In the second case:

$$\Delta \mathbf{M}_{\text{moy},1} = \sqrt[k_2]{\sum \mathbf{p}_i \cdot (\Delta \mathbf{M}_{i,1})^{k_2}}$$
$$\Delta \mathbf{M}_{\text{moy},2} = \sqrt[k_2]{\sum \mathbf{p}_i \cdot ((\Delta \mathbf{M}_{i,1})^{k_2} + (\alpha_{\text{mox}} \cdot \Delta \mathbf{M}_{i,1})^{k_2})}$$

Thus $\Delta M_{moy,2} = \sqrt[k_2]{1 + (\alpha_{max})^{k_2}} \cdot \Delta M_{moy,1}$

The factor linking $\Delta M_{moy,2}$ and $\Delta M_{moy,1}$ is a function of α , $f(\alpha) = \sqrt[k_2]{1+(\alpha)^{k_2}}$ and may be represented on the following figure:



In an extreme case with $\alpha = 1.00$, the value of $f(\alpha)$ is less than 1.10. For lower values $(\alpha < 0,70)$, the values of $f(\alpha)$ are very close to 1.00 (<1.01).

Accordingly, the taking into account of the major stress (or moment) ranges for each type of truck allows, in the majority of cases, obtaining of a very good approximation of the average stress (or moment) range.

In the previous example: $\alpha_{max} = \Delta M_{3,2} / \Delta M_{3,1} = 18.68 / 30.08 = 0.62$ $f(\alpha_{max}) = 1.001$ The calculation taking account of all the ranges $\Delta M_{i,j}$, gives $\Delta M_{moy} = 28.76 \text{ kN.m/m}$ In only accounting for the major ranges, the calculation gives $\Delta M_{moy} = 28.74 \text{ kN.m/m}$

The difference is less than 0.1 % !



AJustifications at ULS relative to fatigueppendix IV -Justifications at ULS relative to fatigue



In practice, in the general case, it is thus sufficient to determine, for each type *i* of truck, the unfavorable and favorable positions in the slow lane(s) giving the extreme stresses, $\sigma_{max,i}$, $\sigma_{min,i}$, and the major stress ranges $\Delta \sigma_{i,1} = \sigma_{max,i} - \sigma_{min,i}$.

The approximate calculation from the average stress is given by $\Delta \sigma_{moy} = k_2 \sqrt{\sum_{i=1}^{5} p_i (\Delta \sigma_{i,1})^{k_2}}$.

Application of "equivalent method"

Application of fatigue load model FLM3

It is sufficient here to calculate variations of the moment due to translation of FLM3 truck, multiplying the axle loads by the factor 1.40.

The factor of 1.75 is applied only for verifications of longitudinal bending for continuous structures, for sections in intermediate supports area.





The stress range $\Delta \sigma_{s,EC}$ is simply obtained from the maximum moment range and from the relationship:

 $\Delta \sigma_{s,EC} = \frac{\Delta M_{max}}{z \times A_s} = \frac{M_{Q,fat,max}}{z \times A_s} \qquad \text{with } M_{Qfat,max} = 46.79 \text{ kN.m/ml and } M_{Qfat,min} = 0$

In application of the "equivalent method", only the maximum stress range due to the FLM3 truck is taken into account. In this example, the secondary stress range is high. It is advisable to note that the calibration of the method with the correction factor $\lambda_{s,l}$, already takes account of this phenomenon.

Correction factor λ_s

The abacus NN.2 gives the value of $\lambda_{s,1}$.





The length of the influence line, in abacus abscissa, is the width of the cantilever, or 2.35 m.

Reading of the coefficient $\lambda_{s,1}$ is done on the curve 3c) (reinforcing steel – carriageway slab).

Or: $\lambda_{s,1} = 1,10$

The value of the other factor is 1.00.

Thus, finally $\lambda_s = 1.10$

Fatigue dimensioning of reinforcment

The condition
$$\Delta \sigma_{s,equ} \leq \frac{\Delta \sigma_{Rsk}}{\gamma_{s,fat}}$$
 gives

$$A_{s,min,fat} = \gamma_{s,fat} \times \frac{\lambda_s \times \Delta M_{Q,fat,max}}{z \times \Delta \sigma_{Rsk}}$$

 $M_{LM\underline{1},Fr\acute{e}qu,d\acute{e}f}$

 $z \times A$

Or reinforcement area

 $A_{s,fat,lim} = 11.70 \text{ cm}^2/\text{m}$

Application of "simplified" method

To calculate the stress range, the direct relationship is used: $\Delta \sigma_s =$

With the values: $z \approx 0.250$ m; and $M_{LM1,Fréqu,déf} = 69.10$ kN.m/m

Condition $\Delta \sigma_s \leq 100$ MPa leads to the minimum area $A_{s,min,fat} = 27.60 \text{ cm}^2/\text{m}.$

Comparison of different results

The general and "equivalent" methods give practically the same result: $A_{s,fat,lim} = 11,80 \text{ cm}^2/\text{m}$.

In this case, the hypothesis of the "equivalent method" the linearity of the relationship between $\Delta \sigma \& \Delta M$, is validated . It is thus logical to obtain comparable results.

Application of the simplified method leads, as in the previous example, to an over-dimensioning of reinforcement.

General conclusions

The major steps of the **general method**, using the FLM4 model, are summarized here. It is necessary to determine:

- forces (N₀; M₀) under basic combination
- locationt of slow lane
- variation of forces in function of truck i position x on the structure: $M_{FLM4,i}(x)$
- variation of stress during the translation of the truck i: σ_{si} (N_0 ; $M_0 + M_{FLM4,i}(x)$)
- stress ranges $\Delta \sigma_{si,j}$

The condition $D_{ed} \le 1$ is equivalent to compare calculated stress range with limit stress range:

 $K \times \Delta \sigma_{s,moy} \leq \frac{\Delta \sigma_{Rsk}}{\gamma_{s,fat}}$





With
$$\Delta \sigma_{s,moy} = \sqrt[k_2]{\sum p_i (\Delta \sigma_{si,j})^{k_2}}$$
 average stress range and $K = \sqrt[k_2]{\frac{N_{obs} \times N_{years}}{N^*}}$

A simplification of the method consists of determining for each type i of truck, the unfavorable and favorable positions giving $\sigma_{si,max}$ and $\sigma_{si,min}$, and taking account only of the maximum stress range $\Delta \sigma_{si,max} = \sigma_{si,max} - \sigma_{si,min}$. In the majority of cases, the following calculation will give a good approximation of the average stress range:

$$\Delta \sigma_{s,moy} \approx \sqrt[k_2]{p_1 (\Delta \sigma_{s1,max})^{k_2} + p_2 (\Delta \sigma_{s2,max})^{k_2} + p_3 (\Delta \sigma_{s3,max})^{k_2} + p_4 (\Delta \sigma_{s4,max})^{k_2} + p_5 (\Delta \sigma_{s5,max})^{k_2}}$$

For the "equivalent method", using the FLM3 model, it is necessary to determine:

- forces (N₀; M₀) under basic combination
- •
- location of slow lane
- forces with FLM3 truck in unfavorable and favorable position
- corresponding stresses $\sigma_{s,max}$ and $\sigma_{s,min}$ and maximum range $\Delta \sigma_{s,EC} = \sigma_{s,max} \sigma_{s,min}$
- correction factor λ_s
- •
- The condition is: $\Delta \sigma_{s,equ} \leq \frac{\Delta \sigma_{Rsk}}{\gamma_{s,fat}}$ with $\Delta \sigma_{s,equ} = \lambda_{s,max} \times \Delta \sigma_{s,EC}$

The general method and the equivalent method lead to comparable results if the hypothesis of linearity between forces and stress is verified (simple bending). In case of bending combined whith axial forces, the "equivalent method" proved to be secure.

The "simplified" method is always very secure compared to the other two.





ANNEXE VIII. - JUSTIFICATIONS AT ULS RELATIVE TO BRITTLE FAILURE

Case of the PSIDP

In this appendix, the example of the PSIDP common to the whole of the chapters in this guide is considered.

Only the calculation carried out on the section of the first span at 2.50 m from the first pier, noted as section no. 7, is detailed here.

A table summarizing the quantities of reinforcement to arrange in the other sections is then supplied, allowing identification of the most critical sections with regard to the criterion of brittle failure.

Reminder of mechanical characteristics of section 7:

- height:	h = 0.900 m
- area:	$S = 8.01 m^2$
- inertia:	$I = 0.535 m^4$
- distance cdg/f _s :	v =0.386 m
- distance cdg/f _i :	v' =0.514 m
- cabling:	20 × 12T15S arranged 1.5 cm under medium fiber

Reminder of stresses obtained in the section, with P_{m,infini} (cf. comment on combinations Chap. 6 VI.2):

$$\begin{split} P_{m, \text{ infini}} &= 44.75 \text{ MN} \\ M_{\text{ELS freq, max}} &= 0.81 \text{ MNm} \\ M_{\text{ELS freq, min}} &= -5.63 \text{ MNm} \end{split}$$

The criterion concerns only tensioned zones under the stresses of the characteristic SLS, determined by ignoring the primary effects of the prestress (Chap 6. VI.1):

 $M_{ELS cara -P, max} = 2.92 \text{ MNm}$ and $M_{ELS cara -P, min} = -7.28 \text{ MNm}$

The criterion of brittle failure in this section will thus concern both the upper fiber and the lower fiber.

In the case of the combination obtained for $M = M_{ELS freq, max}$, $\sigma_{c,f} = 4.81$ MPa in lower fiber

(respectively, $\sigma_{c,f} = 1.53$ MPa in upper fiber for $M_{ELS \text{ freq, min}}$).

Calculation of number of strands to remove to obtain cracking at frequent SLS (method a)

This first step consists of determining α_i as:

$$\sigma_{c,f} - \alpha_{i} P_{m,\infty} \left(\frac{1}{S} + \frac{e_{0i} \times y}{I} \right) = -f_{ctm}$$





or:
$$\alpha_{i} = \frac{\sigma_{c,f} + f_{ctm}}{P_{m,\infty} \left(\frac{1}{S} + \frac{e_{0i} \times y}{I}\right)} = \frac{4,81 + 3,2}{44,75 \times \left(\frac{1}{8,01} + \frac{-0,015 \times -0,514}{0,535}\right)} = 129\%$$

(respectively, $\alpha_i = 92\%$ for $M_{ELS \text{ freq, min}}$)

Which corresponds in the first case to removing all the 20 12T15S tendons and in the second case to removing the equivalent of 18.5 12T15S tendons .

<u>Verification of the ultimate resistance of the "reduced prestress" section under the cumulative effect of the</u> <u>combinations of the frequent SLS and the reduction of the calculated prestress force (method a):</u>

The vector of stresses to apply to the section is obtained by deducting from the frequent SLS stresses the isostatic effect of the removed prestress determined at the previous step:

$$N_{tot} = (1 - \Sigma \alpha_i) P_{m,\infty} = (1 - 1.00) \times 44.75 = 0 \text{ MN}$$

$$M_{tot} = M_{ELS \ Freq} - \Sigma \alpha_i \ P_{m,\infty} \ e_{0i} = 0.81 - 1.00 \times 44.75 \times -0.015 = 1.48 \text{ MNm}$$

$$(-5,02 \ MNm \ for \ M_{ELS \ freq, \ min})$$

The last calculation step consists then of verifying, from an area calculation, that the couples of values (N_{tot} ; M_{tot}) are in the diagram of ULS resistance of the section, after removal of tendons assumed to be corroded, and if applicable to determine the additional reinforcement required.

For N = 0 MN, the maximum ULS moment acceptable is 0 MNm, which leads to a total area of reinforcement to arrange in the lower fiber of 36 cm^2 .

Respectively, for N = 3.38 MN, the minimum acceptable ULS moment is -4,1 MNm, which leads to a total reinforcement area to arrange in the upper fiber of 52 cm².

Calculation of minimum reinforcement according to method b):

The minimum reinforcement to arrange according to method (b) is determined using equation [EC2-2 Expr.(6.101a)]:

$$A_{s,min} = \frac{M_{rep}}{z_{s} f_{yk}}$$

where: $M_{rep} = \frac{-f_{ctm} \cdot I}{-v'} = \frac{-3.2 \times 0.535}{-0.514} = 3.33$ MNm for cracking in internal fiber;

(respectively $M_{rep} = -4.43$ MNm for cracking in upper fiber)

 $f_{yk} = 500 \text{ MPa}.$

The lever arm of the reinforcement at the ULS compared to the center of compression , z_s , is obtained directly thanks to the area calculation software:

 $z_s = 0.81$ m for the steels arranged on the lower fiber



(respectively $z_s = -0.80$ m for the steels arranged in the upper fiber)

Whence:

 $A_{s,min} = \frac{3,33}{0.81 \times 500} = \underline{82 \text{ cm}^2}$ in lower fiber.

Respectively $A_{s,min} = \underline{111 \text{ cm}^2}$ in upper fiber.

The same calculations, carried out on the other sections, lead to the results shown in the table below.

In this table it is stated that:

- method (b) systematically makes up a safety envelope of method (a);

- the sections that may require additional reinforcement in upper fiber relative to verification of the criterion of brittle failure in this example (PSIDP) are those situated near piers, between the 2/3 of the edge span and the $1/10^{\text{th}}$ of the center span (sections 6 to 9).

- the sections that may require additional reinforcement in lower fiber relative to verification of the criterion of brittle failure in this example (PSIDP) are those situated on edge spans, between mid-span and the pier (sections 4 to 7).

1													Λ
	culée							pile					clé
n° section	1	2	3	4	5	6	7	8	9	10	11	12	13
abscisse (m)	0,0	2,5	5	7,5	10	12,5	15	17,5	20,2	22,9	25,6	28,3	31
Ferraillage requis sel	on mét	hode A	(cm²)										
Fibre sup	XXX	XXX	XXX	XXX	XXX	42	52	85	54	XXX	XXX	XXX	XXX
Fibre inf	0	0	0	46	41	0	36	XXX	0	0	0	0	0
Ferraillage requis sel	on mét	hode B	(cm ²)										
Fibre sup	XXX	XXX	XXX	XXX	XXX	111	111	111	111	XXX	XXX	XXX	XXX
Fibre inf	82	82	82	82	82	82	82	XXX	82	82	82	82	82

Culée	Abutment
Pier	Pier
N° section	Section no.
Abscisse	Abscissa
Ferraillage requis selon method A; B	Reinforcement required according to method A; B
Fibre sup	Upper fiber
Fibre inf	Lower fiber
clé	midspan



ī

PS: The quantities of reinforcement shown in the table above include the reinforcement arranged for other reasons, particularly justification relative to longitudinal bending.

Case of box bridge constructed by balanced cantilever method

In this appendix the example of the bridge constructed by balanced cantilever method, at variable height is considered, common to all the chapters of this guide.

Only the calculation done on the corresponding section at the end of the first standard arch stone of the center span, noted as section 18, is detailed here.

A summary table of the quantities of reinforcement to be laid in the other sections is then supplied, which allows identification of the most critical sections relative to the brittle failure criterion.

Reminder of mechanical characteristics of section 18:

- height:	h = 5.21 m
- area:	$S = 8.86 m^2$
- inertia:	$I = 38.43 m^4$
- distance cdg/f _s :	v =2.22 m
- distance cdg/f _i :	v' =2.99 m
- longitudinal beam cab	ling: 24 × 12T15S placed at 16 cm from the upper fiber
- joined cabling:	$0 \times 12T15S$ placed at 16 cm from the lower fiber
- external cabling:	$8 \times 19T15S$ placed at 28 cm from the upper fiber

Reminder of stresses in section, with P_{m,infini} (cf. comment on combinations Chap. 6 VI.2):

$$\begin{split} P_{m, infini} &= 83.46 \text{ MN} \\ M_{ELS \text{ freq, max}} &= 31.3 \text{ MNm} \\ M_{ELS \text{ freq, min}} &= -17.3 \text{ MNm} \end{split}$$

The criterion concerns only the zones tensioned under characteristic SLS stresses, determined by ignoring the primary effects of the prestress (Chap. 6 VI.1):

 $M_{ELS \text{ cara -P, max}} = -118 \text{ MNm}$ and $M_{ELS \text{ cara -P, min}} = -185 \text{ MNm}$

These two values are negative and thus only the upper fiber will be considered .



or:

Justifications at ULS relative to brittle failure

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In the case of the combination obtained for $M = M_{ELS \text{ freq, min}}, \sigma_{c,f} = 8.42 \text{ MPa}$ (11,23 MPa for $M_{ELS \text{ freq, max}}$).

Calculation of number of strands to remove to obtain cracking at frequent SLS (method a):

This first stage consists of determining α_i , corresponding to the longitudinal beam tendons, such as:

$$\sigma_{c,f} - \alpha_{i} \cdot P_{m,\infty} \left(\frac{1}{S} + \frac{e_{0i} \times y}{I} \right) = -f_{ctm}$$

$$\alpha_{i} = \frac{\sigma_{c,f} + f_{ctm}}{P_{m,\infty} \left(\frac{1}{S} + \frac{e_{0i} \times y}{I} \right)} = \frac{8,42 + 4,4}{83,46 \times \left(\frac{1}{8,86} + \frac{2,06 \times 2,22}{38,43} \right)} = 66\%$$
(81 % for M_{ELS freq, max}).

which corresponds in both cases, to obtain cracking, to removal of all 24 longitudinal beam tendons: $\frac{24 \times (12 \times 150)}{24 \times (12 \times 150) + 8 \times (19 \times 150)} = 65\%.$

Verification of the ultimate resistance of the "reduced prestress" under the cumulative effect of the combinations of the frequent SLS and the reduction of the calculated prestress (method a):

The vector of stresses to apply to the section is obtained by deducting from the frequent SLS stresses the isostatic effect of the removed prestress determined at the previous step:

$$N_{tot} = (1 - \Sigma \alpha_i) P_{m,\infty} = (1 - 0.65) \times 83.46 = 29.21 \text{ MN}$$
$$M_{tot} = M_{ELS Freq} - \Sigma \alpha_i \times P_{m,\infty} \times e_{0i} = -17.3 - 0.65 \times 83.46 \times 2.06 = -129.05 \text{ MNm}$$
$$(-80.45 \text{ MNm for } M_{ELS freq, max})$$

The last calculation step consists then of verifying, from an area calculation, that the couples of values (N_{tot} ; M_{tot}) are in the diagram of ULS resistance of the section, after removal of tendons assumed to be corroded, and if applicable to determine the additional reinforcement required.

For N = 29.21 MN, calculation of the area shows that the minimum acceptable ULS moment is -82.5 MNm, which leads to a total area of reinforcement in upper fiber of <u>197 cm²</u>.

 $(0 \text{ cm}^2 \text{ for } M_{\text{ELS freq, max}})$

Calculation of minimum reinforcement according to method b):

The minimum reinforcement to lay according to method (b) is determined from equation [EC2-2 Expr.(6.101a)]:

$$A_{s,min} = \frac{M_{rep}}{z_{s} f_{yk}}$$

where: $M_{rep} = \frac{-f_{ctm} \cdot I}{v} = \frac{-4,4 \times 38,43}{2,22} = -76.2$ MNm;

 $f_{yk} = 500 \text{ MPa}.$



The lever arm of the reinforcement at ULS in relation to the center of compression, z_s , is obtained directly thanks to the area calculation software:

 $z_s = -4.98 \text{ m}$

Whence: $A_{s,min} = \frac{-76,2}{-4,98 \times 500} = 306 \text{ cm}^2.$

The same calculations, carried out on the other sections, give results shown in the table below.

From this table comes:

- method (b) systematically makes up a safety envelope of method (a);

- the sections that may require additional reinforcement in upper fiber relative to verification of the brittle failure criterion in this example (bridge constructed by variable-height corbel) are those situated near the center-span supports (segments Vd1 and Vd2) and up to a third of the span from the pier on the edge span (segments Vg1 to Vg5).

- the sections that may require additional reinforcement in lower fiber relative to verification of the brittle failure criterion in this example (variable height) are those near the edge span abutments (segments Vg13 and Vg12).



Récapulatif	Summary
Culée	Abutment
pile	Pier
Clé	midspan
N° section	Section no.
Ferraillage requis selon méthode A	Reinforcement required according to method A
Ferraillage requis selon méthode B	Reinforcement required according to method B
Fibre sup	Upper fiber
Fibre inf	Lower fiber



Appendix V - Justifications at ULS relative to brittle failure -Justifications at ULS relative to brittle failure

PS: The quantities of reinforcement shown in the table above include the reinforcement placed for other reasons, particularly justification relative to longitudinal bending.

Verification of form stability - simplified methods and example of two piersAppendix VI - Verification of form stability - Simplified

ANNEXE IX.VERIFICATION OF FORM STABILITY - SIMPLIFIED METHODS AND EXAMPLE OF TWO PIERS

The study deals with the two piers of the bridge constructed by balanced cantilevers method.

The initial geometric imperfections are deducted from a global inclination of each pier l'Eurocode 2 [EC2-1-1 5.2(7), EC2-2 5.2(106)] which allows calculation of an eccentricity at the top of the pier.

<u>Initial data</u>

General data

- Tops of piers of heights L=21.0m and L=32.0m embedded at bottom and free on top
- Global inclinations for geometric imperfections $\theta_i = \theta_0 \times \alpha_h = \frac{1}{200} \min\left(\frac{2}{\sqrt{L}}; 1\right)$ and derived

eccentricities at top of piers

• For L=21.0m $\theta_i = 0,00218$ rad or $e_i = 21,0 \times 0,00218 = 0,046$ m

 $(>e_0 = 0.02m [EC2-1-1 \ 6.1 \ (4)])$

• For L=32.0m $\theta_i = 0,00177 \text{ rad}$ or $e_i = 32,0 \times 0,00177 = 0,057 \text{ m}$

 $(>e_0 = 0.02m [EC2-1-1 \ 6.1(4)])$

- Fault of positioning of vertical loads at top of pier; layout fault and distortion of support devices, or $e_{pos}=0.05m$
- Constant and symmetrical area (reinforcement included), or a rectangular area of width b = 4.60m, height h = 2.30m and density of $0.025MN/m^3$
- Area of reinforcement corresponding to a minimum geometric ratio of reinforcement ρ =0.002, or 44HA25 distributed on two faces
- Characteristic resistance of concrete , f_{ck} =30MPa
- Characteristic yield strength of class B reinforcement, f_{yk} =500MPa
- Vertical loads applied to top of pier resulting from longitudinal bending calculation of deck, or N_{qp} =24.67MN, N_{ULS} =39.22MN
- Horizontal loads at top of pier, H_{ULS} =0.90MN
- Pier loaded at t₀=20 days (average simplified value used for both piers. In practice this value should be more representative of the actual phasing of the completion of the structure)
- Critical section easily identified as situated at embedding level.





: Elevation and transverse section of piers – Definition of parameters

Characteristics of the constant section for the structural analysis

Characteristics of concrete

Height	h = 2.30m	
Width	b = 4.60m	
Center of gravity	$Y_{G} = 1.15m$	
Area	$A_{\rm C} = 10.58 {\rm m}^2$	
Inertia	$I_{\rm C} = 4.6640 {\rm m}^4$	
Radius of gyration	$i = \sqrt{\frac{I_C}{A_C}} = \sqrt{\frac{4,664}{10,58}} = 0,664m$	
Notional size	$h_0 = \frac{2A_C}{u} = \frac{2 \times 10,58}{2 \times (4,60+2,30)} = 1,533m = 1533mm$	



Verification of form stability - simplified methods and example of two piers Appendix VI - Verification of form stability - Simplified



Characteristic strength	$f_{ck} = 30MPa$
Mean compressive strength	$f_{cm} = f_{ck} + 8 = 30 + 8 = 38MPa$

	EC2-1-1	EC2-2
Design value of modulus	$E_{cd} = \frac{E_{cm}}{\gamma_{cE}}$	$E_{cm} = 22000 \left(\frac{f_{cm}}{10}\right)^{0,3}$
	$E_{cd} = \frac{32837}{1,2} = 27364$ MPa	$E_{cm} = 22000 \left(\frac{38}{10}\right)^{0.3} = 32837 MPa$
Design value of compressive	$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c}$	$\gamma_{\rm cf} = 1.1 \frac{\gamma_{\rm S}}{\gamma_{\rm C}} = 1.1 \times \frac{1.15}{1.5} \approx 0.85$
strength at ULS	$f_{cd} = \frac{1 \times 30}{1.5} = 20 MPa$	$\gamma_{\rm cf} f_{\rm ck} = 0,85 \times 30 = 25,5 {\rm MPa}$
Parameters of Sargin type law	$k = \frac{1,05 E_{cd} \varepsilon_{c1} }{f_{cd}}$	$k = \frac{1,05 \text{ E}_{cm} \epsilon_{cl} }{\gamma_{cf} f_{ck}}$
	$k = \frac{1,05 \times 27364 \times 0,002162}{20}$	$k = \frac{1,05 \times 32837 \times 0,002162}{25,5}$
	k = 3,1060	k = 2,9233

Characteristics of reinforcement

Diameter of a steel bar	$\emptyset = 25$ mm
Number of steel bars distributed on two faces	Nb = 44
Cover	c = 0.07m
Total section of steel bars	$A_s = 0.0216m^2$
Inertia of steel bars in relation to center of gravity of concrete	$I_s = 0.0252m^4$
Radius of gyration	$i_s = \sqrt{\frac{I_s}{A_s}} = \sqrt{\frac{0,0252}{0,0216}} = 1,080m$
Effective depth	$d = \frac{h}{2} + i_s = \frac{2,30}{2} + 1,08 = 2,23m$
Geometric ratio of reinforcement	$\rho = \frac{A_s}{A_c} = \frac{0.0216}{10.58} = 0.00204 > \rho_{\text{minimal}} = 0.002$


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Mechanical ratio of reinforcement	$\omega = \frac{A_{s}f_{yd}}{A_{c}f_{cd}} = \frac{0,0216 \times 434,78}{10,58 \times 20} = 0,044$
Steel, class B	
Strain under maximum load	$\varepsilon_{uk} = 0.05$
Minimum value of (ft/fy)k	k = 1.08
Characteristic yield strength	$f_{yk} = 500 MPa$
Design value of modulus of elasticity	$E_s = 200000 MPa$

	EC2-1-1	EC2-2
End of ULS elastic branch		
Value of strain	$\epsilon_{yd} = \frac{f_{yd}}{E_s} = \frac{434,78}{200000} = 0,002174$	$\frac{1.1f_{yk}}{E_s} = \frac{550}{200000} = 0,00275$
Design yield strength	$f_{yd} = \frac{f_{yk}}{\gamma_S} = \frac{500}{1,15} = 434,78 \text{MPa}$	$1,1f_{yk} = 1,1 \times 500 = 550$ MPa
End of inclined ULS branch	$\varepsilon_{uk} = 0.05$	
Value of strain	$\varepsilon_{ud} = 0.9\varepsilon_{uk} = 0.9 \times 0.05 = 0.045$	$\varepsilon_{uk} = 0,05$
Design yield strength	$k \times f_{yd} = 1,08 \times 434,78 = 469,56 MPa$	$1,1k \times f_{yk} = 1,1 \times 1,08 \times 500 = 594$ MPa

Axial forces

Piers's self weight

<u>21m pier</u>	<u>32m pier</u>
$N_{pp} = A_c \times dens \times L$	$N_{pp} = A_c \times dens \times L$
$N_{pp} = 10,58 \times 0,025 \times 21,00$	$N_{pp} = 10,58 \times 0,025 \times 32,00$
$N_{pp} = 5,555MN$	$N_{pp} = 8,464 MN$

Axial forces acting on ULS at foot of piers

21m pier	<u>32m pier</u>
$N_{Ed} = N_{ELU} + \gamma_G N_{pp}$	$N_{Ed} = N_{ELU} + \gamma_G N_{pp}$
$N_{Ed} = 39,22 + (1,35 \times 5,555)$	$N_{Ed} = 39,22 + (1,35 \times 8,464)$
N _{Ed} = 46,719MN	N _{Ed} = 50,646MN





First-order moments at foot of piers

Quasi-permanent combination

 $\frac{21 \text{m pier}}{M_{0Eqp} = N_{qp} \times (e_i + e_{pos}) + N_{pp} \frac{e_i}{2}}$ $M_{0Eqp} = (24,67 \times (0,046 + 0,05)) + (5,555 \times \frac{0,046}{2})$ $M_{0Eqp} = 2,497 \text{MN.m}$ $\frac{32 \text{m pier}}{M_{0Eqp} = N_{qp} \times (e_i + e_{pos}) + N_{pp} \frac{e_i}{2}}{M_{0Eqp} = (24,67 \times (0,057 + 0,05)) + (8,464 \times \frac{0,057}{2})$ $M_{0Eqp} = 2,881 \text{MN.m}$

ULS combination

$$\frac{21 \text{m pier}}{M_{0\text{Ed}} = \left(N_{ELU} \times (e_i + e_{\text{pos}})\right)} + \left(\gamma_G \times N_{pp} \frac{e_i}{2}\right) + \left(H_{ELU} \times L\right)} \\ M_{0\text{Ed}} = (39,22 \times (0,046 + 0,05)) + \left(1.35 \times 5,555 \times \frac{0,046}{2}\right) + (0,90 \times 21,00) \\ M_{0\text{Ed}} = 22,838\text{MN.m}} \frac{32 \text{m pier}}{M_{0\text{Ed}} = \left(N_{ELU} \times (e_i + e_{\text{pos}})\right)} + \left(\gamma_G \times N_{pp} \frac{e_i}{2}\right) + \left(H_{ELU} \times L\right) \\ M_{0\text{Ed}} = (39,22 \times (0,057 + 0,05)) + \left(1,35 \times 8,464 \times \frac{0,057}{2}\right) + (0,90 \times 32,00) \\ M_{0\text{Ed}} = 33,322\text{MN.m}}$$

Application of simplified criteria to ignore creep and second-order effects

Creep

The final value of the creep coefficient may be calculated from the basic equation described in annex B of the Eurocode:

$$\begin{aligned} \varphi(t,t_{0}) &= \varphi_{\rm RH} \cdot \beta(t_{\rm cm}) \cdot \beta(t_{0}) \cdot \beta_{\rm c}(t,t_{0}) \\ \varphi(t,t_{0}) &= \left[1 + \frac{1 - {\rm RH}/100}{0,1 \cdot \sqrt[3]{h_{0}}} \right] \times \left[\frac{16,8}{\sqrt{t_{\rm cm}}} \right] \times \left[\frac{1}{(0,1+t_{0}^{0,20})} \right] \times \left[\frac{(t-t_{0})}{(\beta_{\rm H}+t-t_{0})} \right]^{0,3} \\ \varphi(\infty,20) &= \left[1 + \frac{1 - 70/100}{0,1 \cdot \sqrt[3]{1533}} \right] \times \left[\frac{16,8}{\sqrt{38}} \right] \times \left[\frac{1}{(0,1+20^{0,20})} \right] \times 1 = 1,225 \times 2,725 \times 0,521 \times 1 = 1,739 \end{aligned}$$

The effective creep coefficient is determined with the maximum moments at the base of the piers.

21m pier

32m pier



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$$\phi_{ef} = \phi(\infty, 20) \frac{M_{0Eqp}}{M_{0Ed}} \phi_{ef} = 1,739 \times \frac{2,497}{22,838} = 0,190$$

$$\phi_{ef} = 0,190$$

$$\phi_{ef} = 1,739 \times \frac{2,881}{33,322} = 0,150$$

Three conditions a), b) and c) to be satisfied before creep can be ignored

21m pier32m piera)
$$\phi(\infty,20) = 1,739 \le 2$$
 condition verifieda) $\phi(\infty,20) = 1,739 \le 2$ condition verifiedb) $\lambda = \frac{l_0}{i} = \frac{2 \times 21}{0,664} = 63,3 \le 75$ conditionb) $\lambda = \frac{l_0}{i} = \frac{2 \times 32}{0,664} = 96,4 \le 75$ condition notverifiedb) $\lambda = \frac{l_0}{i} = \frac{2 \times 32}{0,664} = 96,4 \le 75$ condition notverifiedc) $e_1 = \frac{M_{0Ed}}{N_{Ed}} = \frac{22,838}{46,719} = 0,489 \text{ m} \ge \text{h} = 2,30 \text{ m}$ condition not verifiedc) $e_1 = \frac{M_{0Ed}}{N_{Ed}} = \frac{33,322}{50,646} = 0,658 \text{ m} \ge \text{h} = 2,30 \text{ m}$

The creep effect may not be ignored for the two piers.

Second-order effects

$$\begin{array}{l} \underline{21m \ pier} \\ \lambda_{\lim} = \frac{20 \times A \times B \times C}{\sqrt{n}} \\ n = \frac{N_{Ed}}{A_c f_{cd}} = \frac{46,719}{10,58 \times 20} = 0,221 \\ A = \frac{1}{1+0,2\varphi_{ef}} = \frac{1}{1+0,2 \times 0,190} = 0,963 \\ B = \sqrt{1+2\omega} = \sqrt{1+2 \times 0,044} = 1,043 \\ C = 1,7 - r_m = 1,7 - 1 = 0,7 \\ \lambda_{\lim} = \frac{20 \times 0,963 \times 1,043 \times 0,7}{\sqrt{0,221}} = 29,9 \\ \lambda = 63,3 > \lambda_{\lim} \end{array} \qquad \begin{array}{l} \underline{32m \ pier} \\ \lambda_{\lim} = \frac{20 \times A \times B \times C}{\sqrt{n}} \\ n = \frac{N_{Ed}}{A_c f_{cd}} = \frac{50,646}{10,58 \times 20} = 0,239 \\ A = \frac{1}{1+0,2\varphi_{ef}} = \frac{1}{1+0,2 \times 0,150} = 0,971 \\ B = \sqrt{1+2\omega} = \sqrt{1+2 \times 0,044} = 1,043 \\ C = 1,7 - r_m = 1,7 - 1 = 0,7 \\ \lambda_{\lim} = \frac{20 \times 0,963 \times 1,043 \times 0,7}{\sqrt{0,229}} = 29,9 \\ \lambda = 96,4 > \lambda_{\lim} \end{array}$$

The second-order effects should be taken into account for the two piers





Simplified method based upon an estimation of the curvature 1/r [EC2-1-1 5.8.8]

Description of method

The simplified method based upon an estimation of the curvature is suitable for isolated elements of constant and symmetrical cross-section (reinforcement included) subjected to a axial constant force.

It is to determine the total moment obtained from the sum of the first and second order moments [EC2-1-15.8.8.2(1) and (2)].

$$\underbrace{\mathbf{M}}_{\text{Ed}} = \underbrace{\mathbf{M}}_{\substack{\text{0Ed}}} + \underbrace{\mathbf{M}}_{2}$$
moment
total
$$\lim_{1^{\text{er}} \text{ ordre}} + \underbrace{\mathbf{M}}_{2^{\text{nd}} \text{ ordre}}$$

The method allows calculation of the second-order moment [EC2-1-1 5.8.8.2(3)] from an estimation of the curvature of the structure in equilibrium [EC2-1-1 5.8.8.3(1) to (4)], from the choice of a coefficient c depending on the distribution of the curvatures of the first and second order moments [EC2-1-1 5.8.8.2(4)] and an eccentricity e_2 created by the second order effects. The product of this eccentricity e_2 and of the design acting axial force gives the second order moment.

$$\underbrace{M_2}_{\substack{\text{moment}\\2^{nd} \text{ ordre}}} = \underbrace{N_{\text{Ed}}}_{\substack{\text{effort}\\normal}} \left(\frac{1}{r} + \frac{l_0^2}{c} \right)$$

où:
$$\frac{1}{r} \quad \text{courbure estimée}$$

$$c = \pi^2 \text{ si courbure sinusoïdale, } c = 8 \text{ si courbure constante}$$

Illustration of method – Example of the 21m-high pier



Fig: Calculation parameters of M2 on a "mast" type pier

1.1.a)Calculation of curvature



$$\frac{1}{r} = K_r K_{\phi} \frac{1}{r_0}$$

$$K_r = \min\left(\frac{n_u - n}{n_u - n_{bal}}; 1\right) = \min\left(\frac{(1 + \omega) - n}{(1 + \omega) - n_{bal}}; 1\right) = \min\left(\frac{(1 + 0,044) - 0,221}{(1 + 0,044) - 0,4}; 1\right) = 1$$

$$K_{\phi} = \max\left(1 + \beta\phi_{ef}; 1\right) = \max\left(1 + \left(0,35 + \frac{f_{ck}}{200} - \frac{\lambda}{150}\right)\phi_{ef}; 1\right)$$

$$K_{\phi} = \max\left(1 + \left(0,35 + \frac{30}{200} - \frac{63,3}{150}\right) \times 0,190; 1\right) = 1,015$$

$$\frac{1}{r_0} = \frac{\varepsilon_{yd}}{0,45d} = \frac{0,002174}{0,45 \times 2,230} = 0,0021664$$

$$\frac{1}{r} = 1 \times 1,015 \times 0,0021664 = 0,0021989$$

$$K_r$$

: Parameter K_r

Choice of coefficient c

In the example concerned, the first-order moment is linear and the second-order moment sinusoidal.

0,4

c depends on the total curvature and the inequalities $8 < c < \pi^2$ are verified. As the second-order moment is numerically higher than the first-order moment (due to the imperfections and the horizontal force at the head of the pier) the total curvature is essentially parabolic (since M₂>>M_{0Ed}), the coefficient is thus taken as $c = \pi^2$

 $1+\omega$

n

Calculation of eccentricity e_2

$$e_{2} = \frac{1}{r} \frac{l_{0}^{2}}{c}$$

$$e_{2} = 0,0021989 \times \frac{(2 \times 21)^{2}}{\pi^{2}} = 0,393m$$

Calculation of second-order moment

The self weight of the pier causes a variation of the axial force along the length of the pier of (46.719/39.22)-1=19.1% and in any event the method is not applicable. However, the weight of the pier is low relative to the



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load on top and has little influence on the final result. The second-order moment will be over-valued by applying, to the top of the pier, the design axial force N_{Ed} calculated at embedding.

 $M_2 = N_{Ed} \times e_2$ $M_2 = 46,719 \times 0,393 = 18,361MN.m$

Calculation of total moment

 $M_{Ed} = M_{0Ed} + M_2$ $M_{Ed} = 22,838 + 18,361 = 41,199MN.m$

Verification of constant section

 $M_{Ed} = 41,199MN.m \le M_{resistant} = 49,474MN.m$

The critical section is validated.

Simplified method based upon an estimation of nominal rigidity EI [EC2-1-1 5.8.7]

Description of method

The simplified method based upon a nominal rigidity may be used for the isolated elements of a given cross-section.

The method allows calculation of an amplification factor relative to the second-order effects from a coefficient β and from the nominal buckling load N_B based upon the nominal rigidity [EC2-1-1 5.8.7.3(1)].

$$\underbrace{M_{Ed}}_{\substack{\text{moment}\\\text{total}}} = \underbrace{M_{0Ed}}_{\substack{\text{moment}\\1^{\text{er}} \text{ ordre}}} \left[1 + \frac{\beta}{\frac{N_B}{N_{Ed}}} - 1 \right]$$

 $\beta = \frac{\pi^2}{c_0}$ depends upon the distribution of first and second order moments [EC2-1-1 5.8.7.3(2) to (4)]

In the case of isolated elements of the constant section subjected to a continuous axial force, a sinusoidal distribution of the second-order moment is assumed and c0 takes a value that depends on the distribution of the first-order moment M_{0Ed} , or for example:

For a constant distribution of M_{0Ed}, $c_0 = 8$ and $\beta = \frac{\pi^2}{c_0} = \frac{\pi^2}{8} \approx 1,234$

Pour a parabolic distribution of M_{0Ed} , $c_0 = 9.6$ and $\beta = \frac{\pi^2}{c_0} = \frac{\pi^2}{9.6} \approx 1,028$

For a triangular distribution of M_{0Ed} , $c_0 = 12$ and $\beta = \frac{\pi^2}{c_0} = \frac{\pi^2}{12} \approx 0.822$





In the case of isolated elements where either the axial force and/or the section vary, or that a transverse load is applied, $\beta = 1$ normally constitutes a reasonable simplification.

 $N_B = \frac{\pi^2 EI}{l_0^2}$ is calculated from an estimation of nominal rigidity EI obtained from the addition of two terms, one relative to concrete , the other to reinforcement, subject to the geometric ratio of reinforcement $\rho = \frac{A_s}{A_c} \ge 0,002$, [EC2-1-1 5.8.7.2 (1) to (4)].

$$EI = \underbrace{K_c E_{cd} I_c}_{b\acute{e}ton} + \underbrace{K_s E_s I_s}_{armatures}$$

The whole of these parameters is clarified in the digital example.

Illustration of method – Example of the 21m-high pier

Calculation of nominal rigidity EI

$$EI = K_{c}E_{cd}I_{c} + K_{s}E_{s}I_{s}$$

$$K_{c} = \frac{K_{1}K_{2}}{1 + \varphi_{ef}}$$

$$K_{1} = \sqrt{\frac{f_{ck}}{20}} = \sqrt{\frac{30}{20}} = 1,225$$

$$K_{2} = \min\left(n\frac{\lambda}{170}; 0,20\right) = \min\left(0,221 \times \frac{63,3}{170}; 0,20\right) = 0,082$$

$$K_{c} = \frac{1,225 \times 0,082}{1 + 0,190} = 0,084$$

$$K_{s} = 1$$

 $EI = (0,084 \times 27364 \times 4,6640) + (1 \times 200000 \times 0,0252) = 15760,56MN.m^{2}$

Calculation of nominal buckling load

$$N_{\rm B} = \frac{\pi^2 \text{EI}}{l_0^2}$$
$$N_{\rm B} = \frac{\pi^2 \times 15760,56}{(2 \times 21)^2} = 88,181\text{MN}$$

Calculation of total moment



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$$M_{Ed} = M_{0Ed} \left[1 + \frac{\beta}{\frac{N_B}{N_{Ed}} - 1} \right]$$

Г

The self weight of the pier causes a variation of axial force along the length of the pier of (46,719/39,22)-1 =19,1%; the element is subjected to a transverse load, $\beta = 1$.

$$M_{Ed} = 22,838 \times \left[1 + \frac{1}{\frac{88,181}{46,719} - 1} \right] = 48,572 MN.m$$

Verification of the constant section

Annexe X.

$$M_{Ed} = 48,572 MN.m \le M_{resistant} = 49,474 MN.m$$

The critical section is validated.

<u>General method of EN1992-1-1 [EC2-1-1 5.8.6]</u>

Illustration of method without software - Example of the 21m-high pier

It will be recalled that when the designer has no access to software allowing interfacing of geometric and material non-linearity, it is possible to find, for a section judged a priori non-critical, the state of equilibrium by expressing in two ways the relationship linking the bending moment in the critical section with its curvature:

- The external moment-curvature law where the bending moment acting upon the section is the sum of the first-order moment M_{0Ed} and the second-order moment. To simplify, distribution of curvatures along the structure is considered linear, which allows determination of the second-order effect according solely to the curvature of the critical section;
- The internal moment-curvature law, where the resisting bending moment results from the state of stresses of the section subject to an imposed curvature with a given axial force:
- The intersection, or not, of the two curves representative of the external and internal laws allow verification of the existence, or not, of a state of equilibrium. If so, the intersection of the two curves gives the value of the total moment M_{Ed} at equilibrium.











: Verification of state of equilibrium and determination of total moment M_{Ed}

Loi interne	Internal law
Loi externe	External law
Moment 1er ordre	First-order moment
Moment total à léquilibre (1er et 2e ordre) Total moment at equilibrium (first an order)	

5.1.a) External law M(1/R)_{external}

In the hypothesis of a buckling level, a sinusoidal deformation and a small deflection e_2 it may be assumed:

$$\begin{aligned} \mathbf{f}(\mathbf{x}) &= \mathbf{e}_2 \sin \frac{\pi \mathbf{x}}{\mathbf{l}_0} \\ \frac{1}{\mathbf{R}} &= \frac{\mathbf{d}^2 \mathbf{f}}{\mathbf{dx}^2} \end{aligned} \qquad \left\{ \frac{1}{\mathbf{R}} = \mathbf{e}_2 \left(\frac{\pi}{\mathbf{l}_0} \right)^2 \sin \left(\frac{\pi \mathbf{x}}{\mathbf{l}_0} \right) \right. \end{aligned}$$

The value e_2 is deducted from the curvature l/R for $l_0/2$ and the result is then used in the simplified method based on the estimation of the curvature.



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$$e_2 = \frac{{l_0}^2}{\pi^2} \frac{1}{R}$$

: Calculated deformation and parameters of a bi-articulated element

Déformée sinusoidale	Sinusoidal deformation	
		_

The total moment is the sum of the first and second order moments determined from the eccentricity value e2 . At the level of the section $x = \frac{l_0}{2}$, the total moment is equal to:

$$M\left(\frac{1}{R}\right) = \underbrace{M_{0Ed}}^{\text{moment } 1^{\text{er} \text{ ordre}}} + \underbrace{N_{\text{comb}} e_2 + \sum_{i=1}^{i=n} \gamma_G P_i \left(e_2 - e_2 \sin \frac{\pi \times x_i}{l_0}\right)}_{M\left(\frac{1}{R}\right) = M_{0Ed} + \left[N_{\text{comb}} + \gamma_G \sum_{i=1}^{i=n} P_i \left(1 - \sin \frac{\pi \cdot x_i}{l_0}\right)\right] \frac{l_0^2}{\pi^2} \frac{1}{R}$$

With

 $\gamma_G P_i$ self weight at abscissa x_i

N_{comb} axial force of combination considered, applied to end of element

In the case of a constant load P_i the equation becomes:



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$$\begin{split} M\left(\frac{1}{R}\right) &= M_{0Ed} + \left[N_{comb} + \gamma_{G}P_{i}\int_{0}^{\frac{l_{0}}{2}} \left(1 - \sin\frac{\pi \times x}{l_{0}}\right)dx\right]\frac{l_{0}^{2}}{\pi^{2}}\frac{1}{R} \\ M\left(\frac{1}{R}\right) &= M_{0Ed} + \left[N_{comb} + \gamma_{G}\left(P_{i}\frac{l_{0}}{2}\right)\left(1 - \frac{2}{\pi}\right)\right]\frac{l_{0}^{2}}{\pi^{2}}\frac{1}{R} \\ M\left(\frac{1}{R}\right) &= M_{0Ed} + \left[N_{comb} + \gamma_{G}N_{pp}\frac{\pi - 2}{\pi}\right]\frac{l_{0}^{2}}{\pi^{2}}\frac{1}{R} \\ M\left(\frac{1}{R}\right) &= M_{0Ed} + \left[N_{comb} + \frac{\gamma_{G}N_{pp}}{2,752}\right]\frac{l_{0}^{2}}{\pi^{2}}\frac{1}{R} \\ M\left(\frac{1}{R}\right) &= 22,838 + \left[39,220 + \frac{1,35 \times 5,555}{2,752}\right] \times \frac{(2 \times 21)^{2}}{\pi^{2}} \times \frac{1}{R} \\ M\left(\frac{1}{R}\right) &= 22,838 + 7496,813 \times \frac{1}{R} \end{split}$$

The value of the external moment for the curvature corresponding to the structure's equilibrium $\frac{1}{R} = 0,00030143$ (see below) is equal to:

 $M(0,00030143) = 22,838 + 7496,813 \times 0,00030143 = 25,098MN.m$

Internal law M(1/R)_{internal}

From the materials' stress-strain diagrams , the value and the position of the internal forces of the reinforced concrete section are determined for a given curvature 1/R as is the axial acting compressive force N_{Ed} (steps 1 to 3), to obtain the internal resisting moment (step 4). This calculation is carried out for several values of curvature to be able to trace the representative curve of the internal law M(1/R). The tensile strength of the concrete is ignored and creep is taken into account by multiplying all the strain values by the factor $(1+\phi_{ef})$.

In the calculations, the values ε , σ , F are positive in compression, negative in tension. The sign sc corresponds to steels positioned above the center of gravity of the concrete section (compressed or least-tensioned steels), st to steels positioned under the center of gravity of the concrete section (tensioned steels or least-compressed).

Calculation of the internal moment is detailed below for the curvature $\frac{1}{R} = 0,00030143$ corresponding to the structure's equilibrium.

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: Definition
of
parameters
for for

calculation of internal resisting moment

SECTION ENTIEREMENT COMPRIMEE

Section entièrement comprimée	Totally compressed section	
Section partiellement comprimée	Partially compressed section	

• Step 1 - Calculation of strains

From the basic relationships $\frac{1}{R} = \frac{\epsilon_b - \epsilon_{st}}{d}$, (d distance between ϵ_b and ϵ_{st}), the strains on the height h of the reinforced concrete section subjected to a curvature 1/R are determined.

To begin, take a curvature 1/R and a strain ε_b on the furthest fiber of the concrete, such that $\varepsilon_b \leq \varepsilon_{c1}(1+\phi_{ef})$ with ε_{c1} the value of the strain at the peak stress, to obtain the height of compressed concrete, or:

 $Y_{hc} = \frac{\varepsilon_b}{1/R} \begin{cases} \geq h \text{ si la section est entièrement comprimée} \\ < h \text{ si la section est partiellement comprimée} \end{cases}$



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For
$$\frac{1}{R} = 0,00030143$$
 and $\varepsilon_b = 0,00054841 \le \varepsilon_{c1}(1 + \varphi_{ef}) = 0,002162 \times (1 + 0,190) = 0,002573$, the

compressed height is equal to $Y_{hc} = \frac{\varepsilon_b}{1/R} = \frac{0,00054841}{0,00030143} = 1,819 \text{ m} \le h = 2,30 \text{ m}$ (partially-compressed section).

The strains become:

• The length of the concrete section

$$\begin{split} \boldsymbol{\epsilon}_{ci} &= \frac{\boldsymbol{\epsilon}_{bi}}{\left(1 + \boldsymbol{\phi}_{ef}\right)} \\ \boldsymbol{\epsilon}_{ci} &= \frac{\left(\left|\boldsymbol{Y}_{hc}\right| - \left|\boldsymbol{Y}_{i}\right|\right) \cdot \left(1/R\right)}{\left(1 + \boldsymbol{\phi}_{ef}\right)} \\ \boldsymbol{\epsilon}_{ci} &= \frac{\left(1,819 + \boldsymbol{Y}_{i}\right) \times 0,00030143}{\left(1 + 0,190\right)} \quad \text{avec} - 2,30 \leq \boldsymbol{Y}_{i} \leq 0 \end{split}$$

• On the steels above the section's center of gravity

$$\begin{split} \epsilon_{sc} &= \epsilon_{b} - \left[\left(\left| \mathbf{Y}_{G} \right| - \left| \mathbf{Y}_{sc} \right| \right) \cdot (1/R) \right] \\ & \text{avec} \begin{cases} \left| \epsilon_{sc} \right| \text{ sans limitation si branche supérieure horizontale} \\ \left| \epsilon_{sc} \right| \leq \left| \epsilon_{ud} \right| \text{ si branche supérieure inclinée} \end{cases} \\ \epsilon_{sc} &= 0,00054841 - (1,150 - 1,080) \times 0,00030143 \end{split}$$

 $\varepsilon_{sc} = 0,00052731 \le \varepsilon_{ud} = 0,045$ condition vérifiée

• On the steels below the section's center of gravity

$$\begin{split} \epsilon_{st} &= \epsilon_{b} - \left[\left(|\mathbf{Y}_{G}| + |\mathbf{Y}_{st}| \right) \cdot (1/\mathbf{R}) \right] \\ & \text{avec} \begin{cases} |\epsilon_{st}| \text{ sans limitation si branche supérieure horizontale} \\ |\epsilon_{st}| \leq |\epsilon_{ud}| \text{ si branche supérieure inclinée} \end{cases} \\ \epsilon_{st} &= 0,00054841 - (1,150 + 1,08) \times 0,00030143 \end{split}$$

- $\varepsilon_{st} = -0,00012379 \ge \varepsilon_{ud} = -0,045$ condition vérifiée
- Step 2 Calculation of stresses
- The length of the concrete section





$$\begin{split} \sigma_{ci} &= MAX \left(f_{cd} \left[\frac{k \left(\frac{\varepsilon_{ci}}{\varepsilon_{c1}} \right) - \left(\frac{\varepsilon_{ci}}{\varepsilon_{c1}} \right)^2}{1 + (k - 2) \left(\frac{\varepsilon_{ci}}{\varepsilon_{c1}} \right)^2} \right] ; 0 \right) \\ \sigma_{ci} &= MAX \left(20 \times \left[\frac{3,1060 \times \left(\frac{\varepsilon_{ci}}{0,002162} \right) - \left(\frac{\varepsilon_{ci}}{0,002162} \right)^2}{1 + 1,1060 \times \left(\frac{\varepsilon_{ci}}{0,002162} \right)} \right] ; 0 \right) \end{split}$$

• On the steels situated above and below the section's center of gravity

$$\begin{aligned} |\boldsymbol{\varepsilon}_{s}| \leq |\boldsymbol{\varepsilon}_{yd}| &\Rightarrow \boldsymbol{\sigma}_{s} = \boldsymbol{\varepsilon}_{s} \frac{\boldsymbol{f}_{yd}}{\boldsymbol{\varepsilon}_{yd}} \\ |\boldsymbol{\varepsilon}_{s}| > |\boldsymbol{\varepsilon}_{yd}| &\Rightarrow \begin{cases} \text{branche horizontale} \\ \boldsymbol{\sigma}_{s} = \boldsymbol{f}_{yd} \\ \text{branche inclinée} \\ \boldsymbol{\sigma}_{s} = \frac{\boldsymbol{\varepsilon}_{s}}{|\boldsymbol{\varepsilon}_{s}|} \cdot \left[\left| \boldsymbol{f}_{yd} \right| + \left| \frac{\boldsymbol{\varepsilon}_{s} - \boldsymbol{\varepsilon}_{yd}}{\boldsymbol{\varepsilon}_{ud} - \boldsymbol{\varepsilon}_{yd}} \cdot \left(\mathbf{k} \cdot \boldsymbol{f}_{yd} - \boldsymbol{f}_{yd} \right) \right] \end{aligned}$$

with $\varepsilon_s = \varepsilon_{sc}$ ou ε_{st} et $\sigma_s = \sigma_{sc}$ ou σ_{st}

Either for the steels above the section's center of gravity

$$\varepsilon_{sc} = 0,00052731 \le \varepsilon_{yd} = 0,002174$$

 $\Rightarrow \sigma_{sc} = \varepsilon_{sc} \frac{f_{yd}}{\varepsilon_{yd}} = 0,00052731 \times \frac{434,78}{0,002174} = 105,457 MPa$

Or for the steels below the section's center of gravity

$$\varepsilon_{st} = -0,00012379 \ge \varepsilon_{yd} = -0,002174$$

 $\Rightarrow \sigma_{st} = \varepsilon_s \frac{f_{yd}}{\varepsilon_{yd}} = -0,00012379 \times \frac{434,78}{0,002174} = -24,757 MPa$

- $\circ \quad Step \ 3-Calculation \ of \ internal \ forces$
- Force resulting from concrete and position in relation to center of gravity

$$F_{ci} = \left(\frac{\sigma_i + \sigma_{i+1}}{2}\right) \times \left(\frac{b_i + b_{i+1}}{2}\right) \times \left(\left|h_{i+1}\right| - \left|h_i\right|\right) \cdot$$



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$$F_{c} = \sum_{i=1}^{i=n} F_{ci}$$

$$Y_{ci} = \left(\frac{h_i + h_{i+1}}{2}\right)$$

$$\boldsymbol{Y}_{c} = \boldsymbol{Y}_{G} - \frac{\displaystyle\sum_{i=1}^{i=n} \boldsymbol{Y}_{ci} \times \boldsymbol{F}_{ci}}{\displaystyle\sum_{i=1}^{i=n} \boldsymbol{F}_{ci}}$$

These equations may be solved digitally by cutting the section's height h into n pieces.

h _i (m)	b _i (m)	$\varepsilon_{bi} = \varepsilon_{ci}^* (1 + f_{ef})$	€ ci	s _{ci} (MPa)	F _{ci} (MN)	Y _{ci}	F _{ci} *Y _{ci}
0	4.6	0.00054841	0.00046085	9.98	2.609	-0.029	-0.075
-0.058	4.6	0.00053107	0.00044628	9.746	2.546	-0.086	-0.22
-0.115	4.6	0.00051374	0.00043172	9.507	2.483	-0.144	-0.357
		I					I
		I					
-2.128	4.6	-0.00009289	-0.00007806	0	0	-2.156	0
-2.185	4.6	-0.00011022	-0.00009263	0	0	-2.214	0
-2.243	4.6	-0.00012756	-0.00010719	0	0	-2.271	0
-2.3	4.6	-0.00014489	-0.00012176	0			
				total F _{ci} =	45.848	total F _{ci} *Y _{ci} =	-29.146
				Resulting r	position = -29	146/45 848 =	-0.636



: Stress in concrete

Abscisses	Abscissae
Position force Fc	Position of force Fc
Contraintes dans le béton	Stresses in concrete

 $F_{c} = 45,848MN$

- $Y_c = Y_G 0,636 = 1,15 0,636 = 0,514m$
- On the steels above the section's center of gravity

$$F_{sc} = \sigma_{sc} \times A_{sc}$$





$$F_{sc} = \sigma_{sc} \times A_{sc} = 105,457 \times \frac{0,0216}{2} = 1,139 MN$$

• On the steels below the section's center of gravity

$$F_{st} = \sigma_{st} \times A_{st}$$

$$F_{st} = \sigma_{st} \times A_{st} = -24,757 \times \frac{0,0216}{2} = -0,268MN$$

• Verification of resulting axial internal force

$$N_{int} = F_c + F_{sc} + F_{st}$$

 $N_{int} = 45,848 + 1,139 - 0,268 = 46,719 = N_{Ed}$ condition vérifiée

Nota: If this condition is not verified, the strain ϵ_b chosen at the start (or the curvature l/R) is readjusted so that the axial internal force N_{int} equal the acting force N_{Ed} .

• Step 4 – Calculation of internal resisting moment

After balancing N_{int} = N_{Ed} , the internal moment for the given curvature l/R is deducted:

 $M(1/R)_{interne} = \begin{cases} |F_c \times Y_c| + |F_{sc} \times Y_{sc}| - |F_{st} \times Y_{st}| & \text{section entièrement comprimée} \\ |F_c \times Y_c| + |F_{sc} \times Y_{sc}| + |F_{st} \times Y_{st}| & \text{section partiellement comprimée} \\ M(0,00030143)_{interne} = (45,848 \times 0,514) + (1,139 \times 1,08) + (0,268 \times 1,08) \\ M(0,00030143)_{interne} = 25,09MN.m \end{cases}$

Verification of state of equilibrium and determination of total moment $M_{\mbox{\scriptsize Ed}}$

1/R	M(1/R) _{external}	M(1/R) _{internal}	Sc extreme	Ssc extreme	Sst extreme	
0.00000000	22.838	0.000	4.334	39.985	39.985	
0.00016200	24.052	15.438	7.708	76.198	6.217	
0.00023400	24.592	21.388	8.994	92.214	-8.870	
0.00030143	25.098	25.098	9.980	105.457	-24.757	Equilibriun
0.00037800	25.672	28.157	10.925	119.070	-44.218	1
0.00043200	26.077	29.888	11.512	128.043	-58.573	



VERIFICATION DE L'EQUILIBRE & DETERMINATION DU MOMENT M $_{\rm Ed}$ pour N $_{\rm Ed}$ = 46.719MN et $\phi_{\rm e}$ \leftarrow 0.190



— M (1/R) interne

VERIFICATION DE L'EQUILIBRE ET DETERMINATION DU MOMENT	VERIFICATION OF EQUILIBRIUM AND DETERMINATION OF MOMENT			
Moment total	Total moment			
Courbure l/R	Curvature l/R			

The structural analysis was carried out with a simultaneous verification of the critical section in the calculation process. The critical section is validated.

It will be noted that the difference with the PCP software calculation below is $\left(\frac{25,098}{24,441} - 1\right) = 2,7\%$

Results for the two piers with software calculation

The examples are taken again with Sétra's PCP software which allows an overall scientific treatment with creep and a simultaneous taking into account of the second geometric order and of the material non-linearity.

Case of 21m pier





 $M_{Ed} = 24,441MN.m$ e₂ = 0,03846m $\frac{M_{Ed}}{M_{0Ed}} = \frac{24,441}{22,838} = 1,070$

Case of 32m pier

 $M_{Ed} = 43,220MN.m$ $e_2 = 0,227m$ $\frac{M_{Ed}}{M_{0Ed}} = \frac{43,220}{33,322} = 1,297$



: Diagrams of moments on pier L = 32m

Verification of constant section

For the two piers, the structural analysis was carried out with a simultaneous verification of the calculation process. The constant section is validated for the two piers.

Comments on simplified methods

The values of curvature 1/R and of rigidity EI determined from the PCP results are compared with the estimations of the two simplified methods for the 21m-high pier.



$$\frac{1}{R}_{PCP} = e_2 \times \frac{\pi^2}{l_0^2} = 0,03846 \times \frac{\pi^2}{(2 \times 21)^2} = 0,00021518 < \frac{1}{r}_{estimée} = 0,0021989 \approx \frac{1}{R}_{PCP} \times 10,2$$

EI_{PCP} = $\frac{M_{Ed}}{1/R} = \frac{24,441}{0,00021518} = 113583,97$ MN.m² > EI_{estimée} = 15760,56 \approx EI_{PCP} \div 7,2

as well as the ratios between moments including the effects of second and first-order moments.

Reminder of PCP result: $\frac{M_{Ed}}{M_{0Ed}} = \frac{24,441}{22,838} = 1,07$

Result of method based upon $\frac{1}{r}$: $\frac{M_{Ed}}{M_{0Ed}} = \frac{41,199}{22,838} = 1,80$

Result of method based upon EI: $\frac{M_{Ed}}{M_{0Ed}} = \frac{48,572}{22,838} = 2,13$

The PCP software calculation demonstrates the very safe nature of the two simplified methods.

For the 32m-high pier, the forces and moments obtained with the two simplified methods greatly exceed the resisting moment of the critical section ($M_{Ed} = 78.853$ MN.m with the method based upon $\frac{1}{r}$, $M_{Ed} = 310.482$ MN.m with the method based upon EI).

General method of EN1992-2

The method is illustrated for the two piers with a verification successively using the two inequalities (5.102 b) and (5.102 a) to highlight the small difference between the results obtained with the two criteria.

The actions effects are determined with Sétra's PCP software and the interaction diagram of the constant section calculated with Sétra's CDS software.

Verification with the criterion (5.102 b)

The process of load incrementation allowed the ultimate resistance of the section of the 21m-high pier to be reached, and the overall breakage of the structure for the 32m-high pier.

The results $\binom{N}{M}$ are shown for the necessary points of passage (en MN, MN.m).

Case of 21m pier

Project load
$$q_{ELU} = [\gamma_G G + \gamma_Q Q] = [1,35 G + 1,35 Q] \Rightarrow \text{point } U\begin{pmatrix} 46,719\\24,031 \end{pmatrix}$$

Failure load $q_{ud} = \lambda q_{ELU} = 2,95 q_{ELU} \Rightarrow \text{point A} \begin{pmatrix} 138,239\\ 91,948 \end{pmatrix}$

Load $\frac{q_{ud}}{\gamma_{O'}} = \frac{q_{ud}}{1,27} \Rightarrow \text{point } D\begin{pmatrix}108,850\\61,584\end{pmatrix}$

Point U is situated before point D in the loading direction, meaning that the pier is correctly dimensioned .





The safety level obtained is equal to $\lambda = \frac{q_{ud}}{q_{ELU}} = 2,95 > \gamma_{O'} = 1,27$.

Case of 32m pier

Project load
$$q_{ELU} = [\gamma_G G + \gamma_Q Q] = [1,35 G + 1,35 Q] \Rightarrow \text{point } U \begin{pmatrix} 50,646\\ 39,129 \end{pmatrix}$$

Failure load $q_{ud} = \lambda q_{ELU} = 1,30 q_{ELU} \Rightarrow \text{point } A \begin{pmatrix} 65,839\\ 61,156 \end{pmatrix}$

Load
$$\frac{q_{ud}}{\gamma_{O'}} = \frac{q_{ud}}{1,27} \Rightarrow \text{point } D\begin{pmatrix}51,841\\40,176\end{pmatrix}$$

Point U is situated before point D in the loading direction , meaning that the pier is correctly dimensioned .

The safety level obtained is equal to
$$\lambda = \frac{q_{ud}}{q_{ELU}} = 1,30 > \gamma_{O'} = 1,27$$
.





Fig./Tab.(9): Failure method of piers and application of safety format with the couple (axial force, moment)

Ruine par instabilité d'ensemble	Failure by overall instability
Ruine par rupture de section	Failure by section breakage
Chemin de chargement	Loading direction
Points de passage obliges	Necessary points of passage
Sous charge de ruine	Under failure load
Sous charge de projet	Under project load
Diagramme d'interaction de la section	Section interaction diagram





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: Application of safety format with the couple (load, Moment)

Implementation of method with use of criterion (5.102 a)

This practical implementation is explained via the two figures PP.3 and PP.4 in annex PP of Eurocode 2 part 2.

It may easily be observed that point A is in both cases represented as being on the curve (a), which suggests there is a critical section here. Then the figures show a verification by the criterion (5.102a): the partial factor γ_{Rd} is clearly used and γ_{Sd} is not because it serves only for criterion (5.102c).



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: Application of safety format for a vectorial combination (M,N) and an over-proportional behavior [EC2-2 Fig.PP.4]

Each cross-section is the subject of a graph (N, M) where are found the curve (a) of a section interaction diagram. The different A, B, C, D, U defined by Eurocode 2 are determined during the various loading sequences.

From the failure load, the next step is determination of the forces and moments (N, M) at points B and C and of a curve (b) obtained by a change of scale of the curve (a) to establish a safety format.

Determination of (N_B, M_B) represented by a point B

The load
$$\frac{q_{ud}}{\gamma_O} = \frac{\lambda q_{ELU}}{\gamma_O}$$
 with $\gamma_O = 1,20$ gives force and moment noted B (N_B,M_B).

Determination of (N_c, M_c) represented by a point C

The force and moment N_B and M_B are divided by γ_{Rd} , or N_c = $\frac{N_B}{\gamma_{Rd}}$ et M_c = $\frac{M_B}{\gamma_{Rd}}$ with γ_{Rd} =1.06

A point C is obtained C (N_C, M_C) .

Curve(b) of interaction (N,M) and determination of (N_D, M_D) represented by a point D

Normally a reduced safety domain is developed defined by a curve (b) obtained by a change of scale of the curve (a) in relation to the origin O (0, 0) and passing by point C (N_c , M_c). It cuts the loading direction in a point D (N_D , M_D).

Control relative to safety

Control of safety is satisfied if point U (N_U , M_U), representing the force and moment obtained by the loading q_{ULS} of the ULS basic combination, is situated before the point D (N_D , M_D) on the loading direction, or if it is inside the safety zone (b) deducted from (a).

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e d'Étude:

Results of calculations for the 21m-high pier

ULS	FORCES (N, M) AND REMINDER OF CALCULATION COEFFICIENTS			
factor γ_{G}	1.35			
factor γ_Q	1.	35		
POINT U $\begin{pmatrix} N_{\rm U} \\ M_{\rm U} \end{pmatrix}$	$ \begin{pmatrix} 46,719 \\ 24,031 \end{pmatrix} $			
Coefficient λ	2.	95		
POINT A $\begin{pmatrix} N_A \\ M_A \end{pmatrix}$	$\begin{pmatrix} 138,239\\ 91,948 \end{pmatrix}$			
INEQUALITIES	(5.102 a)	Reminder (5.102 b)		
factor γ_{0} ,		1,27		
factor γ_0	1.20			
POINT B $\begin{pmatrix} N_B \\ M_B \end{pmatrix}$	$\begin{pmatrix} 115,199\\ 66,323 \end{pmatrix}$			
factor γ _{Rd}	1.06			
POINT C $\begin{pmatrix} N_C \\ M_C \end{pmatrix}$	$\begin{pmatrix} 108,678\\ 62,569 \end{pmatrix}$			
POINT D $\begin{pmatrix} N_D \\ M_D \end{pmatrix}$	$ \begin{pmatrix} 109,943 \\ 62,333 \end{pmatrix} $	$\begin{pmatrix} 108,850\\61,584 \end{pmatrix}$		
Critical section validated ¹ ?	YES	YES		

¹ Reminder: The section is validated if the point U is before the point D in the loading direction.

The results obtained with the inequality (5.102 b) are there for comparison. The two inequalities give approximately the same force limits (N_D, M_D).

Results of calculations for 32m-high pier

ULS	FORCES (N, M) AND REMINDER OF CALCULATION COEFFICIENTS		
factor γ_{G}	1.35		
factor γ_Q	1.35		
POINT U $\begin{pmatrix} N_{\rm U} \\ M_{\rm U} \end{pmatrix}$	$ \begin{pmatrix} 50,646\\ 39,129 \end{pmatrix} $		
Coefficient λ	1.30		
POINT A $\begin{pmatrix} N_A \\ M_A \end{pmatrix}$	$\begin{pmatrix} 65,839\\61,156 \end{pmatrix}$		
INEQUALITIES	(5.102 a)	Reminder (5.102 b)	





ULS	FORCES (N, M) AND REMINDER OF CALCULATION COEFFICIENTS		
factor y ₀ ,		1.27	
factor γ ₀	1.20		
POINT B $\begin{pmatrix} N_B \\ M_B \end{pmatrix}$	$\begin{pmatrix} 54.866\\ 43,\!176 \end{pmatrix}$		
factor γ _{Rd}	1.06		
POINT C $\begin{pmatrix} N_C \\ M_C \end{pmatrix}$	$\begin{pmatrix} 51,760\\ 40,861 \end{pmatrix}$		
POINT D $\begin{pmatrix} N_D \\ M_D \end{pmatrix}$	$\begin{pmatrix} 52,540\\ 40,976 \end{pmatrix}$	$ \begin{pmatrix} 51,841 \\ 40,176 \end{pmatrix} $	
Critical section validated ¹ ?	YES	YES	

¹ Reminder: The section is validated if the point U is before the point D in the loading direction.

The results obtained with the inequality (5.102 b) are there for comparison. The two inequalities give approximately the same force limits (N_D, M_D) .





:Application of safety format according to inequality (5.102 a) with the couple (axial force, moment)

Justifications of use of expression (5.102b)

It will be recalled that the three verification criteria proposed by Eurocode 2 are as follows:

$$\gamma_{\text{Rd}} \stackrel{\text{effet des actions}}{\overline{E}(\gamma_{\text{G}} \overline{G} + \gamma_{\text{Q}} \overline{Q})} \leq \overline{R}\left(\frac{q_{\text{ud}}}{\gamma_{\text{O}}}\right) \quad \text{Inégalité (5.102 a)}$$

ou

$$\underbrace{\overline{E(\gamma_{G}G + \gamma_{Q}Q)}^{\text{effet des actions}}}_{E(\gamma_{G}G + \gamma_{Q}Q)} \leq \underbrace{R(\underbrace{q_{ud}}{\gamma_{O'}})}_{R(\underbrace{q_{ud}}{\gamma_{O'}})} \text{Inégalité (5.102 b)}$$

ou

$$\gamma_{\text{Rd}} \gamma_{\text{Sd}} \underbrace{\widetilde{E(\gamma_{\text{g}}G + \gamma_{\text{q}}Q)}}_{\text{E}(\gamma_{\text{g}}G + \gamma_{\text{q}}Q)} \leq \underbrace{R\left(\frac{q_{\text{ud}}}{\gamma_{\text{O}}}\right)}_{\text{Rd}}$$
 Inégalité (5.102 c)

with

 γ_{Rd} = 1.06 partial factor associated with the uncertainty of the resistance model





 $\gamma_{sd} = 1.15$ partial factor associated with model uncertainty, actions and/or their effects

 γ_g partial factor relative to permanent forces G, not taking account of model uncertainties

 γ_q partial factor relative to variable forces, not taking account of model uncertainties

 $\gamma_G = \gamma_{Sd} \gamma_g$ partial factor relative to permanent loads G

 $\gamma_Q = \gamma_{Sd} \gamma_q$ partial factor relative to variable loads Q

 $\gamma_0 = 1.20$ global safety factor

$$\gamma_{\rm O'} = \gamma_{\rm O} \times \gamma_{\rm Rd} = 1.27$$

The national annex has re-written these three inequalities in a form in compliance with the instructions of Eurocode 0.

$$\widehat{E}\left(\gamma_{G}G + \gamma_{Q}Q\right) \leq \frac{1}{\gamma_{Rd}} \times \qquad \widehat{R}\left(\frac{q_{ud}}{\gamma_{O}}\right) \qquad \text{Inégalité (5.102 a)}$$
ou
$$\widehat{E}\left(\gamma_{G}G + \gamma_{Q}Q\right) \leq \frac{1}{\gamma_{Rd}} \times \qquad \widehat{R}\left(\frac{q_{ud}}{\gamma_{O'}}\right) \qquad \text{Inégalité (5.102 b)}$$
ou
$$\widehat{E}\left(\gamma_{G}G + \gamma_{Q}Q\right) \leq \frac{1}{\gamma_{Rd}} \times \qquad \widehat{R}\left(\frac{q_{ud}}{\gamma_{O'}}\right) \qquad \text{Inégalité (5.102 b)}$$
ou
$$\widehat{E}\left(\gamma_{g}G + \gamma_{q}Q\right) \leq \frac{1}{\gamma_{Rd}} \times \qquad \widehat{R}\left(\frac{q_{ud}}{\gamma_{O}}\right) \qquad \text{Inégalité (5.102 c)}$$

It is generally required that one of the three inequalities be satisfied. A more detailed examination however allows an awareness of the fact that the three inequalities are not totally equivalent and that each one is valid under precise conditions. For this it is necessary to revert to the general verification format defined by Eurocode 0 and briefly mentioned in [Chapter 2 IV].

The general inequality to verify is $E_d \le R_d$ with as a first member the effects of actions and as a second the domain of resistance [EC0 Expr.(6.8)]. The resistance domain is defined by the whole of the limits of the actions effects reached in a given section of the structure.

This inequality in fact comes as several inequalities when each member is expressed by bringing up in an explicit way the various partial model factors:

$$E_d = \gamma_{Sd} E(\gamma_F F) \text{ or } E_d = E(\gamma_F F) \qquad [EC0 Expr.(6.2) \text{ and } (6.2a)].$$

et $R_d = (1/\gamma_{Rd}) R(X/\gamma_m)$ or $R_d = R(X/\gamma_M)$ [EC0 Expr.(6.6) and (6.6a)].

There are two ways to express the first and second members, thus there are in total four possible ways to express the verification criterion.

Eurocode 0 makes clear that in fact none of these expressions relate to buckling [EC0 6.4.2(3)P]. Buckling may in effect occur before the materials' resistance limits are reached, in which case there is a question regarding the choice of symbol to put on the second member to define the corresponding limit state. For simplicity Eurocode 2 part 2 kept the symbol R and chose to use the applied loads to define the limits; whence the use of $R(q_{ud})$, even if, in the case of buckling, it is no longer resistance in the strict sense of the word that intervenes.

By use of the symbols from Eurocode 2 part 2, $(\gamma_G G + \gamma_Q Q)$ for the loads corresponding to the ULS basic combination, and q_{ud} for the design ultimate failure load, the safety verification criterion is expressed by noting



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that the maximum values of the actions effects are limited by the minimum values of the limits of "resistance", or by the inequality:

$$E(\gamma_G G + \gamma_Q Q) < R(q_{ud} / (\gamma_{Rd} \times \gamma_o))$$

The behavior called **"over-proportional"** is found when the actions effects increase faster than the actions; this particularly happens in the case of buckling. The basic inequality may be detailed by the following inequalities:

$$\underbrace{\gamma_{\text{Sd}} E\left(\frac{\gamma_{\text{G}}}{\gamma_{\text{Sd}}} G + \frac{\gamma_{\text{Q}}}{\gamma_{\text{Sd}}}Q\right) < E\left(\gamma_{\text{G}}G + \gamma_{\text{Q}}Q\right) < R\left(\frac{q_{\text{ud}}}{\gamma_{\text{Rd}} \times \gamma_{\text{O}}}\right) < \frac{1}{\gamma_{\text{Rd}}} \times R\left(\frac{q_{\text{ud}}}{\gamma_{\text{O}}}\right)}_{\text{infealitie}}$$

It is easy to recognize in this series of inequalities the three inequalities a), b) and c) of Eurocode 2 part 2 rewritten correctly. It is also easy to see that the only verification of inequality b) ensures the verification of the two inequalities a) etc).

On the other hand, with the behavior called **"sub-proportional"**, or when the actions effects increase more slowly than the actions (a plasticized section for example), the series of inequalities is written differently, as below:

$$\overbrace{E(\gamma_{G}G + \gamma_{Q}Q) < \underbrace{\gamma_{Sd}}_{in\acute{e}galit\acute{e}} \underbrace{(5.102b)}_{(5.102c)}}^{in\acute{e}galit\acute{e}} \underbrace{(5.102b)}_{(5.102c)} < \frac{1}{\gamma_{Rd}} \times R\left(\frac{q_{ud}}{\gamma_{O}}\right) < R\left(\frac{q_{ud}}{\gamma_{Rd} \times \gamma_{O}}\right)$$

It is also easy to ensure that inequalities a) and b) are satisfied as soon as inequality c) is verified.

In conclusion, for the behavior called "**over-proportional**" (e.g. buckling), the inequality (5.102b) makes it safe and for the behavior called "**sub-proportional**", it is inequality (5.102c) that makes it safe. The inequality (5.102a) which is only a simplified variation of inequality (5.102c) without showing the partial factor of the side of the actions effects of the first member, has only limited interest.

The guide recommends use of the inequality (5.102b) in all cases, since it is a particularly simple use: it does not require determination of interaction diagrams; it prevents having to ask if the behavior may be "over or sub" proportional. Further it looks to safety in the case of buckling. And lastly the inaccuracy it causes relative to safety in the case of sub-proportional behavior is acceptable since the partial factor for the inequality model only intervenes for a value of 1.06 in a global partial factor of 1.27.

Summary of results of the four methods on the embedded section

	EC2-1-1			EC2-2
STRESSES UNDER LOAD q _{uls}	METHOD BASED ON 1/r	METHOD BASED ON EI	GENERAL METHOD	GENERAL METHOD
\mathbf{N}_{Ed}	46.719			
M _{0Ed}	22.838			

Results of calculations for the 21m-high pier



Verification of form stability - simplified methods and example of two piersAppendix VI - Verification of form stability - Simplified



\mathbf{M}_{Ed}	41.199	48.572	24.441	24.031
M_{Ed}/M_{0Ed}	1.80	2.13	1.07	1.05

The pier is correctly dimensioned relative to the stresses given by the four structural analysis methods

Results of calculations for the 32m-high pier

	EC2-1-1			EC2-2
STRESSES UNDER LOAD q _{ULS}	METHOD BASED ON 1/r	METHOD BASED ON EI	GENERAL METHOD	GENERAL METHOD
N _{Ed}	50.646			
MoEd	33.322			
M _{Ed}	78.853 (*)	310.82 (*)	43.220	39.129
M _{Ed} /M _{0Ed}	2.37	9.32	1.30	1.17

(*) The stresses obtained by the two simplified methods greatly exceed the resisting moment of the critical section.

Conversely, the pier is correctly dimensioned relative to the stresses given by the two general methods.

This example confirms that the simplified methods are very safe and that the general methods are to be preferred to obtain a more realistic and accurate dimensioning of slender structures.





ANNEXE XI.EXAMPLES OF CALCULATION OF CRACK OPENINGS AT SLS

Eurocode allows treatment of a large number of scenarios. The following examples allow illustration of the different calculation methods. The calculations are not systematically detailed, but all the calculation hypotheses are given, enabling the reader to find the numerical values of the results.

PSIDP corbel (transverse bending, reinforced concrete)

Section at gusset level, guide GS2 side, (thickness 35cm) is verified.

 $M_{QP} = 24 \text{ kNm/m}$

 $M_{\rm fréq} = 93 \text{ kNm/m}$

 $M_{carac} = 119 \text{ kNm/m}$

 $M_{ULS} = 161 \text{ kNm/m}$

The dimensioning of the bent reinforcement at ULS gives $A_s = 12 \text{ cm}^2/\text{m}$.

Or a reinforcement with HA14 s = 125 ($A_s = 12.3 \text{ cm}^2/\text{m}$), with a cover c = 30mm.

Calculation of stresses under characteristic SLS:

The calculation is done in a cracked section with an equivalence coefficient n = 15 (cf chapter 4 of this guide).

 σ_s = 342 MPa < $0.8 f_{yk}$ = 400 MPa for B500 steels

Verification of crack opening under frequent SLS:

The calculation is done with an equivalence coefficient n = 15 (cf chapter 4 of this guide) and gives successively:

x = 91 mm (compressed height in cracked section)

 $h-d = c+\phi/2 = 37mm$

 $\sigma_s = 267 \text{ MPa}$

 $k_t = 0.6$ (combination of force corresponding to a loading essentially short-term)

 $h_{c ef} = min [2.5 \cdot (h-d); (h-x)/3] = min [2.5 \cdot 37; (350-91)/3] = 86 mm$

 $\rho_{eff} = A_s / h_{c.ef} = 12.3.10^{-4} / 0.086 = 1.43\%$

 ε_{sm} - ε_{cm} = 0,6 σ_s/E_s = 801 μ m/m

(the first part of [EC2-1-1 Expr.(7.9)] gives a value less than this lower limit)

 $k_1 = 0.8$

 $k_2 = 0.5$

 $s_{r,max} = k_3 \cdot c + k_1 \cdot k_2 \cdot k_4 \cdot \phi / \rho_{eff} = 3.40 \cdot 30 + 0.8 \cdot 0.5 \cdot 0.425 \cdot 14 / 1.43\% = 269 \text{ mm}$

 $w_k = (\varepsilon_{sm} - \varepsilon_{cm}) \cdot s_{r,max} = 801.10^{-6} \cdot 269 = 0.22 \text{ mm} < 0.30 \text{ mm}$

The limitation of the crack opening at 0.3mm under frequent SLS is covered by the ULS reinforcement. However, the bearing capacity of the reinforcement is high, with a great risk of



fatigue. In this example it is advisable either to verify the fatigue of the reinforcement, or to increase their section to limit the bearing capacity to 300 Mpa under characteristic SLS.

<u>PSIDP (longitudinal bending, in partial prestress)</u>

General data

In this example, the longitudinal prestress was dimensioned on the basis of the minimum Eurocode criterion, or the non-decompression under quasi-permanent combination, to obtain operation of the structure in partial prestress.

This dimensioning leads to the following prestress: $15 \times 12T15S$ tendons (A_p = 15×1800 mm²), whose layout is given below:



: PSIDP cabling

In this example the case of the section in the middle of the large span is dealt with

At mid-span, the prestress line is $e_{00} = -0.213$ m.

The distance between the upper slab and the center of gravity of the tendons is:

d_p=v+0,380=0,386+0,380=0,766 m

The reinforcement area is $A_s = 72 \text{ cm}^2$ (section dimensioned at bending ULS), based on HA20.

The distance between the upper slab and the center of gravity of the steels is $d_s=0,84$ m

Stress values:

 $P_{m,CT} = 34.99 \text{ MNm}$ (short-term value)

 $P_{m,LT} = 32.32 \text{ MNm}$ (value after recorded losses) $P_{k,inf,LT} = 0.9P_{m,LT} = 29.09 \text{ MN}$

 $M_{g+g'} = 9.39 \text{ MNm}$

 $M_{GT} = 0.85$ MNm (moment due to positive thermal gradient)

 $M_{LM1,fréq} = 4.77 \text{ MNm}$

 $M_{LM1,carac} = 7.40 \text{ MNm}$

The section's mechanical characteristics given in appendix.

Calculation of stresses under SLS

Under Quasi-Permanent SLS, the section is compressed. Calculation of the stresses is thus carried out on a non-cracked section. For simplicity, all the stress calculations were done in gross section. The QP SLS combination giving the lowest compression in lower fiber is obtained for the moment:



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 $M = M_{g+g'} + 0.5M_{GT} + P_{k,inf,LT} \cdot e_{00} = 3.62 \text{ MNm}$

The corresponding stresses are:

 $\sigma = 6.2$ MPa in upper fiber

 $\sigma' = 0.2$ MPa in lower fiber

The stress at the level of the prestressing tendons' center of gravity is $\sigma_{cp} = 1.0$ MPa

Under characteristic SLS, the tensile stresses ion lower fiber exceed f_{ctm} . The calculations are thus done in a cracked section .

When there is cracking, it is acceptable that the bonded tendons in the tensioned zone contribute by their stress increases to control of cracking over a distance ≤ 150 mm from the reinforcement center.

The calculations are done from coefficients of equivalence E_s/E_{cm} and E_p/E_{cm} (i.e. no account taken of creep, in compliance with the recommendations of chapter 4 of this guide).

The stress increase of prestressing steels beyond the nil state of strain of the adjacent concrete is balanced by

 $\xi_1 = \sqrt{\xi \cdot \frac{\phi_s}{\phi_p}}$, ratio of adherence capabilities of reinforcement for both prestressed concrete and reinforced

concrete. For calculation of stresses, this term is used for all the prestressing steels in the tensioned concrete (whereas for the calculations of crack openings and for the minimum reinforcement for control of cracking, only the stress increase of the prestressing steels situated in $A_{c,eff}$ is counted; see the digital application below)

The initial stress increase (before return to zero, noted $\Delta \sigma'_p$) is integrally counted.

From table 6.2 of EC2, $\xi = 0.5$ (prestress by bonded post-tension made up of strands, $f_{ck} < 50$ MPa)

$$\phi_{\rm s} = 20 \text{ mm}, \ \phi_{\rm p} = 1.6 \sqrt{A_{\rm p}} = 1.6 \sqrt{1800} = 68 \text{ mm}, \ \xi_{\rm 1} = \sqrt{0.5 \cdot \frac{20}{68}} = 0.34$$

Further, $\Delta \sigma'_{p} = \sigma_{cp} \cdot \frac{E_{p}}{E_{cm}} = 6.0 \text{ MPa}$

The equilibrium equations for calculation in cracked section are as follows:

$$N = F_{c} - A_{s} \cdot \sigma_{s} - A_{p} \cdot (\Delta \sigma'_{p} + \xi_{1} \Delta \sigma_{p})$$

where $F_{\rm c}$ is the compressive stress in the concrete

$$M = M_c + A_s \cdot \sigma_s (d_s - v) + A_p \cdot (\Delta \sigma'_p + \xi_1 \Delta \sigma_p) (d_p - v)$$

where M_c is the moment due to compressive stresses in the concrete, expressed in relation to the section's center of gravity.

Further, the equations of strain compatibility are written, and the system obtained is resolved.

The calculations are not detailed here and the following results are obtained:

Under frequent SLS:

$$N = P_{k,inf,LT} = 29,09 \text{ MN}$$
$$M = M_{g+g'} + 0.5M_{GT} + M_{LM1,fréq} + P_{k,inf,LT} \cdot e_{00} = 8,39 \text{ MNm}$$





$$\Delta \sigma_{\rm p} = 26 \,_{\rm MPa}, \, \Delta \sigma_{\rm p} + \Delta \sigma'_{\rm p} = 32 \,_{\rm MPa}$$

 $\sigma_c = 13,2$ MPa (compressive stress in upper fiber)

x = 0.299m (position of neutral axis, useful for crack opening calculation)

Under characteristic SLS:

 $\sigma_s = 94 \text{ MPa}$

$$N = P_{k,inf,LT} = 29,09 \text{ MN}$$

$$M = M_{g+g'} + 0,6M_{GT} + M_{LM1,carac} + P_{k,inf,LT} \cdot e_{00} = 11,11 \text{ MNm}$$

$$\sigma_{s} = 298 \text{ MPa}$$

$$\Delta \sigma_{p} = 86 \text{ MPa}, \ \Delta \sigma_{p} + \Delta \sigma'_{p} = 92 \text{ MPa}$$

 $\sigma_c = 21,0$ MPa (compressive stress in upper fiber)

Verification of stresses under characteristic SLS

Under characteristic SLS, the following verifications are done:

 $\begin{aligned} \sigma_{s} &= 298 \text{ MPa} < 0.8 \text{ f}_{yk} = 400 \text{ MPa} \\ \sigma_{c} &= 21.0 \text{ MPa} < 0.6 \text{ f}_{ck} = 21 \text{ MPa} \\ \sigma_{pm} &= \frac{P_{m}}{A_{p}} + \Delta \sigma_{p} + \Delta \sigma'_{p} = \frac{32.32}{15 \cdot 1800 \cdot 10^{-6}} + 92 = 1289 \text{ MPa} < 0.80 \text{ f}_{pk} = 0.80 \times 1860 = 1488 \text{ MPa} \end{aligned}$

The limits are respected.

Verification of crack opening under frequent SLS

Here the calculation of crack opening by the direct method [7.3.4] is detailed. Since the section is approximately rectangular in the tensioned zone, the proposed formulae may be applied.

Or:

 $σ_s=94$ MPa $h_{c,ef} = min (2.5.(h-d_s); (h-x)/3) = min (2.5.60; (900-299)/3) = 150 mm$ $A_{c,ef} = 6.20 \times 0.150 = 0.93 m^2$ $A'_p = A_p = 15 \times 1800 mm^2$ (since all prestressing tendons are situated in $A_{c,eff}$) $ρ_{p,eff} = (A_s + \xi_1^2 A'_p)/A_{c,eff} = 1.11 \%$ $k_t = 0.6$ $ε_{sm} - ε_{cm} = 0.6 σ_s/E_s = 282 µm/m$ (the first term of equation (7.9) of EC2 gives a negative result) $s_{r,max} = 3.4 \times 30 + 0.425 \times 0.8 \times 0.5 \times 20 / 1.11 \% = 408 mm$ $w_k = s_{r,max} \times (ε_{sm} - ε_{cm}) = 282.10^{-6} \times 408 = 0.12 mm$





The value obtained is less than the acceptable limit of 0.2mm.

Calculation of minimum reinforcement

The minimum reinforcement is not dimensioning here, since it corresponds to the reinforcement that would be required to balance the cracking moment (i.e. the moment creating a tension $f_{ctm}=3.2$ MPa in lower fiber, with the axial stress $P_{k,inf,LT}$), whereas the moment under characteristic SLS is much higher to the cracking moment.

The calculation is carried out however to illustrate the procedures.

Since the section is approximately rectangular in the tensioned zone, formula (7.1) may be applied. By ignoring the contribution of the prestressing steels, this gives successively:

 $M_{fiss} = (N/S + f_{ct,eff}) I/v' = 7.11 MNm$

Under M_{fiss} and $P_{k,inf,LT}$, the tensioned height before cracking is 24cm

 $A_{ct} = 0.24 \times 6.2 = 1.5 \text{ m}^2$

 $\sigma_{\rm c} = N/S = P_{\rm k,inf,LT} \ / \ S = 3.6 \ MPa$

 $k_1 = 1.5$ (compressed section)

 $h^* = h (car h < 1.0m)$

 $k_c = 0.4 [1 - 3.6/(1.5 \times 3.2)] = 0.1$

k = 1.0 (since the stresses are considered as due to outside stresses)

 $A_{s,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / f_{yk} = 0.1 \times 1.0 \times 3.2 \times 1.5 / 500 = 9.6 \text{ cm}^2$, less than the reinforcement placed.

An alternative is to calculate the stresses in a cracked section under the effect of the moment of cracking, and to verify that the stresses in the reinforcement are less than 500 MPa.

Tie rod in reinforced concrete and prestressed concrete subjected to external loads

This example deals simultaneously with prestressing and reinforcing steels.

Outline:

- tie rod 250×250 , concrete C35/45
- 4HA16 situated at four corners (distance $c+\phi/2 = 50$ mm), $f_{yk} = 400$ MPa
- 2 bonded mono-strands, T13S, $E_p = 200\ 000\ MPa$, $\sigma_{p0} = 1350\ MPa$ (tension in the tendons before release of cylinder)

External loading (except for prestress) N = -430 kN (traction)

Calculation of crack openings for a short-term loading

General data

 $A_s = 800 \text{ mm}^2$

 $A_p = 200 \text{ mm}^2$, $P_0 = A_p \cdot \sigma_{p0} = 270 \text{ kN}$

 $A_c = 250^2 - 800 - 200 = 61\ 500\ mm^2$ (final section)

Equivalent diameter of prestressing strands: the T13S strands are made up of a central wire of 4.40 mm diameter and of 6 peripheral wires of 4.25 mm diameter. The formula proposed in 6.8.2(2)P is $\phi_P = 1.75 \phi_{wire}$. or $\phi_P = 1.75 \times 4.25 = 7.44$ mm



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Ratio of adherence capacity of strands to that of HA steels: $\xi = 0,6$ (strands in pretension) Ratio of adherence capacity of strands to that of HA steels, corrected for their diameters:

$$\xi_1 = \sqrt{(0.6 \times 16 / 7.44)} = 1.136$$

This special case gives $\xi_1 > 1$. Although not explicitly stated, it would be against the nature of the text to take a value greater than 1.0. $\xi_1 = 1$ will thus be used for the stress calculation (same adherence).

Calculation of crack opening

Calculation of stress in steels at crack level:

The calculation is done with the low value of the prestress range: $r_{inf} = 0.90$

 $N_{ext} = \textbf{-}r_{inf} \!\!\times \!\! A_p \!\!\times \!\! \boldsymbol{\sigma}_p - A_s \!\!\times \!\! \boldsymbol{\sigma}_s$

$$\sigma_p = E_p \times \varepsilon_p$$

 $\sigma_s = E_s \times \varepsilon_s$

 $\varepsilon_{p} = \varepsilon_{p0} + \varepsilon_{s}$ (case of pretension)

 $=> N_{ext} = -r_{inf.} \times P_0 - (r_{inf} \times A_p + A_s) \times \sigma_s$

 $\sigma_s = (-0.9 \times 270 + 430) / (0.9 \times 200 + 800) = 191 \text{ MPa}$

 $\sigma_{\rm p} = 0.9 \times 1350 + 191 = 1406 \text{ MPa}$

In this calculation, the same adherence between different steels is assumed, in compliance with earlier comments on the calculation in a cracked section.

Calculation of spacing between cracks:

Eurocode does not deal in a general way with the simultaneous contribution of reinforcement and prestressing steels to crack control. A certain number of assumptions must be made in this paragraph to be able to use the proposed formulae.

 $k_1 = (800 \times 0.8 + 200 \times 1.6) / (800+200) = 0.96$

Nota: the value of 1.6 for the bonded strands is certainly strict, as this value applies mostly in the case of smooth bars. The calculation is thus safe. Moreover, in the absence of information in EN1992, it was decided here to use a value of k_1 intermediate determined from a balancing of the reinforcement sections.

$k_2 = 1,0$ (pure tension)

 $\Phi_{\rm eq} = 16 \, \rm mm$

The balancing formula (7.12) of ϕ_{eq} applies only in the case of reinforcement of different diameters. Here there is only one reinforcement reference diameter, and this is the value to introduce in the formula.

It should be noted that in the general case the definition $\xi_{1}^{2} = \xi \phi_{eq} / \phi_{p}$ should be used

 $A_{c,eff} = A_c$

In effect, the useful area of each reinforcing steel is delimited by a square of 2.5(h-d) = 125mm side. The useful areas of the four reinforcing steels thus suffice to cover the section, without even counting the prestressing steel.





 $A'_{p} = A_{p} = 200 \text{ mm}^{2}$ $\rho_{p,ef} = (800 + 1.136^2 \times 200) / 61500 = 1.72 \%$ $\phi_{eq} / \rho_{p,ef} = 16 / 1.72\% = 930 \text{ mm}$ $s_{r,max} = 3.4 \times (50-16/2) + 0.425 \times 0.96 \times 1.0 \times 930 = 522 \text{ mm}$ Calculation of ε_{sm} - ε_{cm} : $l_{r} = 0.6$ (short terms coloulation)

$$\alpha_{e} = E_{s} / E_{cm} = 200 \ 000 / 34 \ 077 = 5.87$$

$$\frac{\sigma_{s} - k_{t} \frac{f_{et,eff}}{\rho_{p,eff}} (1 + \alpha_{e} \rho_{p,eff})}{E_{s}} = 0.33 \ mm/m \qquad < 0.6 \ \sigma_{s} / E_{s} = 0.57 \ mm/m$$

Calculation of w_k:

 $W_k = 522 \times 0.57 = 0.30 \text{ mm}$

Verification of minimum reinforcement

Calculation in non-cracked section under applied stress, with $r_{inf} = 0.9$ $N_{ext} = A_c \times \sigma_c - r_{inf} \times A_p \times \sigma_p - A_s \times \sigma_s$ $\sigma_{\rm p} = E_{\rm p} \times \varepsilon_{\rm p}$ $\sigma_{\rm s} = E_{\rm s} \times \varepsilon_{\rm s}$ $\sigma_{\rm c} = E_{\rm c} \times \varepsilon_{\rm c}$ $\varepsilon_{\rm p} = \varepsilon_{\rm p0} + \varepsilon_{\rm s}$ (case of pretension) and $\varepsilon_{\rm c} = -\varepsilon_{\rm s}$ $=> N_{ext} = -r_{inf} \times P_0 - (A_c / \alpha_e + r_{inf} \times A_p + A_s) \times \sigma_s$ $\sigma_s = (-0.9 \times 240 + 430) / (0.9 \times 200 + 800 + 61500 / 5.87) = 16.3 \text{ MPa}$ $\sigma_{\rm c} = -\sigma_{\rm s} / \alpha_{\rm e} = -2.8$ MPa The concrete is tensioned, thus minimum reinforcement must be used [EC2-1-1 7.3.2(4)] $k_c = 1.0$ (pure tension)

k = 1.0 (tension due to external loads) $f_{ct.eff} = f_{ctm} = 3.2 \text{ MPa}$ $A_{ct} = A_c = 61500 \text{ mm}^2$ $\sigma_s = f_{yk} = 500 \text{ MPa}$ $A_{s,min} = (1.0 \times 1.0 \times 3.2 \times 61500) / 500 = 394 \text{ mm}^2$ It is the reinforcement required to balance a tensile stress of 3,2 MPa in the concrete

 $A_{s,min} > A_s$: the reinforcement planned is sufficient.

Nota: it would have been possible to take account of the contribution of the bonded strands to determine the minimum reinforcement, [EC2-1-1 7.3.2(3)]: $A_{s,min} = 394 - \xi_1 A_p \Delta \sigma_p / \sigma_s$ (useless here)

<u>Reinforced concrete slab subjected to delayed shrinkage</u>


This example corresponds to a shear wall cast in-situ between two existing, perfectly rigid, concrete structures.

This type of problem is dealt with in a general way in Eurocode 2 part 1-1, and taken up in detail in Eurocode 2 part 3 (silos and tanks), where problems with crack control under delayed strains are particularly sensitive. So Eurocode 2 part 3 will be used as reference when necessary.

Description of slab:

- slab 1m \times 0.24m, concrete C35/45, steels f_{yk} = 500 MPa
- reinforcement HA12 s=200 in upper layer and HA14 s=200 in lower layer, c = 3cm

Choice of value of f_{ct,eff}

Since this is a problem of imposed strain, the date on which cracking is likely to occur should be determined: at 28 days or before ?

 $N = -E_c.A_c.\varepsilon_{cs}$ axial hyperstatic stress due to delayed shrinkage of concrete

 $\sigma_c = N / (A_c + \alpha_e A_s) \approx N / A_c$ stress in concrete (calculation in non-cracked section)

The curve $\sigma_c(t)$ may be drawn and compared to the tensile strength of the concrete:



: Comparison between tensile stress due to delayed shrinkage and f_{ctm}(t)

Evolution des contraintes dans le béton	Development of stresses in concrete
Contraintes (Mpa)	Stresses (Mpa)
Temps (jours)	Time (days)
Contrainte sous l'effet du retrait	Stress under effect of shrinkage
Résistance du béton fctm(t)	Resistance of concrete fctm(t)



From these curves it cannot be said that cracking under the effect of delayed shrinkage will occur before 28 days, hence $f_{ct,eff}$ is chosen as equal to f_{ctm}

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Calculation of stresses in steels for calculation of crack openings:

Eurocode 2 part 1-1 gives the value to take account of for a tie rod subjected to delayed strains:

$$\sigma_{s} = k_{c} \times k \times f_{ct,eff} \times A_{ct} / A_{s} \qquad [EC2-1-1 \ 7.3.3(2) \ note]$$

Eurocode 2 part 3 confirms this value:

 $\sigma_{s} = k_{c} \times k \times f_{ct,eff} / \rho_{s} \qquad [EC2-3 Anx.M Expr.(M.2)]$

Coefficients:

 $k_c = 1.0$ for a tie rod in pure tension

k = 0.65 (coefficient of reduction specific to stress calculation under imposed strains)

Here the two layers are not symmetrical, hence the coefficient k_c is not strictly equal to 1.0. Calculation of the stresses in the two layers may be done directly when the tie rod is subjected to stresses.

 $N = -k \times f_{ct,eff} \times A_c$ and M = 0

Which gives $\sigma_{s,sup} = 440$ MPa and $\sigma_{s,inf} = 330$ MPa (instead of $\sigma_s = 0.65 \times f_{ct,eff} \times A_{ct}/A_s = 374$ MPa if the reinforcement had been symmetrical)

i.e.: $k_{c,sup} = 440 / 374 = 1.17$ for the upper layer, and $k_{c,inf} = 330 / 374 = 0.87$ for the lower layer

Calculation of crack opening in upper layer:

 $h_{c,ef} = min (2.5 \times (h-d); h/2) = min (2.5 \times 36; 120) = 90 mm$

 $A_{c,ef} = 0.09 \text{ m}^2$

$$\rho_{\rm ef} = A_{\rm s,sup} / A_{\rm c,ef} = 0.63 \%$$

 $k_2 = (440+330)/(2 \times 440) = 0.87$ (the coefficient $k_{c,inf}$ is found, calculated above)

 $s_{r,max} = 3.4 \times 30 + 0.425 \times 0.8 \times 0.87 \times 12 \ / \ 0.63 \ \% = 670 \ mm$

 $\varepsilon_{sm} - \varepsilon_{cm} = \max \left[\left(\sigma_s - k_t \times f_{ct,eff} / \rho_{p,eff} \cdot (1 + \alpha_e \times \rho_{p,eff}) \right) / E_s; 0.6 \sigma_s / E_s \right]$ [EC2-1-1 Expr.(7.9)] with $k_t = 0.4$

 $\varepsilon_{sm} - \varepsilon_{cm} = \max \left[(440 - 0.4 \times 3.2/0.63\%.(1 + 5.86 \times 0.63\%)) / 200000; 0.6 \times 440 / 200000 \right]$

= max [1.18 mm/m; 1.32 mm/m] = 1.32 mm/m

Eurocode 2 part 3 gives a formula slightly different for this specific case: $\varepsilon_{sm} - \varepsilon_{cm} = 0.5 \sigma_s/E_s$, instead of the expression (7.9) in Eurocode 2 part 1-1, or 1.10 mm/m. This last formula, which is more favorable, will be used.

 $w_k = 1.10.10^{-3} \times 670 = 0.74 \text{ mm}$

This expression is valid near the steels. The spacing of 200mm being greater than $5(c+\phi/2) = 180$ mm, the expression (7.14) is also used, or

 $s_{r,max} = 1.3h = 312 \text{ mm}$

 $w_k = 1.10.10^{-3} \times 312 = 0.34 \text{ mm}$

This result is not logical, since it gives a crack spacing that is greater at the steel level than between two steels (contrary to figure 7.2 in Eurocode 2 part 1-1). This shows the limits of the formulae, which tend much more to the side of beams under bending than for slabs under tension. For a slab the experimental results show that it would be more just to use $s_{r,max} = 1.3.max(s; h)$, where s is the spacing of the steels, although it doesn't change the result here.



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Calculation of crack opening in lower layer:

$$\begin{split} h_{c,ef} &= min \; (\; 2.5 \times (h\text{-d}); \, h/2 \;) = min \; (\; 2.5 \times 37; \; 120 \;) = 92 \; mm \\ A_{c,ef} &= 0.0925 \; m^2 \\ \rho_{ef} &= A_{s,inf} \; / \; A_{c,ef} = 0.83 \; \% \\ k_2 &= 0.87 \; (unchanged) \\ s_{r,max} &= 3.4 \times 30 + 0.425 \times 0.8 \times 1.17 \times 14 \; / \; 0.83 \; \% = 601 \; mm \\ \epsilon_{sm} - \epsilon_{cm} &= 0.82 \; mm/m \qquad [EC2-3 \; Anx.M.1] \\ w_k &= 0.82.10^{-3} \times 601 = \textbf{0.49 } \; \textbf{mm} \end{split}$$

For interest, calculation far from the steels:

 $s_{r,max} = 1.3h = 312 \text{ mm}$

 $w_k = 0.82.10^{-3} \times 312 = 0.26 \text{ mm}$

Summary

The reinforcement placed is not enough to completely take up the stresses due to imposed strains.

Reinforcement to place to balance imposed strains with $w_k = 0.3 \text{ mm}$

The same steel diameters are adopted for both upper and lower fibers, to balance the problem. Thus $k_c = 1.0$

The minimum non-brittleness reinforcement is given by:

 $A_{s,min} = k_c \times k \times f_{ct,eff} \times A_{ct} / f_{vk} = 1.0 \times 0.65 \times 3.2 \times 0.24 / 500 = 10.0 \text{ cm}^2$, or two layers of HA12 s = 200.

As previously seen, this reinforcement is clearly insufficient to respect a crack opening of 0.3mm. Table 7.2N may be used to dimension the reinforcement (simplified method):

For example with steels HA14:

$$\phi_s = 14 \text{ mm}$$

 $\phi_s^* = \phi_s \times 2.9/f_{ct,eff} \times 8 \times (c+\phi/2) / h_{cr} = \phi_s \times 2.9/3.2 \times 8 \times 0.037 / 0.24 = 1.12 \phi_s = 16 \text{ mm}$
Whence $\sigma_s = 240 \text{ MPa}$ (table 7.2N for $w_k = 0.3 \text{ mm}$)

Knowing the bearing capacity to use for the HA 14, the minimum reinforcement formula is used to determine a spacing.

Whence ρ = $k_c \times k \times f_{ct,eff}$ / σ_s = 1.0×0.65×3.2 / 240 = 0.86 % As=0.0086×24/2.1=10.3cm²/ml or 6.7 HA14/ml

Whence s = 150 mm

A verification by the calculation according to the direct method shows that the choice of two beds of HA14 s = 150 gives $w_k = 0.31$ mm and in this case the two methods give similar results.

Reinforced-concrete shear wall cast in-situ over an older concrete.

This example allows modeling of the behavior of a lower slab of a segment cast in-situ against an alreadyhardened segment, or a pier raising on the previous raising.

The previous example allowed determination of the reinforcement necessary assuming the shear wall is perfectly laterally embedded, which is not the case for the slab of a box bridge or for a pier raising. The



previous shear wall example is thus used again, with a reinforcement less than that resulting from the previous calculation, or HA12 and HA14 with a spacing of s = 200.

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The width of this shear wall is 5m, its height 3.30m and a completion time of 7 days is assumed.

The problem studied is as follows: transverse reinforcement to tie together the effects of differential shrinkage and of thermal shrinkage during concreting.



: Illustration of problem dealt with: cracking of a shear wall fixed one side only

This problem is dealt with in detail in Eurocode 2 part 3, in particular in annexes L and M.

The crack opening is calculated from the expression $\varepsilon_{sm} - \varepsilon_{cm} = R \times \varepsilon_{free}$, where ε_{free} is free shrinkage that occurs in the absence of blockage, and R a reduction coefficient linked to the type of blockage.

For the central part of a shear wall with a L / H ratio = 5.0 / 3.3 = 1.5 the recommended value of R is R = 0,5 at the base of the wall , and R = 0 at the top of the wall (table L.1)

The reinforcement is dimensioned for the most-stressed zone, or R = 0.5

 $\varepsilon_{\text{free}}$ is the sum of two terms:

- thermal shrinkage: for example, for a temperature increase of 40°C, $\varepsilon_{\text{free}} = \alpha.\Delta T = 0.4 \text{ mm/m}$
- differential shrinkage between the two elements: it is given by the curve $\varepsilon_{cd}(t+7) \varepsilon_{cd}(t)$. This curve is at a maximum for t = 0, or $\Delta \varepsilon_{cd} = 0.02$ mm/m (negligible).





: Differential shrinkage between the two elements

Retrait	Shrinkage	
Déformations (mm/m)	Strain (mm/m)	
Temps (jours)	Time (days)	
Retrait du béton support	Shrinkage of support concrete	
Retrait du nouveau béton	Shrinkage of new concrete	
Différence	Inequality	

Whence $\epsilon_{\rm sm}$ - $\epsilon_{\rm cm} \thickapprox 0.5 \times 4.10^{\text{-4}} = 0.20 \ mm/m$.

Calculation of crack opening:

- upper slab: the result is unchanged $s_{r,max}\,{=}\,670$ mm, whence $w_k\,{=}\,0.13$ mm ${<}\,0.3$ mm
- lower slab: $s_{r,max} = 770 \text{ mm}$, $w_k = 0.15 \text{ mm} < 0.3 \text{ mm}$.

The reinforcement planned is sufficient.



div VIII Dibliggraph



ANNEXE XII.BIBLIOGRAPHY

EUROCODES

Eurocode	Title	National annex
NF EN 1990	Basis of structural design	NF P06-100-2
NF EN 1990/A1	Annex A2: Application to bridges	
NF EN 1991-1-1	Actions on structures - Densities, self weight and imposed loads for buildings	NF P06-111-2
NF EN 1991-1-2	Actions on structures - Actions on structures exposed to fire	NF EN 1991-1-2/NA
NF EN 1991-1-3	Actions on structures - Snow loads	NF EN 1991-1-3/NA
NF EN 1991-1-4	Actions on structures – Wind actions	
NF EN 1991-1-5	Actions on structures – Thermal actions	
NF EN 1991-1-6	Actions on structures – Actions during execution	
NF EN 1991-1-7	Actions on structures – Accidental actions from impact and explosions	
NF EN 1991-2	Actions on structures – Traffic loads on bridges	
NF EN 1991-3	Actions on structures – Actions induced by cranes and machinery	
NF EN 1992-1-1	Design of concrete structures – General rules and rules for buildings	NF EN 1992-1-1/NA
NF EN 1992-1-2	Design of concrete structures - General rules - Structural fire design	NF EN 1992-1-2/NA
NF EN 1992-2	Design of concrete structures – Concrete bridges - Design and detailing rules	NF EN 1992-2/NA

NF EN 13670 Execution of concrete structures. Standard project (PR NF EN 13670)

NF EN 206-1 Concrete. Part 1: Specification, performance, production and conformity

NF EN 206-1/A1 et NF EN 206-1/A2 Two amendments to this standard