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Contributions of the Future Eurocode 2 for Assessment of Existing Concrete Structures

Aportaciones del futuro Eurocódigo 2 para la evaluación de estructuras existentes de hormigón

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ABSTRACT

This article highlights the most relevant aspects of the new generation of Eurocodes for the assessment of existing structures, and in particular those governing concrete structures. In this respect, the latest Eurocode 0 will include a new section (*prEN 1990-2. Basis of assessment and retrofitting of existing structures: general rules and actions*) covering the approaches and analysis methods that must be included in this type of assessment, while Eurocode 2 (*FprEN1992-1-1:2023 Design of concrete structures*) includes *Annex I: Assessment of existing structures*, which is informative and covers particular aspects of the assessment of concrete structures.

KEYWORDS: Existing structures, assessment, durability, reliability.

RESUMEN

En el presente artículo se destacan los aspectos más relevantes de la nueva generación de Eurocódigos en lo relativo a la evaluación de estructuras existentes, y en particular, para aquellas de hormigón estructural. En este sentido el Eurocódigo 0 recogerá una nueva parte (*prEN 1990-2. Basis of assessment and retrofitting of existing structures: general rules and actions*) dedicada al planteamiento y análisis que deben recoger este tipo de evaluaciones, en tanto que el Eurocódigo 2 (*FprEN1992-1-1:2022 Design of concrete structures*) incorpora un anejo, el *Annex I: Assessment of existing structures*, de carácter informativo, en el que se recogen algunos aspectos particulares de dicha evaluación para las estructuras de hormigón.

PALABRAS CLAVE: Estructuras existentes, evaluación, durabilidad, fiabilidad.

1. INTRODUCTION

The future Eurocode is sensitive, as it could not be otherwise, to the general interest in the evaluation of existing structures, while reflecting the current state of the art in relation to such evaluation.

In this regard, the new specification will subdivide Eurocode 0 into two parts[1]. The first part would focus on design issues (update of EN1990: *prEN1990:2020* [1]), while the second would cover the assessment of structures: prEN1990-2. Basis of assessment and retrofitting of existing structures: general rules and actions [2]. The latter document has been

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1.- At the time of writing the enquiry stage currently is being completed and it is possible that some adjustments will be introduced before a final version is published. In particular, it is under discussion whether to include the provisions for existing structures in a separate part of EN1990 (in the current version, prEN199O-2: 2022. Basis of structural and geotechnical assessment of existing structures) or to include them in the current Eurocode 0, extending its scope and title to prEN199O: 2022 Basis of structural and geotechnical design and assessment.

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prepared by a horizontal group of *TC250, WG2 Existing Structures*, who had previously drafted *Technical Specification TS 17440* [3] forming the basis for *prEN1990-2*, dated July 2020.

In addition, *FprEn1992* [4] has included *Annex I Assessment of concrete existing structures* with informative purposes. It also has another informative annex - *Annex A Adjustment of partial factors for materials* - which incorporate specifications for the formulation of partial factors in concrete and reinforcement and states particular aspects of the assessment based on core sample tests.

This article aims to highlight the most relevant aspects of *Annex I*, as well as those of *Annex A*, logically based on the principles that should guide this type of assessment as set out in *prEN 1990-2* [2].

Based on the above, the article is divided into 4 sections:

1) *Preliminary considerations for the assessment of existing structures*: here *prEN1990-2* is referred, highlighting aspects that concern both the assessment and the investigation itself.
2) *The assessment of the strength of concrete in existing structures*, in accordance with *Annex A* and notes in *Annex I*.
3) Aspects to be taken into consideration in the *verification of existing concrete structures*, listed in *Annex I*.
4) Considerations on the *durability of existing concrete structures*, also listed in *Annex I*.

Throughout this article, equations, tables, and figures have been numbered sequentially in citation order in it. References to equations, tables, and figures taken directly from *FprEn1992* are additionally given. It does not reproduce sections of the Code in detail and is intended to be read alongside the revised Code.

2.
PRELIMINARY CONSIDERATIONS FOR THE ASSESSMENT OF EXISTING STRUCTURES

The design of a new structure is based on a set of requirements from which the structure is calculated, which then must be built and maintained in accordance with current criteria. As the regulatory criteria used at the time of design the existing structure may be very different than those at the time of assessment, new insight in material behaviour can result in requirements/regulations that differ from those applied during design in the past and should be taken into account in its assessment.

However, the information available will also differ, as it can usually be directly measurable, or at least, general information can be verified by inspection on the completed structure. Therefore, depending on the design and construction records, performed investigations, etc., more precise than that considered for design purposes. It means uncertainties are usually lower.

Finally, considerations regarding the structure's remaining service life are also different; and relative costs of an intervention are usually higher when an existing structure needs to be strengthened.

This means that assessments of these structures must be carried in a way different than that at the time of design.

The initial approach should entail the semi-probabilistic analyses that are normally used in the design of new structures, as they enable updating the overall safety level (β-value) and to update the partial factors based on additional information.

For this approach prEN1990-2 [2] proposes different nomenclature for capacities and loads in Section 8.1, with an emphasis on differentiating the concepts of assessment and design:

Figure 1. Design plan of a bridge and the bridge built: An important difference between the design of a new structure and the assessment of an existing structure is the amount of information available, which in the latter usually allows lower uncertainties.

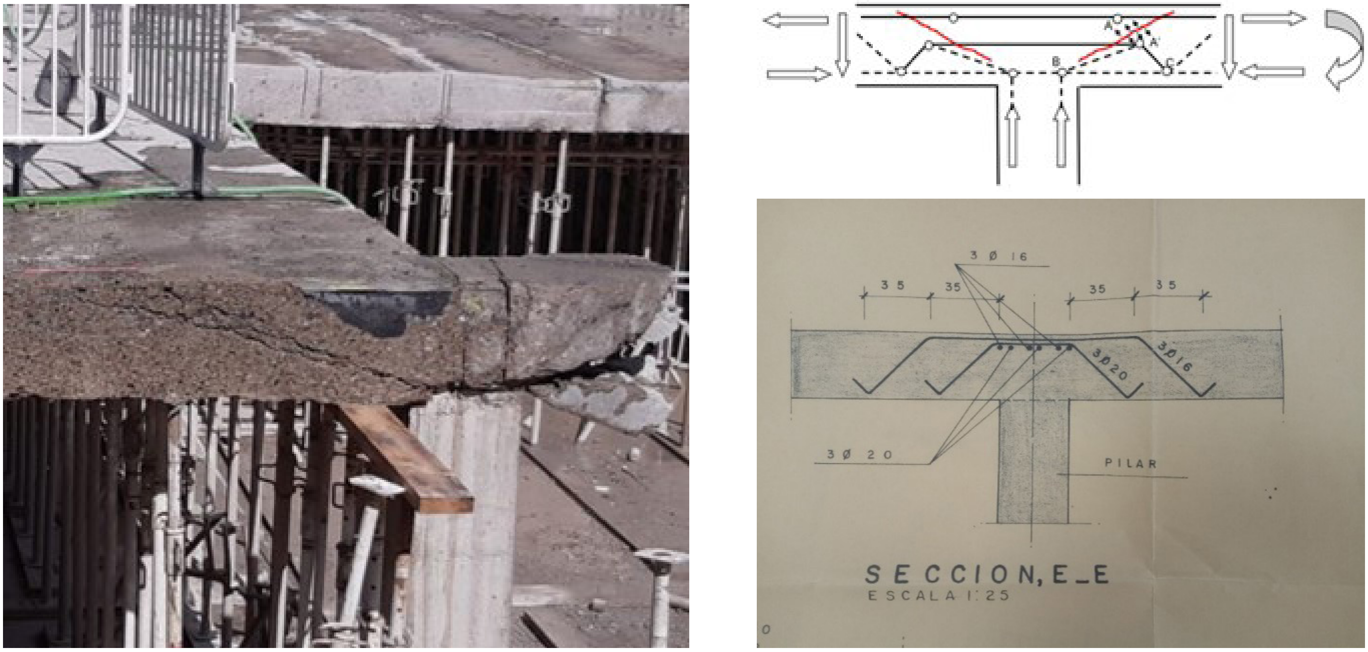

Figure 2. Example of punching failure in a slab due to a mistake in the placement of the specified reinforcement.

$$E_a \leq R_a \tag{1}$$

where

E_a is the assessment value of the effect of actions.

R_a is the assessment value of the resistance

NOTE E_a and R_a can be expressed as functions of the assessment values of the basic variables X_a (including the relevant partial factors, combination factors and conversion factors) as in formula (2) and (3)).

$$E_a = E\,\{X_{a1}, X_{a2}, X_{a3},\ldots., X_{aj}\} \tag{2}$$

$$R_a = R\,\{X_{a1}, X_{a2}, X_{a3},\ldots., X_{aj}\} \tag{3}$$

With regard to the aforementioned partial factors, it is known (and explicitly stated in Eurocode 0, [1]) that they are set based on a certain *probability of failure* (normally by means of a reliability index, β) which is considered acceptable in terms of the consequences of such a failure (i.e. in terms of the accepted risk) over a given reference period. They also cover the uncertainties associated with the materials and actions, as well as those inherent to the resistance models and the effects of the actions included in the codes. As an example, for the design of new "conventional" structures within the framework of the Eurocodes (consequence class CC2), β=3.8 is generally considered for a reference period of 50 years and is associated with a service life failure probability of approximately 0.01%. In principle, this β-coefficient can be reduced in the case of a structure that has already been built by updating the safety coefficients; e.g. using the tools provided by Eurocode 0 [1] regarding reliability management.

However, it is particularly important to bear in mind that existing structures may have been designed and built with materials, techniques and construction specifications that are very different to those covered by recent codes. Engineers must thus be particularly attentive to the validity of the verification models and their underlying assumptions. Aspects such as construction quality, the ductility of materials, the robustness of the structures, durability, etc., did not have the importance that they now do, given the evolution of knowledge and consequently of regulatory developments. Therefore, it requires a high degree of caution and experience to update the partial factors for assessment of existing structures.

In addition, other highly important aspects, especially for the assessment of existing structures, are not taken into account by means of these safety factors. They do not cover "*human error*", which for designs are minimised through control activities (during design and construction phases). As the example in Figure 2, a defect in the placement of the reinforcement steel (the transverse reinforcement was too low and therefore not effective) caused the failure of the slab, which fortunately was detected and corrected despite the low warning capacity of punching shear failure (in this case, by demolishing and rebuilding the slab).

Therefore, a decisive aspect in the assessment of an existing structure is its investigation of its current condition. As specified in *Annex A of prEN1990-2* [2]: *Guidance relating to the assessment process*, the investigation will depend on the type of structure (construction typology, period, etc.) and the information available, and should be carried out with the aim of updating knowledge about the structure, verifying its adequacy with the information available, and complementing this information with respect to aspects that may be incomplete for the appropriate analyses.

Furthermore, *prEN1990-2* [2] supposes that such an investigation will be carried out by experienced and qualified personnel who are aware of the particular aspects that an assessment of each structure would entail.

Qualifications (experience and expertise) are essential to properly plan the investigation, whereby the configuration of the structure to be assessed and its most likely failure modes are identified, followed by planning the appropriate on-site and office-based investigations. The quality of the results of the latter depends directly on the validity of the initial assumptions, which must be based on the structure itself. While for the analysis step, both the procedures and the calculation criteria are generally provided for in the standards, and the existing ones are generally applicable to design, these criteria are always qualitative in on-site investigations and are thus entrusted to the expert performing the assessment. Apart from some noteworthy attempts, neither there are references nor the standards included how to evaluate the influence of this information in the assessment. As an example, Section 3 of Eurocode 8, *UNE-EN 1998-3* [5] propose specific values for the frequency of the investigation according to different knowledge levels (KL), which are not usually practicable, and penalising the capacity of the structure by means of confidence coefficients (CF) when this investigation is not "complete". Anyway assessment for seismic loads is not totally comparable to an assessment for static loads in ULS, and the knowledge level approach for ULS static design is still a challenge to be met.

It is nonetheless highlighted that the future Eurocode, in particular with *prEN 1990-2* [2]. Basis of assessment and retrofitting of existing structures: general rules and actions (with the format and/or location that is ultimately decided), together with *Annex I* of *FprEN1992-1-1*, are a first and great step towards assessment regulations, being one of the first design codes including a part on assessment and providing important tools for this type of concrete structure analysis.

3.
DESIGN STRENGTH OF CONCRETE

One of the key aspects in the evaluation of a concrete structure is the estimation of the concrete strength class, which is determined by measuring the concrete compressive strength.

The future Eurocode 2 [4] provides formulation to the characteristic strength of the concrete based on results obtained from cores, while allowing the engineer a more active role with respect to the uncertainties that have to be taken into account when determining the partial factor for concrete. To this end, Annex A outlines the different issues involved in obtaining these partial factors, while Annex I covers particular aspects of an assessment of existing structures.

Based on the formulation stated in the main text of *FprEN1992-1-1* (5.1.6) and the contents of *Annex A*, it is worth noting the factoring of concrete strength as the result of different log-normal distributions, which in the case of compression and bending are: that of the material itself on site (f_c), the effects of geometry (A_c), the effects on the strength model (θ) and that which takes into account casting of the concrete (η_{is}). This last coefficient allows the conversion between the resistance in the control tests and the resistance in the element, $f_{c,is}$. For each of them, the standard itself proposes bias coefficients, μ (or bias, i.e. ratio between the mean value and the characteristic value) and a variation coefficient, V (Table 1, which corresponds to Table A.3 of *prEN 1992-1-1* [4]).

TABLE 1.
Statistical data assumed for the calculation of partial factor defined in FprEN1992-1. This table corresponds to Table A.3 of FprEN 1992-1-1 [4].

	Coefficient of variation	Bias factor a
Partial factor for reinforcement γ_S		
Yield strength f_y	$V_{fy} = 0.045$	$f_{ym}/f_{yk} = \exp(1.645V_{fy})$
Effective depth d	$V_d = 0.050$ [b]	$\mu_d = 0.95$ [b]
Model uncertainty	$V_{\theta s} = 0.045$ [c]	$\mu_{\theta s} = 1.09$ [c]
Coefficient of variation and bias factor of resistance for reinforcement	$V_{Rs} = 0.081$ [i]	$\mu_{Rs} = 1.115$ [i]
Partial factor for concrete γ_C		
Compressive strength f_c (control specimen)	$V_{fc} = 0.100$	$f_{cm}/f_{ck} = \exp(1.645V_{fc})$ [d]
In-situ factor $\eta_{is} = f_{c,ais}/f_c$ [e]	$V\eta_{is} = 0.120$	$\mu\eta_{is} = 0.95$
Concrete area A_c	$V_{Ac} = 0.040$	$\mu_{Ac} = 1.00$
Model uncertainty	$V_{\theta c} = 0.070$ [f]	$\mu_{\theta c} = 1.02$ [f]
Coefficient of variation and bias factor of resistance for concrete	$V_{RC} = 0.176$ [i]	$\mu_{RC} = 1.142$ [i]
Partial factor for shear and punching γ_V (see 8.2.2, 8.4, I.8.3.1, I.8.5)		
Compressive strength f_c (control specimen)	$V_{fc} = 0.100$	$f_{cm}/f_{ck} = \exp(1.645V_{fc})$ [d]
In-situ factor $\eta_{is} = f_{c,ais}/f_c$ [e]	$V\eta_{is} = 0.120$	$\mu\eta_{is} = 0.95$
Effective depth d	$V_d = 0.050$ b	$\mu_d = 0.95$ [b]
Model uncertainty	$V_{\theta V} = 0.107$ g	$\mu_{\theta v} = 1.10$ [g]
Residual uncertainties	$V_{res,v} = 0.046$ h	–
Coefficient of variation and bias factor of resistance for shear and punching (members without shear reinforcement)	$V_{RV} = 0.137$ i	$\mu_{RV} = 1.085$ [i]

a The values in this column refer to ratio between mean value and values used in the design formulae (characteristic or nominal).
b These values are valid for d = 200 mm. For other effective depths: $V_d = 0.05(200/d)^{2/3}$ and $\mu_d = 1 - 0.05(200/d)^{2/3}$.
c The partial factor γ_S is calibrated for the case of pure bending according to 5.2.4 and 8.1.
d This formula replaces relationship given in Table 5.1 for the purpose of Annex A.
e In-situ factor η_{is} accounts for the difference between the actual *in-situ* concrete strength in the structure $f_{c,ais}$ and the strength of the control specimen f_c. For strength $f_{c,is}$ assessed on extracted 2:1 cores according to EN 13791, see (7).
f The partial factor γ_C is calibrated for the case of axial compression according to 5.1.6 and 8.1.
g The partial factor γ_V is calibrated for the case of punching according to 8.4 and applies also for the case of shear without shear reinforcement according to 8.2.2 (similar statistical values).
h The residual uncertainties refer to aggregate size, reinforcement area and spacing and column size.
i Based on the statistical values above and calculated using Formulae (A.2) and (A.3).

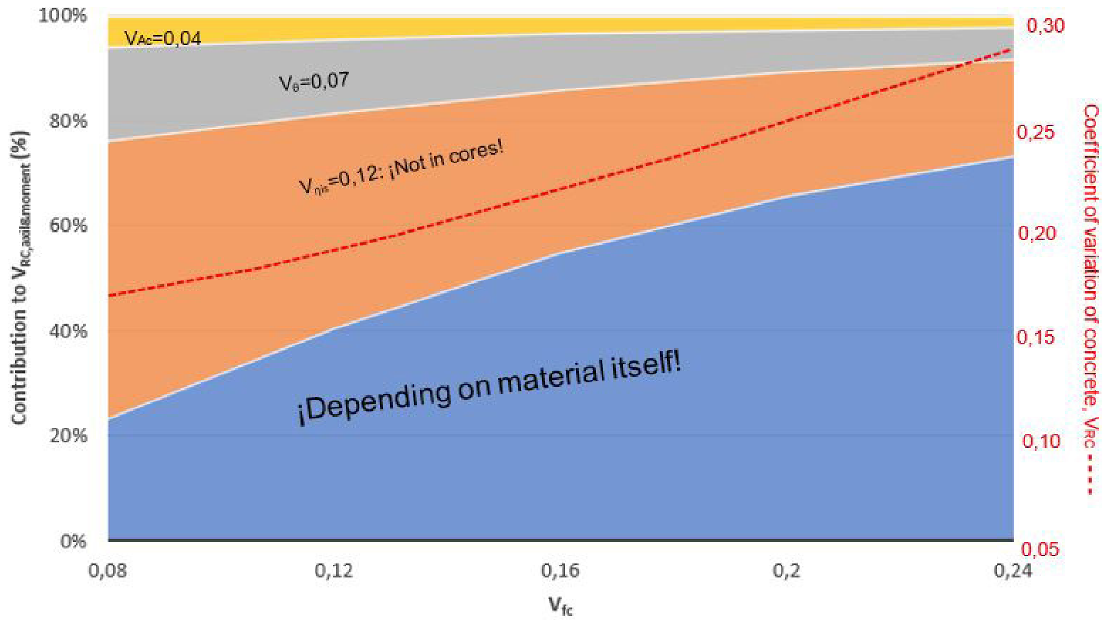

Figure 3. Contribution (in %) of different issues to total coefficient of variation of concrete, V_{RC}. V_{RC} obtained as function of the coefficient of variation of the material, V_{fc} (in red). V_{Ac} = 0.04, V_{θ} = 0.07 and $V_{\eta is}$ = 0.12 are considered.

If the influence of these parameters in the design of a new structure are represented, the relationship shown in Figure 3 will be obtained as an expression of the coefficient of variation of concrete. Note the highly important influence of Eurocode 2 with respect to casting, a factor that logically disappears when strength is assessed on the basis of core sample tests, as the standard itself states below. Effects of casting are included in *Annex I* as part of $k_{\mu fc}$ to obtain the characteristic strength, f_{ck} (see Table 3 further on).

The material's adjusted partial factor is thus obtained as shown below:

$$\Upsilon_M = \frac{e^{\alpha_R \beta_{tgt} V_{RM}}}{\mu_{RM}} \tag{4}$$

where

index M is S for reinforcement, C for concrete in compression, and V for shear;

α_R is the sensitivity factor for resistance (0,8 according to Table A.3 of *FprEN1992-1-1*);

β_{tgt} is the target value of the reliability index for the remaining service life (for example, 50 years) and taken into account the design situation (persistent or transient, fatigue or accidental)

V_{RM} is the coefficient of variation of the resistance which may be calculated from:

$$V_{RC} = \sqrt{V_{fc}^2 + V_{\eta is}^2 + V_{AC}^2 + V_{\theta C}^2} \tag{5}$$

$$V_{RC} = \sqrt{\left(\frac{V_{fc}}{3}\right)^2 + \left(\frac{V_{\eta is}}{3}\right)^2 + V_d^2 + V_{\theta v}^2 + V_{res\,v}^2} \tag{6}$$

where the coefficients of variation of each uncertainty are defined in Table 1 (Table A.3 of *prEN 1992-1-1* [4]), as mentioned before, or updated.

μ_{RM} is the bias factor of the resistance and may be calculated from:

$$\mu_{RS} = \frac{f_{ym}}{f_{yk}} \mu_d \, \mu_{\theta S} \tag{7}$$

$$\mu_{RC} = \frac{f_{cm}}{f_{ck}} \mu_{\eta is} \, \mu_{Ac} \, \mu_{\theta c} \tag{8}$$

$$\mu_{RV} = \left(\frac{f_{cm}}{f_{ck}} \mu_{\eta is}\right)^{1/3} \mu_d \, \mu_{\theta v} \tag{9}$$

where the bias factors of each uncertainty are defined in Table 1 (Table A.3 of *prEN 1992-1-1* [4]) or updated.

In *Annex A*, Item 7, it is specified that in the assessment of existing structures based on the results of core sample tests, the intervention of the η_{is} factor is not considered. The reason is that f_c,is is of interest for calculation purposes and is being obtained directly from the core sample tests, while the coefficient of variation and the bias factor are corrected to consider uncertainties inherent to statistical inference. As mentioned before, effects of coring and casting are included in Annex I as part of $k_{\mu fc}$ (see Table 3, corresponding to Table I.2 in *Annex I*). In this case where compressive concrete strength is assessed according to *EN 13791: 2019* [6], Clause 8, to obtained the adjusted partial safety factor γ_c, formulae (5) to (9) should be replaced by:

$$V_{RC} = \sqrt{V_{fc\,is\,corr}^2 + V_{AC}^2 + V_{\theta C}^2} \tag{10}$$

$$V_{RC} = \sqrt{\left(\frac{V_{fc\,is\,corr}}{3}\right)^2 + V_d^2 + V_{\theta v}^2 + V_{res\,v}^2} \tag{11}$$

$$\mu_{RC} = \mu_{fcis} \, \mu_{AC} \, \mu_{\theta c} \tag{12}$$

$$\mu_{RV} = \mu_{fcis}^{1/3} \mu_d \, \mu_{\theta v} \tag{13}$$

where V_{Ac}, $V_{\theta c}$, μ_{Ac}, $\mu_{\theta c}$ are taken from Table A.3 of *prEN 1992-1-1* [4] or updated, and $V_{fc.is.corr}$ and $\mu_{fc.is}$, are defined as:

$$V_{fc\ is\ corr} = \frac{k_{dn}}{\alpha_R\, \beta_{tgt}} V_{fc\ is} \quad (14)$$

$$\mu_{fcis} = e^{k_n V_{fc\ is}} \quad (15)$$

where:

$k_{d.n}$ is a parameter which depends on the number of samples, according to Table 2.

$V_{fc.is}$ is the coefficient of variation of the core strength according to *EN 13791: 2019* [6], but not smaller than 0,08.

k_n is the parameter which depends on the number of samples and has been used to calculate f_{ck}.is according to *EN 13791: 2019* [6]. See also Table 2.

TABLE 2.
Values of k_n and $k_{d.n}$ as function of the number of test results *n* used to evaluate the *in-situ* concrete compressive strength in the test region. This table corresponds to Table A.5 of *FprEN 1992-1-1* [4].

n	8	10	12	16	20	30	∞
k_n	2.00	1.92	1.87	1.81	1.76	1.73	1.645
$k_{d,n}$(for $\alpha_R\ \beta_{tgt}$=3.04)	5.07	4.51	4.19	3.85	3.64	3.44	3.04

This approach is consistent with *EN 13791: 2019* [6]. This is because the uncertainty associated with inferring the strength of the population from sampling as above is finally normalised, which when the dispersion of the concrete population is unknown (3rd row of Table 2) is done by means of Student's t-distribution with n-1 degrees of freedom. This formulation is limited to the coefficients resulting from using a ß=3.8 reliability index, though this assumption is not explicitly stated in the standard.

Annex I proposes the use of clause (8) of the aforementioned *EN 13791: 2019* [6] for the determination of the characteristic value of the in-situ compressive strength, $f_{ck,is}$, from cores. It points out that this strength must be corrected to obtain the characteristic strength, f_{ck} (to which the entire formulation of the articles including the strength of the concrete refers), by dividing it by a coefficient $k_{\mu fc}$ (equation (16)), which is always less than 1 (Table 3):

$$f_{ck} = \frac{f_{ck\ is}}{k_{\mu\, fc}} \quad (16)$$

TABLE 3.
Parameter $k_{\mu fc}$ considering the representativeness of the in-situ compressive concrete strength assessed according to *EN 13791: 2019* [6], clause 8. This table corresponds to Table I.2 of *prEN 1992-1-1* [4].

Regions and conditions of the structural member where the cores are extracted[a]	$k_{\mu fc}$
a) Cores extracted only from the bottom parts of the concrete masses during casting (lower 70 % of the depth of concrete during casting) not necessarily representing the governing region for the verification	0,95
b) Cores extracted from different regions representing all conditions in the structural member, but not necessarily representing the governing region for the verification	0,90
c) Cores extracted from the region governing for the verification	0,85
NOTE The parameters given in Table I.2(NDP) apply unless a National Annex gives different values.	
[a] The in-situ concrete compressive strength can exhibit significant variations depending on the location (strength typically smaller in the upper part of the element during casting)	

This factor $k_{\mu fc}$ is the link between in situ measured characteristic strength and the characteristic strength to be used for design. It is an important contribution of *Annex I*, since it is not covered in *EN13791* and *EN1992-1-1*. It incorporates both the effect of the "damage" in the extraction ($\eta_{core\text{-}actual}$) and the effect of the casting (η_{is}). Specifically, it assigns an average value of 0.95 to the first of these, similar to that considered by the ACI (6%) [8] and somewhat more distinct to the 0.9 of the still recent EHE-08 [9] (currently replaced by *Código Estructural*, where in its art. 57.8 maintains the 10% difference between the concrete before placing and concrete in the cores, but including the effects of placing, as $k_{\mu fc}$ does); in the second case, the parameters are contained in prEN 1992-1-1 [4].

Lastly, Section I.5.2.2. *Assessment assumptions* specifies the value of coefficients that influence the structural strength but depend on phenomena not directly related to concrete testing: one that accounts for the effect of brittle failure at non-uniform stress distribution in concrete, ηcc (not considered in current Eurocode), which in existing structures will normally have no influence (η_{cc}=1 for f_{ck} < 40 MPa [11]); and the other, ktc, which considers the combination of the favourable effect of concrete strength gain over time due to hydration of the cement paste together with the reduction of concrete strength to take account the effects of sustained loads . With respect to the latter (already specified in current EN1922-1-1 as α_{cc}, though with different values), it is assumed that there is a "certain reduction" only when the overloads represent less than 20% of the total load [12]. For example, k_{tc}=0.85 in the limit case where permanent action (and/or variable actions of duration> 1 hour) represents 100 % of the total effect at assessment level. These values are consistent with those reported in the JCSS Probabilistic Model Code [13], where these effects are taken into account by means of $\alpha(t, \tau) = \alpha(t) \times \alpha(\tau)$.

Though not commented in this paper, in *Annex A* and *I* of *FprEN1992-1-1* steel guidance are also given to update the partial factors.

In summary, *FprEN1992* [4] allows the designer a greater level of intervention in the partial factors to be considered for the resistance of the materials, as it includes a formulation and coefficients which, in the case of that obtained in concrete from the extraction of cores, are aligned with those used in other standards.

4. VERIFICATION OF EXISTING CONCRETE STRUCTURES

Section I.8 of *Annex A* includes some particular aspects of the ULS verifications that may have to be taken into account in the assessment of an existing concrete structure, while I.9 covers those corresponding to the SLS. Following the numbering of the chapters in the main part of the code, section I.11 contains additional rules for reinforcement types (plain bars) and/or detailing of ribbed bars that do not meet modern code requirements.

4.1. Shear and punching

In *Annex I*, apart from some considerations of the effects of reinforcement corrosion which covered in the following sec-

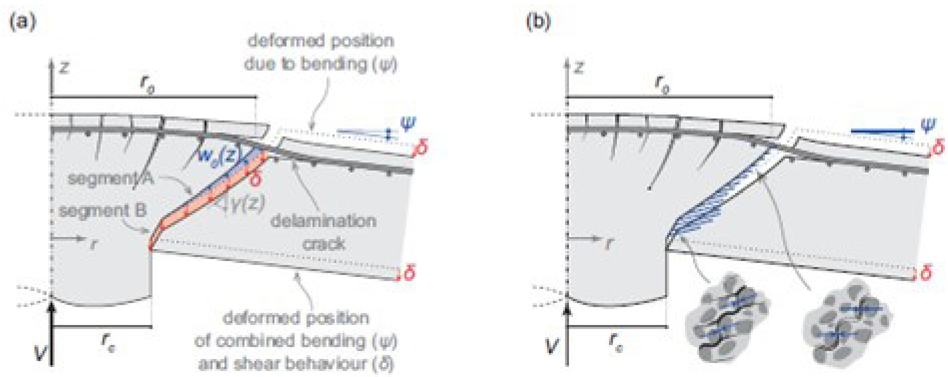

Figure 4. Theoretical principles of the mechanical model of Critical Shear Crack Theory: (a) kinematics of the critical shear crack at failure and resulting (b) internal stresses (extracted from the background document of *FprEN1992-1* [7]).

tion, probably the most relevant clauses of this Annex are those related to the shear and punching resistance. The formulation proposed is based on the *Critical Shear Crack Theory* (CSCT) (Figure 4) [7], both in Annex I and in the main text of *FprEN1992-1-1*.

Treatment of concrete members without shear reinforcement varies widely across different national and international standards: For the *Critical Shear Crack Theory* (CSCT), a shear failure criterion depending on the actual strain in the longitudinal reinforcement is considered, so that the strain ε_v must be determined.

According to Muttoni and Fernández Ruiz included in [7] for the strain in the reinforcement, ε_v, a linear relationship between the acting moment and the strain in the reinforcement can be assumed. ε_v can thus be calculated as:

$$\varepsilon_v = \frac{M_E}{z\, A_s\, E_s} = \frac{M_E}{z\, \rho_l\, b_w\, d\, E_s} = \frac{V_E\, a_{cs}}{z\, \rho_l\, b_w\, d\, E_s} \qquad (17)$$

where

M_E	is the acting bending moment at the control section;
V_E	is the acting shear force at the control section;
$a_{cs}= \lvert M_E / V_E \rvert$	is the effective shear span at the control section;
A_s	is the area of the longitudinal reinforcement;
ρ_l	is the longitudinal reinforcement ratio;
z	is the effective level arm of the longitudinal internal forces;
E_s	is the modulus of elasticity of the longitudinal reinforcement.

Regarding the failure criterion, the original formulation used a hyperbolic failure [7].

In the case of shear, in *I.8.3. Shear,* the new formulation for elements without shear reinforcement is derived from that failure criterion. The shear stress resistance is computed as:

$$\tau_{Rd,c} = 0.33\, \frac{\gamma_{def}^{2/3}}{\gamma_V^2}\, \frac{\sqrt{f_{ck}}}{1+24\, \gamma_{def}\, \varepsilon_v \frac{d}{d_{dg}}} \qquad (18)$$

where

d is the effective depth.

ε_v is the strain in the longitudinal reinforcement at control section. For planar members, it refers to the principal direction of the shear force, a non-linear cross-sectional analysis of the structure may be performed and the obtained internal forces as well as the strain ε_v may be averaged over the same width.

γ_{def} is a partial safety factor which covers the uncertainties related to the calculation of the deformation

NOTE

γ_{def} = 1.33 unless a National Annex gives a different value.

The proposal includes, as does the more simplified formulation in the main text in *FprEN1992*, the following:

- The consideration of a specific concrete reduction coefficient for the tangential stress verification, γ_V. This differentiation is in accordance with the dependence of concrete strength against shear stresses with the cube root of compressive strength. As γ_V is smaller than γ_C, this would imply lower uncertainties regarding the concrete's shear capacity.
- Consideration of the influence of the aggregate's size.

Both factors mean that the formulation generally produces results that are somewhat higher than those of the current Eurocode 2, except in cases of very small aggregate sizes (10 mm).

But as can be seen in formula (17), the formulation proposed in *Annex I* also takes into account the deformation of the tensioned reinforcement in the design section and not only with respect to the amount, as simplistically considered in most standards. This consideration is in line with the *Model Code 2010* [14] and may be of particular importance when elements present relatively minor bending (end columns of floor joists, shear verifications at the exit of abacus in reticular slabs, etc.) or in relatively high overdesigned elements (changes of use with overload reductions).

The importance of the effect of the deformation of the tensioned reinforcement can be seen in the graphs in Figure 5, which shows the shear resistance as a function of the effective shear span (a_{cs} = M_d / V_d) according to different formulations from current Eurocode 2 *-EN1992-1-1:2004-*; from the main text *-FprEN 1992-1-1-*; *Annex I;* and MC-2010. In all cases, a characteristic concrete strength f_{ck}=25 MPa, a effective depth d=0.27 m and a maximum aggregate size of d_{dg} = 36 mm were

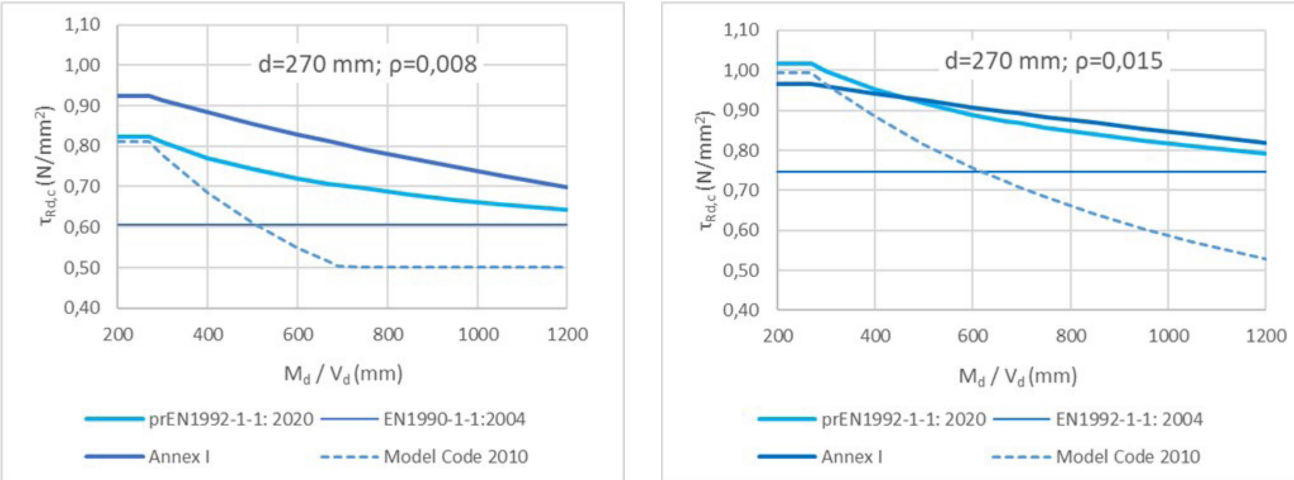

Figure 5. Effect of the effective shear span, $a_{cs} = M_d/V_d$, in concrete shear resistance, τ_{Rd}, with different formulations for a effective depth d = 0,27 m.

considered. Regarding the partial factors, γ_V=1.4 is considered for prEN formulations, and γ_C=1.5 in the case of current Eurocode 2 and *MC-2010*. The results for two reinforcement ratios are shown, low (ρ_1=0.008) and high (ρ_2=0.015).

As shown in Figure 5, the proposed formulation has been found to be quite more favourable than the one in the current Eurocode. Furthermore, it is consistent with that of the Model Code (especially in the case of reduced effective shear span), although it gives substantially higher values for significant bending.

It is also worth noting the good approximation of the formulation of the main text *-FprEN 1992-1-1-* to the proposal in *Annex I*, derived from the original failure criterion, though the first one is the result of a simplification in that hyperbolic failure criterion, as explained hereafter.

Indeed, the formulation of *Annex I* implies that calculating the actual resistance, V_{Rd}, requires solving the set of formulae (17) and (18), which can be easily done iteratively, but it is not convenient for design.

For design purposes a closed-form expression is preferred. That is why *FprEN1992-1-1* involves a simplification, using a parabolic curve instead of the hyperbolic one, which makes it possible to clear the resistant shear, τ_{Rd}, as a function of the effective shear span, a_{cs}. This results in a formulation similar to that currently used in *EN1992-1-1:2004* [15], *(8.27)* in the main text of *FprEN 1992-1-1*, although instead of depending on the effective depth of the section, *d*, it depends on the aforementioned a_{cs} (clause (3) of 8.2.1):

$$\tau_{Rdc} = \frac{0.66}{\gamma_V}\left(100\,\rho_l\,f_{ck}\,\frac{d_{dg}}{a_v}\right)^{1/3} \geq \tau_{Rdc.min} = \frac{11}{\gamma_V}\sqrt{\frac{f_{ck}}{f_{yd}}\,\frac{d_{dg}}{d}} \tag{19}$$

a_v is the mechanical shear span

$$a_v = \sqrt{\frac{a_{cs}}{4}d} \tag{20}$$

a_{cs} is the effective shear span.

$$a_{cs} = \left|\frac{M_{Ed}}{V_{Ed}}\right| \geq d \tag{21}$$

where:

As a first approximation and on the safety side, $a_{cs}=4d$ is considered, so that $a_v=d$. In addition, a minimum shear resistance is given (*(8.20)* in the main text), which is obtained considering that the member reaches yielding of the flexural reinforcement and the shear resistance at the same load level.

Regarding punching, CSCT theory leads to a formulation in the articles which is completely different than the prior procedures in *EN1992-1-1:2004* [15]. This entails changes even in the position of the verification section, which is now located *0.5d* from the front of the column.

As for shear, the strains in the reinforcement are considered by a parameter a_p, obtained from the distances between the column axis and the locations where the bending moments in both directions are equal to cero. As a first approximation, a safe bound of $a_p = 8d$ is considered in the formulation from the main text (8.4.3), though it can be easily calculated (for regular slabs, a_{pi} may be approximated as $0.22L_i$ (where *i* refers to *x* and *y* axes); an elastic -uncracked- model is also proposed in the main text to obtain a_p).

According to *Annex I*, that formulation from the main text (8.4.3) can be used for assessment of existing structures, where it can also be considered the favourable effect of compressive membrane action around internal columns without significant opening, inserts or slab edges at a distance less than $5d_v$ from the control perimeter $b_{0,5}$, multiplying parameter a_p in formula by the following enhancement factor:

$$\eta_{pm} = \left(\frac{h}{d}\right)^{\sqrt{2}}\left(1.2\,\frac{\sqrt{f_{ck}}}{\rho_l\,f_{yk}}\right)^{1/4} \geq 1 \tag{22}$$

Alternative, as for shear, the general method from CSCT theory is allowed in *Annex I*, both in terms of failure criterion and general definition of the load-rotation relationship. The shear stress resistance for punching is computed as:

$$\tau_{Rdc} = 0.75\,\frac{\gamma_{def}^{2/3}}{\gamma_V^2}\,\frac{\sqrt{f_{ck}}}{1+15\,\gamma_{def}\,\psi\,\dfrac{d_v}{d_{dg}}} \tag{23}$$

where

ψ in radians is the maximum rotation of the slab around the supported area. It may be calculated based on non-linear analysis of the structure and accounting for cracking, tension-stiffening effects, yielding of the reinforcement membrane action and other non-linear effects relevant for providing an accurate assessment of the structure. The governing value of ψ is the maximum relative rotation between centre of the supporting area and a distance $2d_v$ from the control perimeter.

d_v shear-resisting effective depth (potentially differing from the effective depth d to account for the penetration of the support and thus reducing the depth available to carry shear).

γ_{def} is a partial safety factor which covers the uncertainties related to the calculation of the deformation.

NOTE γ_{def} = 1.33 unless a National Annex gives a different value.

Though Eq (23) ((*I.17)* in *Annex I*) requires in general an iterative procedure to obtain the intersection between the hyperbolic curve of failure criterion and the load-rotation relationship, it produces more favourable results for small rotations ψ, as in the case of shear. In addition, for unusual geometries or reinforcement layout, a suitable load-rotation relationship can be obtained by non-linear analysis, taking into account slab continuity and membrane action, leading to a stiffer response of the slab, and then, higher punching resistance.

Regarding to slabs with punching shear reinforcement, some specifications are included that take into account the differences in the reinforcement details of existing structures in relation to the specifications in *FprEN1992-1-1* [4].

4.2. Serviceability Limit States

I.9 *Serviceability Limit States* indicates that in most cases SLS verifications may be performed using site-based observations and or measurements, instead of by calculations.

It is also noteworthy that when a reliability index lower than that usually considered for design purposes (according to *prEN 1990* [1]) is accepted in the ULS verifications, the stresses in the concrete and reinforcements under service loads (characteristic) must be within the values shown in Table 4.

TABLE 4.
Limits on reinforcement and the concrete stresses at the characteristic combination of actions. This table corresponds to Table I.6 of *FprEN 1992-1-1* [4].

$\sigma_c \leq 0{,}60 f_{ck}$	$\sigma_s \leq 0{,}80 f_{yk}$	$\sigma_p \leq 0{,}80 f_{pk}$

4.3. Anchorage of plain bars

The formulation in the articles of the main part of *FprEN1992-1* is restricted, as in the current Eurocode, to ribbed bars and tendons. However, for plain bars, reference is made to *Annex I*.

Before commenting on the formulation of the above-mentioned annex, it should be noted that the formulation of the main text in *FprEN1992* is already significantly different from the existing one, although it basically takes into account the same parameters (in particular the effect of the cover -c_d- and of the bond conditions conditions depending on the position of the bar; other effects, such as the confinement or the shape of the anchorage, are considered by modifying c_d or the anchorage length itself - reducing it by 15Φ in the case of hooks). In addition, the type of situation for which the anchorage is checked (persistent and transient or accidental in nature).

This change in formulation takes into account the results of recent research findings [16] and [17], which are reliability-based. In practice, it results in a significant increase in anchorage lengths in most instances compared to the past formulae. For example, anchorage of a 20 mm diameter ribbed bar of a 250 MPa material (in *FprEN 1992-1-1* is declared that due to the database used to calibrate the formulation, it is valid for tensile stress in the bar not greater than 300 MPa) with 30 mm cover, in good bond conditions, which under the prevailing Eurocode would require 429 mm of straight extension, entails an increase of almost 34%, to 575 mm using the new formulation. For larger cover dimensions, the difference is much lower. Comparing the new approach to the Spanish regulation, *Código Estructural* [10], which allow the formulae of previous national codes (EHE-08 [9]) under some conditions, is even higher (300 mm compared to 575 mm -in fact, this value almost corresponds to the basic anchorage length for a bar B500 with the described conditions, which results 600 mm-).

For plain bars, usually anchored by hooks, the situation is even more unfavourable:

- For example, the reduction in anchorage tension due to the effect of the hook in I.11.4.2 is somewhat less than is currently the case (0.7). This reduction of the contribution of the hooks is based in the latest test on plain bars, which have confirmed that the effect of hooks is not as relevant as expected.
- More significant than the small reduction due to the effect of the hook is the anchorage length that is normal in relatively old structures, where the reinforcement covers are usually similar to the bar diameter. For example, for a plain bar 20 mm in diameter with a 20 mm cover and good bond conditions, concrete C25 and 260 MPa of steel stress, the anchorage length is 1.477 mm. This value should be compared with that which would be obtained from the application of the 1990 Model Code, MC-90 [18] (the most recent version, from 2010, does not include a formulation in this regard), whose formulation expressed 600 mm for this case, already somewhat higher than that which would result from the application of Instruction EH-68 [19] at a national level, the last one which considered the anchorage of plain bars (470 mm in the example). In this regard, the following graphs, extracted from the background of Annex I of *FprEN1992-1* [7], show the significant increase in anchorage length using the proposed formulation compared with that of MC-90 [18], at least for normal cases of relatively smaller covers. In fact, for poor bond conditions the differences become huge.

It should also be kept in mind that previous standards (for example, EH-39 [20] in Spain) went so far as to consider the bar anchored from the end of the hook, while not considering

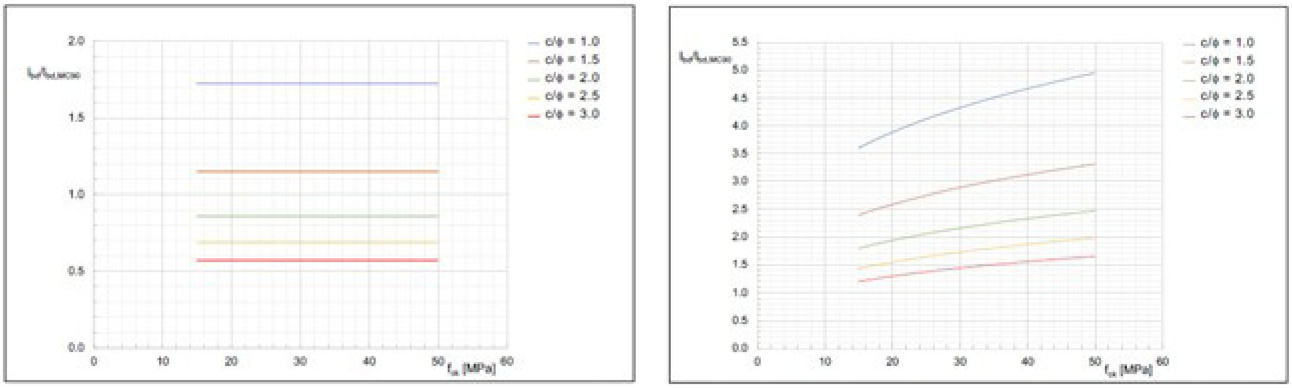

Figure 6. Influence of c/Φ on l_{bd_AnnexI} / l_{bd_MC90} for Good conditions position (GP, left) and Poor conditions position (PP, right), γ_c=1,5, σ_{sd} = 200 MPa (extracted from the background document of *FprEN1992-1* [7]) .

the offset effect of bending moment laws (this concept was not taken into account in design until the publication of the ACI code of 1963 and, in Spain, until the aforementioned Instruction of 1968 was issued).

Under these conditions, the anchorage penalty found in the *Annex I* proposal may be of great importance. Of course, if for the assessment tensile stress the actual anchorage length is lower than l_{bd} , it doesn't mean that the relevant bars should not be taken into account for the verification but that the assessment stress should be reduced accordingly in order to have the actual anchorage length not lower than l_{bd}. This means that all bars present in the structure should be taken into account in the verifications but assuming for them a maximum stress consistent with the actual anchorage length that should not be lower than l_{bd}.

This only confirms that the reinforcement details of structures built with plain steel components (generally up to the 1960s), particularly those relating to anchorage and reinforcement overlaps, call for very strict reinforcement lengths, while raising significant uncertainties about the use of such reinforcements (e.g. in columns), which must be carefully considered in the assessment.

These results contrast with historic performance of concrete structures broadly, where no anchorage failures have been recorded. The surface oxidation that these bars generally present probably improves bond. This fact, combined with low stresses to which the anchoring components are normally subjected (especially at the end columns of beams and slab ribs or at overlaps), as well as the contribution of the hook at least in the ultimate limit state and given the presence of significant bending due to crushing of the concrete, are in principle much higher than those allowed by the standards (including EH-68 [19]), could explain these differences.

Anyway, all of the foregoing means that, in the opinion of the authors, it is necessary to warn of the need for an adequate analysis of aspects such as the contribution of all the reinforcement elements in horizontal structures, or of the longitudinal reinforcement in columns. The new formulation proposed in *FprEN1992-2* is the most advanced one and it is consistent with the reliability-based approach of the Eurocodes reliability-based, so it could afford a good approximation.

5.
DURABILITY

Predictive methods for estimation of degradation processes in concrete are still under discussions and not generally accepted. This would lead to the situation that in engineering practice questions about remaining service life after presence of any deterioration or situations where, for instance, chloride fronts have almost reached the reinforcement, could not be predicted.

Then, neither *FprEN1992-1-1* [4] nor its Annex I lay out predictive methods for the estimation of the deterioration rates of an existing structure, and therefore of its residual service life, leading to the situation that for each specific case parties involved have to come to consensus on the required measures.

This is not a minor issue, as often in the structures under assessment -at least in Autor's experience, mostly in Spain- the carbonation front of the concrete has reached the position of the reinforcement (an example is shown in Figure 7, where the depth of the carbonate front was measured with a phenolphthalein solution, as is normally done) and its service life would be endangered. The Eurocode service life model considers two phases: the time to corrosion initiation, t_i (i.e. the time it takes for the attack front to reach the reinforcement), and the time to corrosion propagation leading to degradation, t_p (time to significant degradation of the structural element). For the example shown, the t_i has been consumed and since the latter depends mainly on the diameter of the reinforcement and the corrosion rate, measures must be taken to reduce it to the limits that allow the remaining service life to be admissible.

Figure 7. Example of a column where the carbonate concrete has reached the position of the rebars.

Figure 8. Different causes of deterioration of reinforced concrete: pitting corrosion (left), AAR (centre), acid attacks (right).

Under these conditions, decisions to try to comply with such standards may be excessive and possibly require the generalised protection of the structure. Such protection may be necessary in certain areas with unfavourable humidity conditions (e.g. damp rooms with an XC3 environment), but probably not in elements where the corrosion rate is very slow (e.g. in XC1 environments) and thus where the risk of structural and aesthetic consequences that may affect the structure's new service life are low.

The previous example highlights the differences between ULS and sustainability requirements. What actions are needed in each situation partly will depend on local building legislation. Fulfilling ULS requirements with sufficient confidence is generally what is stated in building legislation. Sustainability requirements are often less explicitly stated for existing construction, making it possible to distinguish between the different situations as described before.

Of concern are effects that must be considered in the assessment of structures deteriorated by durability defects (Section I.4.1.2): corrosion of reinforcements, sulphate attack (Delayed Ettringite Formation, DEF), Alkali-Aggregate reaction (AAR), acid attacks, etc. with some examples shown in Figure 8.

These effects can include:

- Reduction of the concrete section due to spalling.
- Reduction of the cross-section of the reinforcement
- Reduction of its ductility. In the case of pitting corrosion, it may be necessary to stop its use because of the difficulties in detecting pitting (as its effect is local and often not accompanied by other forms of corrosion; it may not manifest itself externally), and because of the concentration of stresses around the pitting.
- Stress concentration due to localised corrosion (e.g. in prestressed steel).
- Stress corrosion cracking (e.g. in prestressed steel).
- Reduction of the bond between the reinforcement and the concrete.
- The loss of the concrete's properties (e.g. concrete's elastic modulus due to AAR).
- The loss of reinforcement properties (ductility from pitting corrosion) in relation to those deduced from the formulation of the articles.
- Cracking and expansion of concrete (e.g. due to DEF or AAR).

In addition, deterioration in the structure may influence the uncertainties of the strength models or the geometry itself. These aspects need to be taken into account when updating the safety coefficients in a semi-probabilistic analysis.

To address reinforcement corrosion, the following considerations for testing purposes are proposed in I.8.1:

- An initial distinction is made between homogeneous and pitting corrosion. In relation to the former, the parameter P_x, *Corrosion Penetration Depth* is used. It is defined as the *loss in cross sectional radius of the bar due to homogeneous/uniform corrosion along the bar length; while pitting corrosion is defined as the form of localised corrosion that leads to the creation of cavities or crevices in the metal.*
- For reinforcement subjected to compression where the stirrups or ties are heavily corroded, reduced strength is possible due to the bars buckling prematurely.
- In shear-stressed elements, there is the possibility of premature failure of the stirrups due to corrosion (due to carbonation or pitting).
- For corrosion rates $P_x \geq 0.2\text{-}0.4$, cracks with an opening larger than 1 mm, or in the case of pitting corrosion:
  - A reduction of the maximum steel elongation is to be taken into account in the ULS verifications; as is a reduction of the concrete cross-section due to spalling of the cover.
  - A concentration of stresses in the pits.
- For homogeneous corrosion and low to medium corrosion rates ($P_x < 0.2\text{-}0.4$ or crack openings of less than 1 mm), it can be assumed that the stress-deformation diagram of the reinforcements is not affected and that the entire concrete section contributes to strength, although some reduced compressive strength due to cracking can be assumed.

In summary, though predictive methods for estimation of degradation processes in concrete are not included in a standard like Eurocode 2, the current draft of Annex I allows the assessment of structures without significant degradation.

6. CONCLUSIONS

In the previous sections, the most significant aspects of the assessment of existing concrete structures addressed in the future generation of Eurocodes have been presented. These are summarized as:

- *prEN1990-2* [2] sets out the criteria to be taken into account for the investigation and assessment of an existing structure and establishes differences with respect to the conventional design of new structures. In the opinion of the authors, particular importance should be given to knowledge of the structural configuration and most probable failure modes in the existing structure, as this establishes an essential basis for planning and carrying out verifications that are most appropriate and necessary in each case to quantify the safety, durability and functionality of the structure.
- *Annex I* of *FprEN1992* [4] complements these criteria for the specific case of concrete while emphasising the following aspects:
  - In relation to the materials, the procedure for obtaining the design strength of the concrete from the extraction of core samples is set out, which complements that of *Annex A*.
  - In relation to the shear and punching resistance of elements without transverse reinforcement, this *Annex I* proposes a formulation that in certain situations, like in the presence of elements with reduced reinforcement deformations, may allow for a greater contribution from concrete. This formulation, based on the *Critical Shear Crack Theory*, is also included in the main text, with some simplifications.
  - A particular aspect of existing concrete structures, and in particular those reinforced with plain steel bars, is the assessment of the anchorage conditions. In this regard, *Annex I* propose a formulation that, though is very advanced and consistent with the reliability-based approach of the Eurocodes, in the authors' view, should be applied with caution. The formulation reduces the strength contribution of the reinforcement in horizontal structures or contributions of longitudinal reinforcement in columns, which is why caution is recommended in its use and reliance on the expertise of technicians charged with conducting assessments: all bars present in the structure could be taken into account in the verifications but assuming for them a maximum stress consistent with the anchorage length given in the formulation proposed.
- When assessing existing structures, durability can be of particular importance. *Annex I* list some criteria that may guide considerations regarding structural deterioration. However, it does not propose a methodology for assessing the structure's remaining service life or measures that can be used to extend it, which are again left to the experience and knowledge of the technicians performing the assessments.

The future Eurocode will therefore standardise, at a European level, methods for the assessment of existing structures. *Annex I* is informative, so it depends on the adoption of individual countries as to whether this becomes the future standard in each one. This annex, together with *prEN 1990-2. Basis of assessment and retrofitting of existing structures: general rules and actions*, provide the basic principles of such assessments that are being developed in numerous forums. However, the proper application of this type of assessment requires knowledge of the historical context within each country with respect to prior building codes and construction practice, in addition to those currently under development, as shown in the previous sections.

It must be highlighted that these new frameworks are a great step forward since it would be one of the first international design codes to specifically address assessment. However, there is still a long way to go including aspects such as the treatment of deteriorated or damaged structures, the definition of predictive methods for estimation of degradation processes, the consideration of the level of knowledge in the assessment, etc., that need to be further analyzed for their practical implementation. Hopefully in the next version of *EN1992-1-1* or a future amendment of the *FprEN1992-1-1* continued advancements can be made.

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