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Hormigón y Acero 2023; 74(299-300):109-122
https://doi.org/10.33586/hya.2023.3100

Fatigue in Structural Concrete According to the New Eurocode 2

La fatiga en hormigón estructural según el nuevo Eurocódigo 2

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Recibido el 29 de agosto de 2022; aceptado el 20 de marzo de 2023

ABSTRACT

The new Eurocode 2 represents a significant advance in the treatment of fatigue in structural concrete, compared to the old Eurocode. Fatigue acquires greater relevance and visibility in the new standard, and the field of application of this limit state is extended.

This paper shows the most relevant changes in the fatigue chapter of the new Eurocode 2, in which there has been an important formal and/or conceptual change with respect to the old Eurocode 2. The first difference is that in the new Eurocode 2, fatigue has its own chapter and annex, which shows how important this phenomenon has become in recent years. On the other hand, the new proposed fatigue formulation significantly improves the mechanical capacity of the material, which allows an optimisation of those concrete structures in which fatigue is a critical phenomenon..

KEYWORDS: Fatigue, structural concrete, S-N curves, Palmgren Miner rule, bridges.

RESUMEN

El nuevo Eurocódigo 2 supone un avance significativo en el tratamiento de la fatiga en hormigón estructural, en comparación con el antiguo Eurocódigo. La fatiga adquiere una mayor relevancia y visibilidad en la nueva norma, y se amplía el campo de aplicación de este Estado Límite.

Este artículo muestra los cambios más relevantes del capítulo de fatiga en el nuevo Eurocódigo 2, en los que se ha producido un importante cambio, formal y/o conceptual, respecto al antiguo Eurocódigo 2. La primera diferencia es que en el nuevo Eurocódigo 2, la fatiga tiene su propio capítulo y anexo, lo que muestra la importancia que este fenómento ha adquirido en los últimos años.

Por otra parte, la nueva formulación de fatiga propuesta mejora de forma significativa la capacidad mecánica del material, lo que permite una optimización de aquellas estructuras de hormigón en las que la fatiga sea un fenómeno crítico.

PALABRAS CLAVE: Fatiga, hormigón estructural, curvas S-N, regla de Palmgren Miner, puentes.

1. INTRODUCTION

From the 1990s to today, many things have changed in the world and in Europe; in all areas: social, cultural, economic, environmental, etc., as well as scientific and technological. The world of concrete and structures has been no exception, and in the last 30 years there have been significant advances in many fields. These include, for example, the development of the High-Speed Railway network that has been carried out in Spain (with the construction of some 3,000 km of new lines during this time) and throughout Europe, with the help of European Funds. Also noteworthy is the development of wind energy in Spain (with a total of approximately 21,500 wind turbines installed in almost 1,300 wind farms) and worldwide. The vast majority of wind turbines are steel towers, but in recent years different concrete-based solutions have been appearing due to the exigent dynamic requirements of the wind

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How to cite this article: Ríos, C., Lancha, J.C., & Vicente, M.Á. (2023) Fatigue in Structural Concrete According to the New Eurocode 2, *Hormigón y Acero* 74(299-300):109-122, https://doi.org/10.33586/hya.2023.3100

turbines, and larger turbines will lead to a more extensive use of the concrete tower as the standard solution.

However, one thing has not changed in recent years, and that is Eurocode 2, which remains broadly the same structure as that document published at the beginning of the 2000s.

In the field of structural fatigue, and more specifically in the field of concrete fatigue, the evolution in the last 30 years has been more than remarkable, both in the knowledge of the fatigue response of concrete (both in its mass concrete version and in its reinforced, prestressed, fibre-reinforced, etc.) and in the importance of fatigue as a structural design criterion. In this respect, the concrete towers of wind turbines are a good example. For these structural elements, the most restrictive limit state, the one that conditions their design, is fatigue. This is due, in part, to the fact that the international standards and recommendations that regulate it are highly conservative, which makes it less competitive than other structural solutions.

The Eurocode 2, EN 1992-1-1, [1], currently in force, dated 2004, was an important advance in the field of concrete fatigue, but it was based on the state of the art of the 1990s. 20 to 30 years of intense scientific and technological development have rendered it obsolete in certain aspects. A profound change was needed. The new version of Eurocode 2, in its Fatigue chapter, represents a more than remarkable update.

The first change that can be observed is that, in the case of the new version of Eurocode 2, FprEN1992-1-1:2023, [5], fatigue has its own chapter and its own annex, whereas in the equivalent document of 2004 [1], fatigue is included in clause 6, a clause dealing with every Ultimate Limit State. It is necessary to go to the Eurocode 2 part 2 [2] to find a section (not a chapter) and an annex dedicated to fatigue. At that time, end of 1990s, bridges were the only structures where fatigue could be considered as a relevant structural effect. For the rest of the structures, it was not usually taken into account.

The fact that in the new version of Eurocode 2 [5], fatigue in concrete occupies an entire chapter shows the importance that this limit state has acquired in recent years.

In the field of concrete fatigue, the new Eurocode 2 [5] follows a different line from the Model Code 2010 [3], a document that has been a reference in the world of structural concrete in many aspects, and also in fatigue. It also follows a different approach from the one presented in the technical document recently published by the American Concrete Association "ACI PRC-215-21" [4]. The new Eurocode 2 [5] includes a new formulation of the fatigue strength in compression, based on the new formulation introduced for the static compressive strength in Ultimate Limit States, ULS, which leads to remarkable increases of the fatigue strength of concrete in compression, especially for those concretes with strength class above C50, compared to the formulation of the still current version of Eurocode 2 [1].

The changes introduced by the new version of Eurocode 2 [5] in the formulation used to verify the fatigue strength of concrete make it possible to exploit the material's strength capacity between 10% and 20% more than the old formulation allowed, and this change will make it possible to reduce the volume of concrete structures subjected to wind by 5% to 10%.

This will give a decisive boost to the implementation of wind energy production facilities, both on-shore and off-shore, which will reduce the price of energy and simultaneously reduce energy dependence on the outside world.

Furthermore, the use of renewable energy sources helps to reduce the carbon footprint and, consequently, contributes to the fulfilment of one of the Sustainable Development Goals promulgated by the United Nations.

This paper presents, in detail, the most relevant aspects of the fatigue chapter of the new Eurocode 2 [5], in which a major change, formal and/or conceptual, has taken place with respect to the Eurocode 2 currently in force [1].

3.
CASES TO BE CONSIDERED

Fatigue is not a common concern in structures under predominantly static loads, such as standard buildings. On the other hand, most of live loads are always dynamic loads, even in case of building structures. Therefore, it could be possible to affirm that almost any structure is subjected to dynamic loads, being most of them cyclic loads.

However, depending on the number of cycles and the load amplitude or range of these loads, their impact on the structure may be negligible and therefore the verification of the ULS of fatigue not required. Concretely the new Eurocode, FprEN1992-1-1:2023, 10.1, [5], states: *"Structures and structural components subjected to significant numbers of repeated load or deformation induced significant stress cycles shall be verified to endure the expected cyclic actions during the required design life"*.

Key issues are when a cyclic load can be considered fatigue non-relevant, a structural type can be considered non-sensitive to cyclic action or when the number of cycles is non-significant. Current Eurocode EN 1992-1-1, [1], states, 6.8.1 (2):

> *"A fatigue verification should be carried out for structures and structural components which are subjected to regular load cycles (e.g., crane-rails, bridges exposed to high traffic loads)"*.

Hence, it does not provide specific cases to avoid fatigue verification but leave it to the engineer's judgment.

Regarding [5], the following list of cases for which a fatigue verification is not required is provided in clause 10, which is a novelty compared to [1], although not to EN 1992-2 [2], as commented below:

- common buildings subjected to a total number of significant load cycles ≤ $2 \cdot 10^4$,
- prestressing and reinforcing steel, in sections where, under the frequent combination of actions and P_k (prestressing actions), only compressive stresses occur at the extreme concrete fibres;
- external and unbonded tendons, lying within the depth of the concrete section".

Regarding a), the maximum number of cycles to avoid fatigue check is very low. For instance, any building resisting wind actions will be loaded by far more cycles. In EN 1991-1-4, B.3 [6], a relation between the number of cycles N_g and the amplitude of the wind gust, ΔS_k, is provided (Figure 1). It is shown than for 2·104 cycles the corresponding load amplitude of the gust, ΔS_k, would be around 35% of the characteristic load S_k, which is a non-negligible value and could be fatigue relevant (Figure 1). Of course, engineering judgment, again, is fundamental.

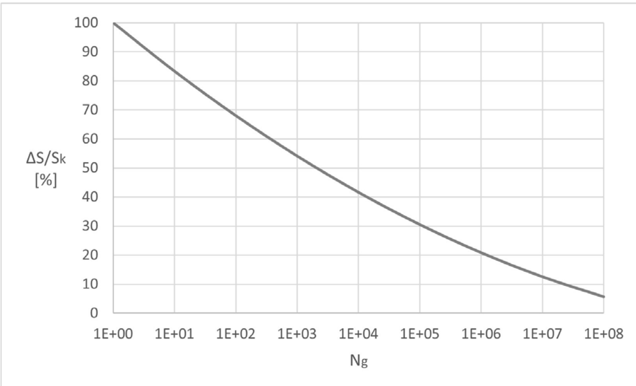

Figure 1. Number of gust loads N_g for an effect $\Delta S/S_k$ during a 50 years period [6].

Background for b) is clear. Prestressing and reinforcing steel in sections compressed under the frequent load combination will not have significant amplitudes, due to the uncracked condition of the section, and the compressive stress ranges are far less damaging than tensile ones. For instance, EN 1993-1-9 [7] states that compressive ranges of non-welded details shall be multiplied by 0.60.This provision b) was included in [2], but not in the general part [1].

Last, c), is based on the well-known fact that internal unbonded and external tendons will not have significant stress increments under service loads. These stress ranges depend on the deformations of the whole structure, there is no strain compatibility between concrete and steel, and this deformation must be controlled under SLS loads. According to the author's experience, this is generally true for bridges and other horizontal structures resisting primarily gravity loads. For support structures for Wind Turbine Generators, where deformations are not usually controlled under SLS loads, fatigue of external tendons should be verified due to the increasingly slenderness of the towers and the subsequent large displacements of the upper anchor of the post-tensioning system as well as the bending stresses at the anchors or other devices such deviators if specific measures are not taken to avoid them.

In EN 1992-2, [2], part of bridges, it is stated:

A fatigue verification is generally not necessary for the following structures and structural elements:

- footbridges, with the exception of structural components very sensitive to wind action;
- buried arch and frame structures with a minimum earth cover of 1.00 m and 1.50 m respectively for road and railway bridges;
- foundations;
- piers and columns which are not rigidly connected to superstructures;
- retaining walls of embankments for roads and railways;
- abutments of road and railway bridges which are not rigidly connected to superstructures, except the slabs of hollow abutments;
- prestressing and reinforcing steel, in regions where, under the frequent combination of actions and P_k only compressive stresses occur at the extreme concrete fibres.

Hence, [2] does provide a list of cases where fatigue can be assumed as negligible. But this list is kept in Annex K, [5], which is an Annex specific for bridges. Just point g) of the above list has been removed, but it is included in the general part, chapter 10, as already commented.

Other proposed exclusions were finally not included in [5], either in chapter 10 or Annex K. For instance a specific and interesting claim of U.K was related to fatigue of reinforcement of deck slabs bridges designed by conventional means. This claim is implemented in the UK National Annex of [2], where it is stated that fatigue verification is not required if the deck slab complies with certain requirements. According to UK's research, fatigue of reinforcement due to live load is typically around 10% of elastic predictions due to compressive membrane action work. This proposal was finally excluded, but the possibility of its inclusion, as well as other national claims, by means of an NCCI (Non-Contradictory Complementary Information for the use of EN Eurocodes at the National level) is allowed.

3. METHODS OF VERIFICATION

Whereas in [1] there is not a summary of the methods for the verification of the Ultimate Limit State, ULS, of fatigue, in [5] such summary is provided in 10.1:

- Simplified methods given in paragraphs 10.4 to 10.7.
- Refined methods:
  - Using damage equivalent stresses in Annex E, E.4 and Annex K, K.10 where applicable or
  - Explicit method using Palgrem-Miner rule in Annex E, E.5 where applicable.

Hence, levels of approximation are provided, being the more accurate the application of the Palgrem-Miner rule.

Damage equivalent stress method is only feasible if damage equivalent stresses, or loads, are provided. These equivalent loads are provided exclusively for bridges in Annex K, both railway and road bridges, and their calculation is based on several simplifications. Nevertheless, it is a more accurate method for standard elements of the bridge, i.e., beams, decks, girders, etc., than the simplified methods.

Palgrem-Miner rule requires the knowledge of the history or time series of the stress or load of interest. Alternatively, it is possible to apply a counting method (rainflow, reservoir) to these time series and get the stress histograms or the corresponding Markov matrices. This is the more accurate method, and the standard one in case of structures for wind turbines and non-standard elements of bridges.

It is noteworthy that damage equivalent stress range and Palgrem-Miner rule were included in [2], part for bridges, but not in the general part, [1]. Main reason could be that, at the time of the elaboration of [1] and [2], bridges were the main concrete structures subjected to cyclic loading, whereas wind turbines were not in the close horizon and offshore structures were specifically excluded of the current Eurocodes. Including these refined methods in the general part is clearly more rational.

4. COMBINATIONS OF ACTIONS

In [1] the following specific combination of actions is provided, 6.8.3. (Eq. 1):

$$(\Sigma_{j\geq 1} G_{kj} \text{"+"} P \text{"+"} \psi_{1,1} Q_{k,1} \text{"+"} \Sigma_{i\geq 1} \psi_{2,1} Q_{k,i}) + Q_{fat} \quad (1)$$

The proposed combination is the frequent combination of actions for transient or persistent situations plus the relevant fatigue load, Q_{fat}

Definition of Q_{fat} is explicitly given:

> *"Q_{fat} is the relevant fatigue load (e.g. traffic load as defined in EN 1991 or other cyclic load)"*

In [5] the "fatigue" combination of actions is slightly modified (Eq. 2):

$$\Sigma F_d = \Sigma_i G_{k,i} + \Sigma_j \psi_{2,j} Q_{k,j} + (P_K) + F_{fat,d} \quad (2)$$

We can identify the proposed combination as the quasi-permanent combination of actions for transient or persistent situations plus the design value of the fatigue action, $F_{fat,d}$, as leading action. $F_{fat,d}$, is defined as the cyclic component of the frequent load.

$F_{fat,d}$ is explicitly defined for road and railway bridges in [5]. Concretely, for road bridges, $F_{fat,d}$ can be taken as the frequent load of Load Model 1 and for railway bridges as the frequent load of Load Model 71 according to according to EN 1991-2 [8]. For other cyclic loads, definition of $F_{fat,d}$ is not explicitly given, and its election shall be based in engineering judgment but always using the frequent value.

It is important to notice that the above values of $F_{fat,d}$ for bridges, frequent values of Load Model 1 and Load Model 71, are only valid for the simplified verification of fatigue according to paragraphs 10.4 to 10.7 of [5]. More refined methods, such damage equivalent stress range or Palmgren Miner rule requires a different definition of $F_{fat,d}$. For the damage equivalent stress approach, $F_{fat,d}$ shall be precisely the equivalent load, which is defined in Annex K for both road and railway bridges. In case of road bridges, the specific fatigue load model from which the equivalent load is calculated is the Fatigue Load Model 3, whereas for railway bridges is the Load Model 71 (or SW/0 when required).

Regarding the differences between combinations proposed in the current version and in the draft, it would seem that the combination in [5] is more favourable than the current one, quasipermanent versus frequent load combination. A closer look yields only small differences.

The reason is that the definition of the cyclic load doesn't change, frequent value of the cyclic action, and this load is the main source of fatigue damage. Nevertheless, other actions may have an impact. This impact is due to the inherent non-linearity of concrete cross sections due to cracking. For structural steel, the amplitude or range of the cyclic load would be the only source of fatigue damage, but in case of concrete, reinforced or prestressed, the cross section shall be considered cracked, and therefore the stress assessment of concrete and reinforcing and prestressing steel shall take into account every force defined in the combination, especially if axial forces are involved. Besides, fatigue of concrete does depend not only on the stress range but also on the mean stress, what, again, oblige to include every force acting on the cross section.

However, impact of changing frequent values of the non-cyclic actions by their quasipermanent value will not have a significant impact. Usually, the more significant non-cyclic action, at least for bridges, would be the thermal load if the structure is statically indeterminate or the cross section is composite. But the difference between ψ_1, frequent value specified in [1], and ψ_2, quasipermanent value specified in [5], is very small. According to EN1991-2 [8], ψ_1 = 0.6 and ψ_2 = 0.5. Hence the impact on the fatigue verification is minimal, and on the other hand, it seems more correct to consider the quasi-permanent value for fatigue verification.

Regarding wind actions, the scenario is slightly different. ψ_1 is taken as 0.5 and ψ_2 is taken as zero, EN1991-2 [8]. Hence, the proposed combination in [5] does not include wind in the fatigue combination whereas the current one does, with the frequent value. If cyclic action of wind is not considered relevant, neglecting the static value as [5] does for the fatigue combination is rational. Of course, wind can be adopted as the leading cyclic action in several cases, clearly in support structures for wind turbines but also in case of other type of structures where wind may induce significant stress ranges and number of cycles, including specific aerodynamic effects such vortex shedding, and this requires engineering judgment.

One last consideration is that this fatigue combination shall not be adopted as a limit for consideration of cracking. Cross sections or structural elements shall be considered cracked, as explained in the next paragraph.

5. INTERNAL FORCES AND STRESSES

First, it is important to note that cyclic internal forces and stresses shall be calculated under service conditions. Appropriate stress-strain relationships shall be adopted, although linear relationship is recommended, and strain compatibility must be assumed. For assessment of stresses, assumption of cracked concrete is prescribed. It is worth to point out that, in prestressed members, according to paragraph 9.2.2 (7), [5], if, under the characteristic combination of actions the tensile stress in the concrete is below $f_{ct,eff}$, effective concrete tensile strength, the section can be considered uncracked. This would be beneficial for reinforcing and prestressing steel, as well as for concrete, but then fatigue of concrete under tensile stress ranges must be verified. This verification is not covered in [5], which just covers, as [1] does, concrete fatigue under compressive stresses, not under tensile stresses or compressive-tensile stresses. Hence, the assumption of cracked concrete is the only possible one. This assumption is correct if no stress reversals occur in the fibber under

TABLE 1.
Ratio of bond strength ξ between tendons and reinforcing steel

prestressing steel	ξ		
	pre-tensioned	bonded, post-tensioned	
		≤ C50/60	≥ C70/85
smooth bars and wires	Not applicable	0.3	0.15
Strands	0.6	0.5	0.25
indented wires	0.7	0.6	0.30
ribbed bars	0.8	0.7	0.35

Note: For intermediate values between C50/60 and 70/85 interpolation may be used

study, i.e., if the fibber is always subjected to tensile stresses, as usually happens in bridges. If stress reversals occur, i.e., if the fibber is subjected to compression-tension stress ranges, for instance in structures for wind turbines, consideration of cracked cross sections shall be carefully analysed, specially if concrete has a significant humidity and other codes and standards should be applied.

Regarding the different bond behaviour of prestressing and reinforcing steel, which has an impact in their stress assessment, the approach has slightly changed. The current formulation in [1] is correct only if reinforcing and prestressing steel are in the same position. The new approach allows to calculate stresses for different locations of reinforcing and prestressing steel. For this, an equivalent area of prestressing steel, A_e, is defined, (Eq. 3), function of the bond strength ratio between tendons and reinforcing steel, implemented by means of parameter ξ. This different bond strength is equivalent to a larger stiffness of the reinforcing steel compared to prestressing steel and therefore techniques for composite cross section analysis, i.e., equivalent sections, can be used:

$$A_e = A_p\sqrt{\xi\frac{\phi}{\phi_p}} \qquad [3]$$

Please note that in Table 10.1, Note 2 [5], it is stated that provided ξ values are valid just for "*tendons directly cast into concrete or contained within corrugated metal ducts*", which means that plastic ducts, widely used, are excluded. If such ducts are used, no provisions are given. This could be problematic since in the European Technical Assessment or Approval of prestressing systems this value is generally not given, but if the tendons are intended to be bonded the plastic ducts must be corrugated, not being expected large differences between the ξ values of metallic ducts which, additionally, are affected by a square root. Table 1 show the values of ξ considered in [5].

A very important novelty is the consideration of the redistribution of stresses in concrete in the compression zone. This redistribution allows to consider a reduction of the concrete stresses, which can be of importance for reinforced concrete and less significant for prestressed concrete.

Application is straight forward; the stress calculation shall be carried out at the fibre located 100 mm from the most compressed edge but limiting the 100 mm distance to 1/3 of the cross-section depth and limiting the calculated value of the stress to 2/3 of the maximum stress at the extreme fibre of the cross-section. This is a simplified approach in general, and sometimes conservative compared to the approach of Model Code 2010, [3].

Redistribution of stresses under fatigue loading has experimental background. Most loaded fibber under cyclic stress will soft and the less stressed fibbers will absorb more stress. According to Zanuy [10], redistributions in reinforced concrete beams, not over-reinforced, practically avoid any failure of compressed concrete under cyclic loads, so that the typical fatigue failure takes place in the reinforcement. This is not true, according to [10], in prestressed members, over-reinforced cross sections, columns or piers or when the cyclic concrete stresses are very high.

In the author's opinion, the formulation for redistribution of stresses provided in [5] is quite conservative. Besides the importance of this redistribution for a proper assessment of highly variable stress zones, is undoubtful. Lack of experimental work on this matter effectively prevents more refined approaches, and this is reflected in [5].

6. REINFORCING AND PRESTRESSING STEEL. S-N CURVES. SIMPLIFIED VERIFICATION

General approach for fatigue verification for steel, reinforcing and prestressing, has not been changed. Simplified and refined methods can be used, both based on the S-N curves. However, these curves have been updated. Below new S-N curves, new corresponding results of the simplified verification and other changes are described.

S-N curves for reinforcing steel are provided in [1], Table 6.3N, which is reproduced below, (Table 2):

TABLE 1.
Ratio of bond strength ξ between tendons and reinforcing steel

Type of reinforcement	*N**	*Stress exponent*		*$\Delta\sigma_{Rsk}$ (MPa) at N* cycles*
		k_1	k_2	
Straight and bent bars[1]	10^6	5	9	162.5
Welded bars and wire fabrics	10^7	3	5	58.5
Splicing devices	10^7	3	5	35

Note 1: Values for $\Delta\sigma_{Rsk}$ are those for straight bars. Values for bent bars should be obtained using a reduction factor $\xi = 0.35 + 0.026$ D/ϕ.
where:
D: diameter of the mandrel.
ϕ: bar diameter.

[5] provides different S-N curves for reinforcing steel in Annex E, Table E.4, given below as Table 3:

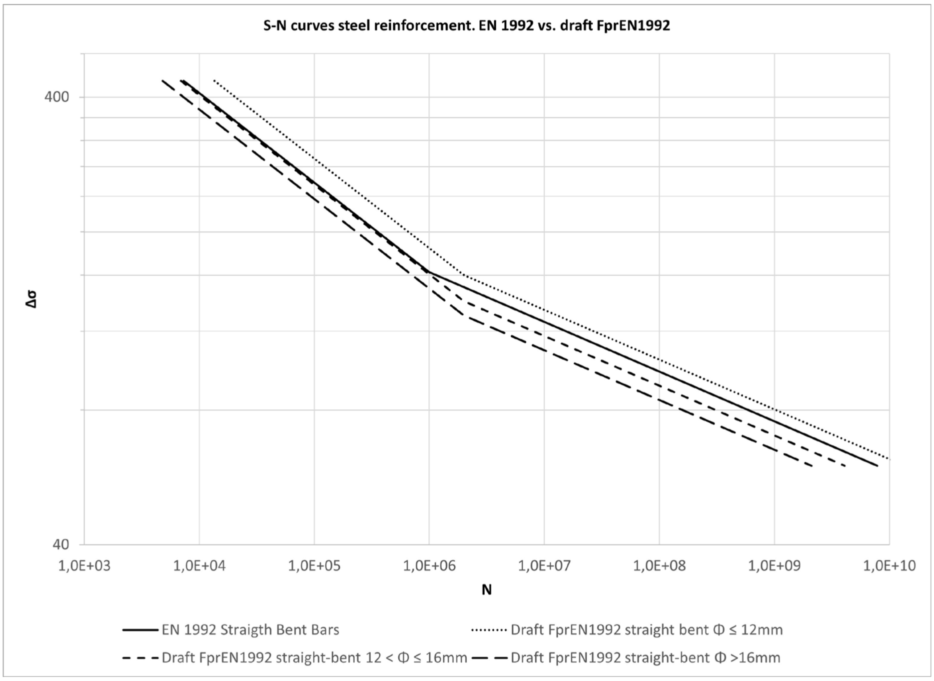

Figure 2. Comparison between S-N curves for non-welded reinforcement [5].

TABLE 3.
Parameters of S-N curves for carbon reinforcing steel [5].

Type of reinforcing steel	Diameter	*$\Delta\sigma_{Rsk}$ 5%-quantile (Test σ_{max}=0,6f_{yk})*			
		$\Delta\sigma_{Rsk}$ [MPa]	***N****	***Stress exponent***	
				k_{f1}	***k_{f2}***
Bars[a]	$\phi \leq 12$ mm	160	2 10^6	5	9
	12 mm < $\phi \leq 16$ mm	140			
	16 mm < $\phi \leq 20$ mm	130			
	$\phi > 20$ mm	130			
	$\phi \leq 12$ mm	100		3	5
Type of reinforcing steel	$\phi > 12$ mm	80			
Couplers[c]	-	35		3	5

a Values for bent parts of bars should be obtained using a reduction factor ξ=0.35 + 0.026 ϕ_{mand}/ϕ. The reduction factor ξ may be omitted for shear reinforcement with 90° stirrups $\phi \leq 16$ mm and depth h ≥ 600 mm.

b Values for $\Delta\sigma_{Rsk}$ of tack welded apply for a distance of 5ϕ at each side of the weld.

c Values for couplers apply unless more accurate S-N curves are available and confirmed by testing.

NOTE: The 10% quantile values for material according to table C1.a and C2.a are based on a confidence level of 90% whereas confidence levels probabilities for design $\Delta\sigma_{Rsk}$ (5% quantile values) are 75% according to EN 1990:2010, Annex D

Hence, S.N curves for rebars has changed. In [15], the corresponding background document for these new S-N curves, the two main reasons for these changes are explained. First, it is stated that bar diameters equal or below 16 mm are more relevant regarding fatigue because of the increasing application of post- and pre-tension, and these smaller bars, 6 to 20 mm, are today efficiently produced as mechanical straightened bars from coils (de-coiled bar). De-coiling has a negative influence in the fatigue properties of the bars, and it must be addressed in the corresponding S-N curves, effectively ruling out other production methods regarding fatigue verifications.

Secondly, tack welding, instead of binding the bars with wires, has become the standard method for efficient prefabricated construction, and of course it has an impact in fatigue design and it must be addressed.

Hence, more than 500 test were carried out, mainly of 12 and 16mm bars, mechanically straightened and cross-welded bars by resistance welding and CO_2 tack welding.

These test campaign led to the modifications of the S-N curves in [5] shown in Table 3. Concretely, to the change of the knee from N^* = 10^6 (straight bars and bent bars) and N* = 107 (welded bars and wire fabrics, as well as splicing devices) to N^* = 2·106 in [5] for every type of reinforcement.

A quick comparison of the S-N curves for straight and bent unwelded bars shows that the proposed S-N curves in [5] are more conservative for ϕ > 16mm, as shown in Figure 2, where characteristic S-N curves are shown. For diameters from 12mm up to 16mm the proposed S-N curves are also more conservative, especially for low ranges, however for diameters lower than 12 mm cycles are clearly better.

Regarding welded bars, S-N curves are compared (Figure 3). For diameters less or equal than than 12 mm S-N curves in [5] are more favourable for low stress ranges and match the current S-N curve in [1] for stresses above the knee. For diameters larger than 12mm, the proposed S-N curve is less favourable stresses above the knee and matches the S-N curve in [1] below the later. Note that just tack weld and welded fabrics are included in [5], whereas in [1] the S-N curves are given for welded bars in general. On the other hand, in paragraph 10.4 of [5], the stress range limit to avoid fatigue verification is given for butt and tack welds, i.e., it can be deduced that butt welded bars may be verified by the S-N curve of Table 3. Other types of welded reinforcement, such as lap or cruciform joints, admitted in [1], would be excluded. Here, a more refined analysis would be required.

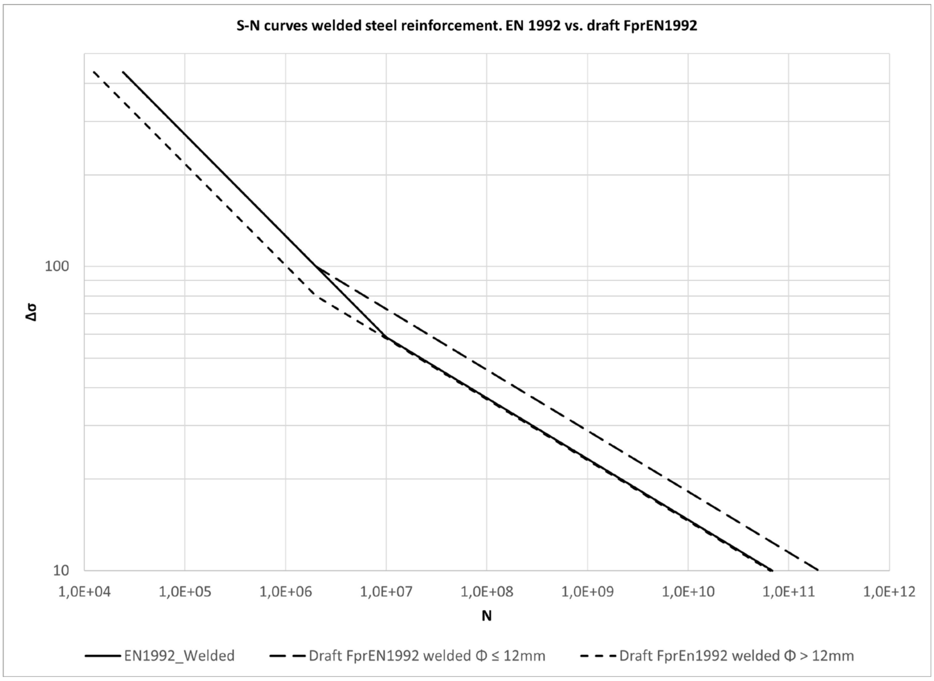

Figure 3. Comparison between S-N curves for welded reinforcement.

It is worth to comment the footnote in Table 3, "*The 10% quantile values for material according to Table C1.a and C2.a are based on a confidence level of 90% whereas confidence levels probabilities for design $\Delta\sigma_{Rsk}$ (5% quantile values) are 75% according to EN 1990:2010, Annex D*". This note can produce some confusion and a brief comment is given below.

In the mentioned tables C.1 and C.2 10% quantile values of $\Delta\sigma_{Rsk}$, i.e., 2·σa, are given, but these values are obtained from testing on bare reinforcing bars. In Table E.1, values of $\Delta\sigma_{Rsk}$ are the same than in tables C.1 and C.2, but for a 5% quantile. These values apply for reinforcement embedded in concrete.

Embedment in concrete will improve the fatigue behaviour of the bars about 10 to15%, but this won't justify the same values for 10 and 5% quantiles. Hence in the note is specified that values of tables C.1 and C.2 mentioned above are given for a 90% confidence level, whereas values in Table 3 are given for a 75% confidence level, which is the standard confidence level obtained by the statistical methods of design assisted by testing, according to EN 1990 Annex D [11]. In conclusion, values are coherent in both tables.

Regarding the influence of bent parts, of critical importance for the assessment of shear reinforcement and other bent bars, it is treated as in [1], including the same formulation for the reduction factor of $\Delta\sigma_{Rsk}$, ξ. But an important novelty is given in Table 3, note [a], where it is stated that for shear reinforcement with 90" stirrups, $\xi \leq$ 16mm and depth h ≥ 600 mm, influence of the bent part may be omitted. For standard hook of bars smaller than 20 mm, $\phi_{mand} = 4{\cdot}\phi$, the reduction factor is $\xi = 0.454$, which leads to a drastic reduction of the fatigue life. Hence, neglecting the influence of the bent implies an important improvement. Aside of the maximum bar diameter and 90º angle, an important requirement is that the element depth shall be larger or equal than 600 mm, what allows the anchorage of the compressive strut of the shear force in the straight part of the bar, before reaching the bent part. This is the main reason to neglect the undoubtable impact of the bent in the fatigue life of the bar. For smaller depths or larger diameters, a detailed analysis of the stress distribution along the bar may also allow some improvement of the fatigue life of bent bars.

For couplers, denoted splicing devices in [1], the S-N curve is modified, being less favourable due to the reduced value of N*, but it is stated that European Technical Product Specifications can be used, since several suppliers provide special couplers with improved fatigue life.

S-N curves for prestressing steel are subjected to minor changes, which are shown in Table 4.

TABLE 4.
Parameters of S-N curves for prestressing steel [5].

S-N curve for prestressing steel	*N**	*Stress exponent*		*$\Delta\sigma_{Rsk}$ (MPa) at N* cycles[a]*
		k_1	*k_2*	
Pre-tensioning	10^6	5	9	185
Post-tensioning				
- single strands in plastic ducts	10^6	5	9	185
- curved tendonsb in steel ducts	10^6	5	9	150
- straight tendons[b] or curved tendons[b] in plastic ducts	10^6	3	7	120
- anchoring devices and couplers	10^6	5	5	80

Note 1: Values in Table E.2 (NDP) apply for prestressing steel complying with Table C.3 to C.5 and prestressing systems complying with 5.4.
a Values correspond to prestressing steel embedded in concrete
b Applies to tendons with wires and strands; tendons with bars are not covered.

First change can be found in the general footnote. These S-N curves are applicable if prestressing system complies with subclause 5.4 of [5], where it is stated that the prestressing system must comply with the relevant standard for prestressing systems, being recommended EAD 160004-00-0301 (i.e., former E.T.A.G 013). In practical words, the prestressing system shall be in possession of the corresponding European Technical Assessment Document (E.T.A).

In footnote b an important provision is given. Prestressing bars are excluded, no S-N curve is given for them. Background for this exclusion is the presence of threads in the bar, worsening the fatigue behaviour of the bars compared to strands or wires. Since cyclic loads on these elements in concrete structures are not uncommon, mostly in connections steel to concrete or concrete to concrete, it is worth to point out that fatigue verification of these bars is usually carried out according to EN 1993-1-9 [7], with a Detail Category (DC) of 50 MPa. This DC is valid if bending stresses in the bar are considered, which is not common unless detailed Finite Element Model of the connection is used. If bending stresses on the bar are not considered in the fatigue verification other codes and standards, [13] and [14] for instance, recommend using a lower DC, 36 MPa, to take into account the additional damage due to the non-contemplated bending stresses.

Regarding simplified verification of reinforcing and prestressing steel is provided in subclause 10.4 of [5], by means of maximum stresses under the fatigue load combination already mentioned. For unwelded and welded reinforcing steel, a comparison with [1], subclause 6.8.6, is given in Table 5.

TABLE 5.
Comparison among simplified verifications for welded and unwelded reinforcing bars in [5] and [1].

New EC2 [5]		*Old EC2 [1]*	
Type of bars, [5]	$\Delta\sigma_{sd,max}$, [5]	Type of bars, [1]	$\Delta\sigma_{sd,max}$, [1]
Unwelded, $\phi \leq$ 12mm	90 MPa	Unwelded	70 MPa
Unwelded, $\phi >$ 12mm	73 MPa		
Butt and tack welded, $\phi \leq$ 12mm	40 MPa	Welded	35 MPa
Butt and tack welded, $\phi >$ 12mm	30 MPa		
Couplers	24 MPa	Couplers	-

Limits are more relaxed in [5], for unwelded reinforcing bars, better for welded reinforcement with diameters $\phi \leq 12$ mm, and more exigent for welded reinforcements with $\phi > 12$ mm. Limit for couplers is a novelty. Limits for prestressing, differentiating pre and post-tensioning, are also given, which is an important novelty of [5].

Regarding the combination of actions to be used, the one specified in 10.2, [5], shall be adopted, but a maximum number of cycles is provided, 108 cycles. In contrast, in [1], no maximum number of cycles is provided. Specification of number of cycles for the simplified verification allows calculating these limits directly, as shown below.

Concretely, it is easy to check, based on the mentioned S-N curves that the provided values correspond to N = 108, being N the number of design cycles. For instance, for straight bars, with $\phi \geq$ 12mm, and applying the corresponding S-N curve (Eq. 4):

$$(1.0\ \Delta\sigma_{sd})^9\ 10^8 \leq \left(\frac{130}{1.15}\right)^9 2\ 10^6 \rightarrow \Delta\sigma_{sd} = 73\ Mpa \qquad (4)$$

the result matches the proposed simplified value, with $\gamma_{F,f}$ = 1.00, $\gamma_{F,f}$ = 1.15, k_{f2} = 9 and $\Delta\sigma_{Rsk}$ = 130 MPa, i.e., it has been adopted for this simplified assessment the S-N curve for $\phi > 20$ mm also for diameters between 12 and 20 mm besides diameters larger than 20 mm. Since the number of cycles is given, this simplified verification can be adjusted for diameters larger than 12 mm and up to 20 mm.

7.
CONCRETE UNDER COMPRESSION

7.1. The concrete fatigue phenomena

At material lever, the number of compression (or compression-tension) cycles that concrete is able to bear before the material failure is driven by several complex phenomena. Some of these phenomena controlling the material fatigue resistance are concrete compressive strength, concrete tensile strength, fibre amount and orientation (for FRC), water content (humidity), concrete fracture energy, cement type and aggregates type and size among others. Other phenomena control the fatigue action, like the peak compressive stress, the valley compressive, or tensile, stress, the load frequency, the stress gradient, the load path and the load history among others.

7.2. Compression fatigue verification methods

Since there is not a general fatigue formulation covering all the above-mentioned aspects in the state of the art, old Eurocodes [1] and [2], as all other concrete codes, includes in its formulations only a small part of these parameters: the most important; covering the high resulting uncertainty by a set of high safety coefficients and parameters.

The new Eurocode, [5], includes three levels of concrete compression fatigue verifications:

- A first level, called simplified verification, that can be found at clause 10.5 of this standard [5], does not take into account the number of load cycles, as far as they are less than ten million, and just limits the maximum peak stress and stress range, under the fatigue combination of loads, with a simple lineal equation.
- A second level, called damage equivalent stress, that can be found at Annex E chapter E.4.3 of this standard [5], considers, not the maximum stresses as the simplified method but the damage equivalent stresses, through a more detailed formulation.
- A third level, called Palgrem-Miner rule, that can be found at Annex E chapter E.5.3 [5] that allows for a detailed account of the damage induced by each individual load cycle, depending on its peak and valley stresses.

All the three methods evaluate the fatigue resistance of concrete working with the stress level, that is the ratio of the true stress (peak or valley or equivalent) to a notional design fatigue strength of concrete $f_{cd,fat}$.

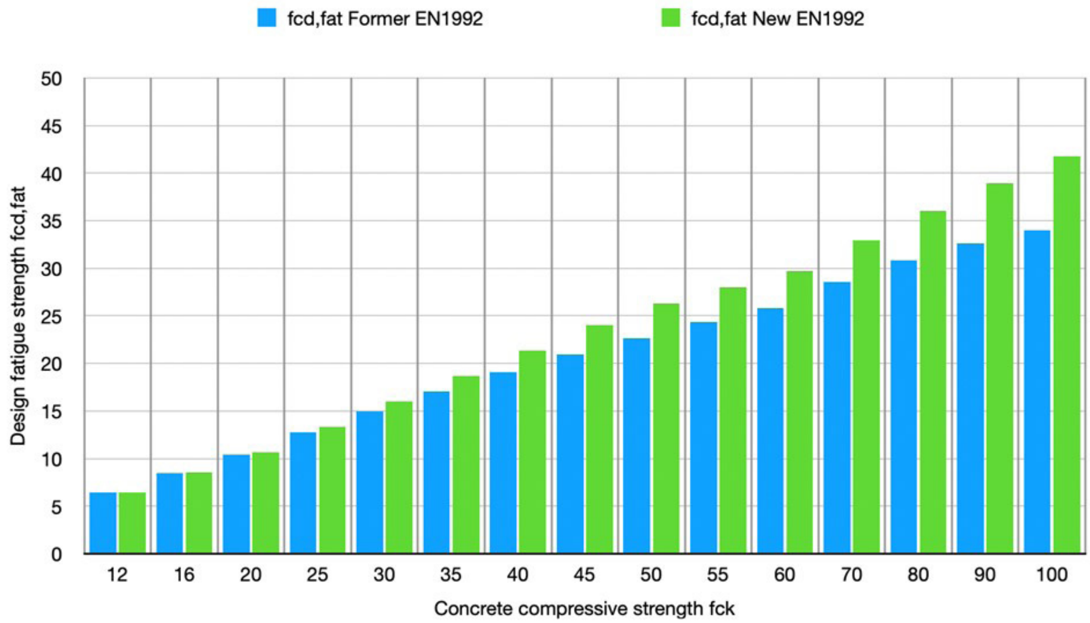

Figure 4. Comparison of $f_{cd,fat}$ between [2] and [5].

7.3. *The design fatigue strength $f_{cd,fat}$*

The design fatigue stress $f_{cd,fat}$ is a notional stress that is used to normalize the fatigue stress levels for the three methods. Therefore, this is the key parameter controlling the concrete compression fatigue at the code formulation.

Physically, $f_{cd,fat}$, may be understood as a stress such as if being reached in only one cycle it produces the failure of the material; therefore the fatigue peak stress of the loading cycles needs to stay as much under $f_{cd,fat}$ as higher is the number of acting cycles.

The equation used in [5] for fcd,fat is as follows (Eq. 5):

$$f_{cd,fat} = \beta_{cc}(t_0)\, f_c\, \frac{f_{ck}}{\gamma_c}\, \eta_{cc,fat} \tag{5}$$

where:

$\beta_{cc}(t_0)$ is a coefficient of concrete strength at first load application t_0.

$\eta_{cc,fat} = min\,\{0.85\,\eta_{cc}\,;0.8\}$

The equation used in [1] for $f_{cd,fat}$ is as follows (Eq. 6):

$$f_{cd,fat} = k_1\, \beta_{cc}(t_0)\, f_c\, (1 - f_{ck}/250) \tag{6}$$

where:

$\beta_{cc}(t_0)$ is a coefficient of concrete strength at first load application .

t_0 is the time of the start of the cyclic loading on concrete in days.

The value of k_1 for use in a country may be found in its National Annex. The recommended value for N = 10^6 cycles is 0.85.

Equation 5 is very similar to the one used in [1] (Eq. 6), being the only difference the $\eta_{cc,fat}$ factor, a coefficient that reduces the fatigue strength for the concrete strength classes over 40 MPa. This new factor $\eta_{cc,fat}$ replaces the former k_1 and $(1- f_{ck}/250)$ factors.

This coefficient, in [5], is based in the general η_{cc} coefficient, that applies for static loading. In [1], which lacked this general η_{cc} coefficient for the static compressive strength, the coefficient was obtained directly from the compressive strength.

Figure 4 shows the current and the new resulting design fatigue strengths for the concrete classes covered by the standard. In both cases, former and new Eurocode, the standard recommended values, above mentioned, for k_1 and $\beta_{cc}(t_0)$ have been used to make the comparison.

The new formulation gives slightly lower fatigue strengths for the lower concrete classes and bigger values for the high strength classes, giving a net strength increase of 23% for the C100.

This change in the fatigue compressive strength in [5] is made in the opposite direction of the change in the static design compressive strength, that is reduced in the new code proportionally to the increase in the f_{ck}, being the bigger reduction applied for the C100 with a 25% reduction of the static compressive strength relative to [1].

7.4. *Simplified verification*

This first method provided in [5], is very similar to the one present in [1]. The criteria is as follows (Eq. 7) [5]:

$$\frac{|\sigma_{cd,max}|}{f_{cd,fat}} \leq 0.5 + 0.45\,\frac{|\sigma_{cd,min}|}{f_{cd,fat}} \leq 0.9 \tag{7}$$

where:

$\sigma_{cd,max}$ is the maximum compressive stress at a fibre under the fatigue load combination according to 10.2 [5].

$\sigma_{cd,min}$ is the minimum compressive stress at the same fibre where $\sigma_{cd,max}$ occurs.

$f_{cd,fat}$ is the design fatigue strength of concrete according to 10.5 [5].

Note that, as commented above, if $\sigma_{cd,min}$ is tensile, it shall be considered as null, since concrete must be considered cracked.

The new formula is exactly the same as the one present in [1], being the only difference the absolute limiting value 0.90 for peak the stress level.

This maximum peak stress level was fixed at 0.90 for concrete classes up to 50 MPa and limited to 0.80 for classes over 50 MPa. In [5] the same 0.90 is used for all concrete classes, giving an additional 12% increase in the strength for the higher concrete classes on top of the increase of the design fatigue strength previously described.

Therefore, the global increase in the fatigue strength for a C100, according to the simplified method, is over 38% compared to [1].

7.5. Damage equivalent stress amplitude method

This second method has been moved from the main article 6.8.7 in [1] to the new Annex E in [5]. The formulation used is the same used in [1], and can be found now at subclause E.4.3, [5]:

$$\frac{|\sigma_{cd,max,equ}|}{f_{cd,fat}} + 0.43\sqrt{1-\left|\frac{\sigma_{cd,min,equ}}{\sigma_{cd,max,equ}}\right|} \leq 1 \tag{8}$$

Where:

$f_{cd,fat}$ is the design fatigue strength of concrete according to 10.5 [5].

$|\sigma_{cd,max,equ}|$ is the upper stress of the damage equivalent stress amplitude for N=10^6 cycles.

$|\sigma_{cd,min,equ}|$ is the lower stress of the damage equivalent stress amplitude for N=10^6 cycles.

Both [1] and [5] use the same term $\sigma_{cd,max,equ}$, nevertheless, the definition for $\sigma_{cd,max,equ}$ in [1] is "*the upper stress of the ultimate amplitude for N cycles*" and his ratio to $f_{cd,fat}$ was called "*maximum compressive stress level*" while in [5] the definition for the same term is the more precise "*the upper stress of the damage equivalent stress amplitude for N=10^6 cycles*"; and the same applies for $\sigma_{cd,min,equ}$.

Most of the structures subjected to fatigue coming from wind, wave and traffic loads are subjected to fatigue cycles of different amplitude, usually a random amplitude following some statistical distribution. The "damage equivalent stress amplitude" is a term commonly used in fatigue of metals subjected to this kind of loads, that refers to a notional constant stress amplitude which for a fixed number of cycles (N=10^6 cycles in this case) produces exactly the same damage as the true variable (usually random) stress amplitude loads.

In the case of metals this "damage equivalent stress" can be calculated directly for the "damage equivalent loads" that can be obtained by simple calculations over time history loads or loads spectra due to the linear nature (and the almost null influence of the mean stress) of the S-N curves in metals.

In the case of concrete under compression, since there is not such a linear S-N relationship, a precise definition of the "damage equivalent loads" is needed for every structural application (i.e., bridges, towers, sea structures, etc.).

In [5] these equivalent loads are provided for the specific case of railway bridges at the new Annex K article K.11.3 [5].

7.6. Palmgren-Miner rule method

This third, and most precise method, the Palmgren-Miner rule, was present in the old Eurocode only at the bridges part of the code, [2]. Now it is included in the new annex E and can be used for all the structures covered by [2], including some of the structures at which the compression fatigue normally drives the design, like the offshore structures and the wind turbine support structures, previously excluded of the scope of [2].

The Equation E.8 used in [5], Annex E, is exactly the same found in [2] (Eq. 9):

$$N_i = 10^{k_i} \tag{9}$$

where:

N_i is the number of cycles to fatigue failure for each stress-level.

k_i is a coefficient which can be obtained with the following formula (Eq. 10).

$$k_i = C\,\frac{1-\left|\dfrac{\sigma_{cd,max,i}}{f_{cd,fat}}\right|}{\sqrt{1-\left|\dfrac{\sigma_{cd,min,i}}{\sigma_{cd,max,i}}\right|}} \tag{10}$$

where:

C = 14 may be taken for concrete under compression and not permanently submerged in water.

$\sigma_{cd,max,i}$ is the maximum compressive stress in stress-level "i", .

$\sigma_{cd,min,i}$ is the minimum compressive stress in stress-level "i", .

$f_{cd,fat}$ is the design fatigue strength of concrete according to 10.5 [5].

The difference between [2] and [5] comes from two sources:

- As previously mentioned, the current EN 1992-2 [2] is allowed to be applied only to bridges, and under the specific bridge loads combinations defined in the same Part 2 of [2], while [5] allows the application to any kind of structure, but those permanently submerged in water.
- The reference concrete fatigue design strength used to drive the stress level is increased in [5] as explained above, leading to a much higher number of resisting cycles for the same stresses at the higher concrete classes.

This increment in the fatigue compressive strength is very significant, since with [5], some structures whose design is driven by the compression fatigue strength of the concrete, like those under predominantly waves or wind loads, can now be designed with this new formulation.

8.
SHEAR

It is possible to split the fatigue verification of members under shear in two cases, members requiring and not requiring shear reinforcement.

8.1. Members not requiring shear reinforcement.

No modifications have been implemented in the simplified verification, except replacing forces, $V_{Ed,max}$ and $V_{Rd,c}$, by stresses, $\tau_{Ed,max}$ and $\tau_{Rd,c}$.

Considering the extensive modification of the shear strength of members not requiring shear reinforcement carried out in [5], it is hard to tell if the simplified verification is more exigent than in [1].

Combination of actions for this simplified verification, and any other one, is the one proposed in 10.2 of [5].

Regarding the methods for refined fatigue assessment, damage equivalent stress range or Palgrem Miner rule, there is no formulation provided for members without shear reinforcement, no S-N curves are given. Model Code 2010 [3], however, provides a S-N curve for fatigue shear strength of members without shear reinforcement (Eq. 11):

$$\log N = 10\,(1 - V_{max} / V_{ref}) \tag{11}$$

where:

V_{max} is the maximum shear force under the relevant representative values of permanent loads including prestress and maximum cyclic loading.

$V_{ref} = V_{Rd,c}$

This S-N curve was not included in [5], but it will allow the use of Palgrem Miner rule in case that histograms or Markov matrices of shear forces are available.

8.2. Members requiring shear reinforcement.

In members requiring shear reinforcement both shear reinforcement and concrete struts must be verified. Hence simplified and refined methods for fatigue verification of reinforcing steel and concrete under compressive stresses can be applied.

Verification of fatigue for shear reinforcement and concrete struts strongly depends on the value of the angle of the struts to the bending reinforcement, θ. In the current EN 1992-1-1 the following formula is proposed in 6.8.2 (3) [1] for this angle when verifying fatigue (Eq. 12):

$$cot\theta_{fat} = \sqrt{cot\theta} \tag{12}$$

This formula considers the fact that the angle of the struts under fatigue loads, which are loads under service conditions, may be significantly larger than the one considered for ultimate loads. For instance, if $cot\theta$ = 2.50 for ULS verifications, $cot\theta_{fat}$ would yield 1.58 for fatigue checks. Hence fatigue design of shear reinforcement can be more determinant than ULS design, especially if the shear reinforcement presents an additional reduction of its fatigue strength due to the presence of a bent.

But the proposed formula is an estimation and does not consider the actual biaxial stress state in the member if axial force is present. For instance, the prestressing force acting on the member can significantly flatten the angle. Hence, although keeping the formulation in [1] for $cot\theta_{fat}$ the possibility of a specific calculation of $cot\theta_{fat}$ by means of the formulation of annex G is allowed by [5], using the maximum shear in the cycle.

Annex G provides information for assessment of SLS stresses, considering cracking, in G.5. Formulation is given for membrane elements, perfectly applicable for thin webs of T beams, box girders, etc., but also for solid cross sections with some adjustments. For this assessment of $cot\theta$ Annex G allows two approaches, elastic calculation, and the following more refined formula, which implies solving a 4th grade polynomial equation (Eq. 13) and takes into account the reinforcement amount:

$$\frac{|\tau_{Edxy}|}{\rho_x} cot^4\theta + \frac{\sigma_{Edx}}{\rho_x} cot^3\theta - \frac{\sigma_{Edy}}{\rho_x} cot\theta - \frac{|\tau_{Edxy}|}{\rho_x} = 0 \tag{13}$$

Regarding the compression strut, the same $cot\theta_{fat}$ shall be used. Reduction of compressive fatigue strength $f_{cd,fat}$ due to transverse tensile stresses is considered by means of factor ν. A simplified value of $\nu = 0.5$ is proposed in chapter 8 of [5] and directly recommended for the simplified verification of concrete under shear. A larger value of ν may be calculated, according to the formulation given in 8.2.3 (7), [5], if the ductility of reinforcement is B or C. In any case, when adopting the recommended simplified value of ν, 0.50, high strength reductions of concrete fatigue strength can be expected, and this can have an impact in the design of thin webs of precast beams and other members under cyclic loads. Hence, it is highly recommended to use the more refined value of ν. It is interesting to point out that even these refined values of ν are very conservative since the transverse reinforcement won't yield under cyclic loads, whereas the proposed formulation for ν assumes a yielded, or close to yield, reinforcement. A more accurate estimation of the strength reduction considering the stress level of the transverse reinforcement can be also found in Annex G. The formulation can be found in G.3, and it allows to consider levels of reinforcement stress lower than the yield strength, increasing consequently the value of ν. Of course, this is closely related to the multiaxial stress states, commented below.

8.3. Shear at interfaces

Treatment of shear at interfaces has completely changed in the new FprEN 1992-1-1:2023 [5]. Current provisions just state that value of the cohesion, c, shall be halved in case of fatigue or cyclic loads, 6.2.5 (5) [1].

In the new draft, approach is totally different. First, detailing may allow to avoid fatigue verifications, i.e., if the reinforcement through the joint is fully anchored and the interface is rough or keyed, no fatigue verification of the interface itself is required. Of course, this does not excuse the verification of concrete and reinforcement next to the interface.

If, as sometimes occurs in precast construction, reinforcement crossing the interface cannot be fully anchored, i.e., anchor (or lap) length is not enough to transmit the full design stress of the reinforcement, f_{yd}, or the interface is not at least rough, strength of the interface shall be checked according to the following equation (Eq. 14):

$$\Delta\tau_{Edi} \leq \Delta\tau_{Rdi} = \mu_{v,fat}|\sigma_n| + \rho\,\frac{\Delta\sigma_{Rsk}}{0.45\gamma_s}\,(\mu_{v,fat}\,sin\alpha + cos\alpha) \tag{14}$$

Where $\Delta\tau_{Edi}$ would be the stress range according to the fatigue combination already described.

This verification is very favourable, since there is a factor 0.45 dividing the fatigue strength of the reinforcement, $\Delta\sigma_{Rsk}$.

Hence, this in fact an increase of the fatigue strength of the reinforcement.

Main reasoning behind this improvement of the fatigue strength of the reinforcement is that, neglecting the cohesion term, *c*, the concrete strut at the interface will flatten, and this has a positive impact in the reinforcement stresses.

This approach is not totally clear, at least in the author's opinion. If cohesion is omitted in the verification of ULS of fatigue, it should also be omitted in the standard ULS verification, and it is not.

On the other hand, Model Code 2010, [3], recommends a reduction of the static strength of 40% if cyclic loads were present, and although this reduction may be quite conservative, compared to it the new approach in [5] is much favourable for the fatigue verification.

Of course, zones adjacent to the interface, which shall be checked, will usually be, in this case, determinant.

9. MULTIAXIAL STRESS STATES.

Multiaxial stress states are common in most of the members subjected to cyclic loads, from bridges to support structures for wind turbines, especially in unavoidable geometric transitions or relatively abrupt geometric changes, but there is little experimental or theoretical background regarding fatigue behaviour of concrete under multiaxial cyclic stresses.

On the other hand, [1], and [5], do consider the reduction of concrete compressive strength under fatigue loads, fcd,fat, in case of transverse tension, since the factor v shall multiply fcd,fat when verifying fatigue of the concrete strut under shear. Hence, it is implicitly assumed that tensile stresses will have an impact on the compressive fatigue strength not only for shear but for any biaxal or triaxial stress states with at least one positive principal stress. This means that, although not directly stated, [1] and [5] are assuming that parameters defining concrete strength under static multiaxial ULS stresses shall be also considered for fatigue strength verification of concrete under multiaxial cyclic stresses.

Accepting this assumption as correct, and it is correct according to [1] and [5], for biaxial stress states with at least one positive principal stress refined formulations, instead of simplified assessment, can be used for a calculation of the reduction factor v. Concretely Annex G in [5] allows assessing the reduction of compressive strength considering the real transverse reinforcement stress, and this will have a significant impact since, under cyclic loads, stress levels of transverse reinforcement will be significantly lower than its yield strength whereas the reduction of concrete strength given by the simplified value of factor v considers the reinforcement yielded or close to yield. Old Eurocode 2, part of bridges, [2], in subclause 6.109, also allowed this refined assessment of the strength reduction, but in a much more conservative way. In the author's experience the application of this Annex G will lead to more rational reductions of concrete strength for fatigue verification of concrete under compressive cyclic stresses and transverse tension.

Regarding confinement of concrete under bi or triaxial compressive stress states, no provisions are given in the current draft. There are few experimental results for confined concrete under cyclic compressive stresses, but several of them indicate an improvement of the fatigue strength, [12]. Additionally, since compressive fatigue strength reduction must be assumed if transverse tensile stresses exist, it seems rational to also consider the improvement of the compressive fatigue strength due to confinement. Despite this, no direct indications to consider confinement are given in [5]. It is worth to mention that in other codes, for instance in [14], it is allowed to consider confinement, but the increase in the fatigue compressive strength is limited to a factor of 1.30.

Last, it shall be pointed out that, as already commented, no provisions for verification of concrete fatigue under tensile or compressive-tensile stresses are given in [5], i.e., concrete shall be considered cracked.

10. APPLICATION TO BRIDGES

No significant changes have been carried out in [5]. Below are described the most important ones.

10.1. λ factors

λ factors are required to calculate the damage equivalent stress range, $\Delta\sigma_{s,equ}$ for prestressing and reinforcing steel in both road and railway bridges. They consider, according to [5], Annex K:

- $\lambda_{s,1}$: Type of element, e.g., simply supported or continuous beam, as well as the damaging effect of traffic by means of the critical length of the influence line or area
- $\lambda_{s,2}$: Traffic volume
- $\lambda_{s,3}$: Design life of the bridge
- $\lambda_{s,4}$: Number of loaded tracks or lines.

The only λ value that has significantly changed is $\lambda_{s,1}$, although this change is specified just for railway bridges, $\lambda_{s,1}$.

This factor must change since it is function of the shape of the considered S-N curves, and these curves have changed for reinforcing steel, welded and unwelded. Concretely the number of cycles at the knee, N^*, has changed, being now $2 \cdot 10^6$ for any reinforcing steel, welded or unwelded. $\Delta\sigma_{Rsk}$ has also changed, but it does not affect the values of $\lambda_{s,1}$. These values of $\lambda_{s,1}$ are given in [5] in Annex K, Table K.2, which is reproduced here in Table 6.

In Table 6 above is specified, in (1) to (4), the parameters of the S-N curves considered for assessment of , slopes k_{f1}, k_{f2} and number of cycles at the knee, N*. However, N* does not match with the value specified in the new S-N curves in Annex E, [5], $2 \cdot 10^6$ cycles. The same values than those in [2], Annex NN, are kept.

A solution for this apparent inconsistence is found in Note 2, where it is stated: "*Different N* values can be considered as follows:* $\lambda_{s,1,N^*new} = \lambda_{s,1,N^*old}\,(N^*_{old}/N^*_{new})^{1/kf1}$".

For a better understanding of this modification, it must be noticed that the aim of the $\lambda_{s,1}$ factor is to get the damage equivalent stress range, i.e., the stress range that leads to the same damage than that calculated with the Palgrem Miner rule, using the stress range histograms produced by the so-called traffic mixes. A general expression for this damage equivalent

TABLE 6.
$\lambda_{s,1}$ values for simply supported and continuous members of railway bridges [5].

a) simply supported members				b) continuous members (interior span)			
	L [m]	STM	HTM		L [m]	STM	HTM
(1)	≤ 2	0.90	0.95	(1)	≤ 2	0.95	1.05
	≥ 20	0.65	0.70		≥ 20	0.50	0.55
(2)	≤ 2	1.00	1.05	(2)	≤ 2	1.00	1.15
	≥ 20	0.70	0.70		≥ 20	0.55	0.55
(3)	≤ 2	1.25	1.35	(3)	≤ 2	1.25	1.40
	≥ 20	0.75	0.75		≥ 20	0.55	0.55
(4)	≤ 2	0.80	0.85	(4)	≤ 2	0.75	0.90
	≥ 20	0.40	0.40		≥ 20	0.35	0.30
c) continuous members (end span)				**d) continuous members (intermediate support area)**			
	L [m]	STM	HTM		L [m]	STM	HTM
(1)	≤ 2	0.90	1.00	(1)	≤ 2	0.85	0.85
	≥ 20	0.65	0.65		≥ 20	0.70	0.75
(2)	≤ 2	1.05	1.15	(2)	≤ 2	0.90	0.95
	≥ 20	0.65	0.65		≥ 20	0.70	0.75
(3)	≤ 2	1.30	1.45	(3)	≤ 2	1.10	1.10
	≥ 20	0.65	0.70		≥ 20	0.75	0.80
(4)	≤ 2	0.80	0.90	(4)	≤ 2	0.70	0.70
	≥ 20	0.35	0.35		≥ 20	0.35	0.40

STM standard traffic mix
HTM heavy traffic mix
(1) Reinforcing steel, pre-tensioning (all), post-tensioning (tendons in plastic ducts and straight tendons in steel ducts); S-N curve with k_{f1}=5, k_{f2}=9 and N*=106 (values may be changed due to changes in k_{f1} and k_{f2} or N*)
(2) Post-tensioning (curved tendons in steel ducts; S-N curve with k_{f1}=3, k_{f2}=7 and N*=106 (values may be changed due to changes in k_{f1} and k_{f2} or N*)
(3) Couplers (prestressing steel); S-N curve with k_{f1}=5, k_{f2}=5 and N*=106 (values may be changed due to changes in k_{f1} and k_{f2} or N*)
(4) Couplers (reinforcing steel); welded bars including tack welding and butt joints; S-N curve with k_{f1}=5, k_{f2}=5 and N*=107 (values may be changed due to changes in k_{f1} and k_{f2} or N*)
NOTE 1 Interpolation between the given L-values according to formula (k.6) [5] may be carried out.
NOTE 2 Different N* values can be considered as follows: $\lambda_{s,1,N^*new} = \lambda_{s,1,N^*old}\,(N^*_{old}/N^*_{new})^{1/k_{f1}}$

stress range, assuming proportionality between bending moment and stresses, is given below (Eq. 15):

$$\Delta_{\sigma_{equ}} = \left(\Sigma_i^n \frac{\Delta\sigma_i^{k_{f1}}}{N^*}\right)^{1/k_{f1}} \tag{15}$$

Where Δ_{σ_i} is the stress range of block i of the histogram, composed of n blocks. It is immediate to deduce that, keeping the same traffic mixes and therefore the values of $\Delta\sigma_i$, a change in the number of cycles N^* can be accounted for by the expression provided in Note 2 of Table 6.

11. REDUCTIONS OF MATERIALS AND IN-TURN CLIMATE IMPACT

Fatigue verification is not determinant for standard buildings, many road bridges, and several other structures. On the other hand, in railway bridges, support structures for wind turbines and other machinery, offshore structures, etc., fatigue usually have a significant impact in the design and hence in the material amount. Regarding this, the new Eurocode may lead to a non-negligible material volume reduction with the consequent favourable in-turn climate impact. A non-exhaustive summary is provided below.

- Shear reinforcement: Shear reinforcement was sometimes driven by fatigue in railway and even road bridges, as well as foundations for wind turbines and other machinery, mainly due to the significant reduction of the strut angle used at ULS verifications when verifying fatigue and the fatigue strength reduction due to the hook or bent at the links and stirrups. The possibility of a more refined calculation under service conditions of the strut angle, the optimization of the compressive strength reduction of concrete under transverse tensile stresses and the exclusion of the fatigue strength reduction due to the bent for depths larger than 600mm and diameters equal or less than 16mm may lead to local but non-negligible material savings.
- Welded reinforcement. The improvement of the S-N curves for reinforcing bars of diameters less or equal than 12mm, quite common in prestressed structures under dynamic loads will lead to some reduction of the amount of steel reinforcement or will allow the use of tack welding for reinforcement meshes and cages, rationalizing the production, which always has a positive impact in terms of sustainability.
- Fatigue of concrete. With the current formulation thicknesses of some slender webs and slabs of T-girders, box-girders, etc., of concrete railway bridges, and without a doubt thickness of support structures for wind turbines, which are driven by the fatigue verification of concrete under compression, can be reduced. The new formulation improves the compressive fatigue behaviour of concrete,

according to the trend in modern codes and standards such Model Code 2010 [3].

As an example, fatigue strength of concrete C50/60 in [1], yields a $f_{cd,fat}$ of 22.67 Mpa, 45% of f_{ck}. Stress limit to avoid non-linear creep of concrete under the quasipermanent combination of actions is, according to [1], 0.45·fck. This means that stress control is determined in several cases by the fatigue verification provided in [1], not by the SLS verifications. Several precast and even in situ concrete bridges and other structures with decades in service may not comply with the current limits of [1]. A modification of this fatigue verification of concrete under compressive stress was mandatory and this improvement is consistent with the lack of fatigue related pathologies in bridges and support structures for wind turbines. For instance, in High-Speed Railway bridges built in Spain, some of them with more than 40 years in service, no concrete fatigue related pathologies are known by the authors.

Impact in concrete volume reduction for bridges will not be large, but some optimizations could be expected, especially for precast elements. This optimization will be more certain in case of support structures for wind turbines, although the application of [3] for the design of this structures has already led to important reductions of the concrete amount of these members. This conclusion could be extrapolated to other dynamically loaded structures, from crane bridge beams to offshore structures.

- Refined tools. The extension of the tools for the refined assessment of fatigue, such the Palgrem-Miner rule, now just provided for bridges in [2], will also contribute to a more rational design of other structures cyclically loaded, allowing the optimization of the volume of materials.

12.
CONCLUSIONS

This paper has carried out an in-depth review of the most relevant changes in the field of concrete fatigue in the new Eurocode 2 compared to its predecessor.

The first difference, more formal than technical but relevant in any case, is that in the new Eurocode 2 fatigue has its own chapter and an annex, which gives it a visibility and relevance that it did not have in the previous version of this standard. This is a clear demonstration of the importance that fatigue in concrete has acquired in recent years.

From a technical point of view, the most important change between the new Eurocode 2 and its predecessor lies in the improvement of the S-N curves of concrete in compression. After several decades of designing and building structures subjected to significant cyclic loading, mainly bridges and viaducts for railways and support structures for wind turbines, the virtual absence of pathologies is a clear indication that the current formulation was overly conservative. The new Eurocode 2 proposes curves that are more in line with reality (that is, less conservative), which will make it possible to optimize the design of structures, reducing their cost and increasing their sustainability.

Other tools, strongly supported by research and professional practice, have also been introduced to help optimize fatigue design. For example, the introduction of gradient redistribution of concrete under cyclic compressive loading, the possibility of not checking shear reinforcement hooks for fatigue if they meet certain requirements, optimization of the angle of the concrete strut to be considered in the verifications, etc.

In other cases, there are no substantial differences between the new Eurocode 2 and its predecessor, for example in the simplified verification of elements without shear reinforcement, multiaxial stress states or in the calculation of equivalent fatigue loads in road and railway bridges. In both cases, it is regrettable that the new standard has not been somewhat more daring, but precisely the correct behaviour of structures subjected to fatigue designed in recent decades has made it advisable to maintain the approach of the previous version.

Finally, it should be noted that one of the major new features of the new Eurocode 2 is the inclusion of an improved approach for fatigue design of joints between concretes of different ages.

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(*) This document is available through the National members at CEN TC250/SC2.