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Comparative Analysis of Concrete Fatigue Calculation Methods and Proposed Recommendations for the Determination of Design Parameters in Wind Turbine Towers

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Abstract: While the theories and methods for designing the steel sections of wind turbine towers are relatively well established, the design of concrete tower sections, particularly the methodologies for concrete fatigue design, varies across different codes and standards. These methodologies often involve complex calculation parameters and formulas, which can be prone to misinterpretation and misapplication. This paper primarily traces the evolution of provisions in the fib Model Code and the Eurocode, offering recommendations for determining concrete mechanical properties, structural analysis methods, and fatigue design approaches. Furthermore, a concrete fatigue calculation example is presented based on an engineering case study. This example illustrates key considerations for selecting critical parameters and applying the relevant calculation formulas. The aim is to provide a more in-depth understanding and improve the application of concrete fatigue design principles.

Keywords: concrete wind turbine tower fatigue; structural analysis; design methodology; code comparison; determination of design parameters

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1 Introduction

Wind turbine towers can be constructed in various structural forms and with various materials, including steel, concrete, timber, and steel–concrete composite structures. Among these, steel–concrete composite structures have been widely adopted because of their favorable mechanical properties and excellent economic efficiency [1,2]. Considering transportation and installation requirements, the concrete portion of steel–concrete composite towers is typically constructed using a segmental precast assembly process. Prestressing is applied after the segments are hoisted into position at the site, and the precast concrete elements are integrated into a monolithic structure. The steel section usually consists of hollow steel tubes connected to the concrete structure via flanges.

Like conventional structural design, tower design must consider both the ultimate limit state (ULS) and the serviceability limit state (SLS). However, unlike traditional structures, wind turbine towers are subjected to cyclic loading, also referred to as fatigue loading, that is transmitted from the wind turbine. Throughout the design lifetime of a wind turbine, the number of these fatigue load cycles can reach hundreds of millions. Generally, fatigue strength verification is required for structures that are subjected to more than 2×10^4 load cycles. Consequently, fatigue loading often becomes a pivotal controlling factor in tower design, necessitating detailed analysis and calculation.

Numerous researchers have investigated the fatigue performance and underlying calculation principles for concrete structures [3]. Based on experimental data, they summarized the fatigue characteristics and influencing factors of concrete and briefly outlined the treatment of fatigue in various international codes. However, most of these studies are confined to traditional structural forms such as highway and railway bridges and crane girders and do not extend to wind turbine tower structures. Furthermore, the majority of the research tends to be theoretically oriented and often fails to address practical considerations in engineering design.

With respect to the fatigue design of tower structures, different design codes prescribe different methodologies and impose different design requirements for steel and concrete materials. Within the wind energy industry, the fib Model Code and the Eurocode for structural design are typically adopted as the primary basis for fatigue design. Notably, these codes undergo a process of evolution and updating. Consequently, the specific parameters and design requirements can differ significantly between different versions of the same code.

The Model Code (MC) [4,5], published by *fib*, represents a comprehensive code system for the design of concrete structures. Its scope encompasses the entire life cycle of concrete structures, covering design principles, materials, interface characteristics, design, construction, maintenance, and demolition. In the 1990 version (hereafter referred to as MC90) [4], provisions related to fatigue were placed within the chapter on the ultimate limit state (ULS), alongside verifications for structural members under basic stress, buckling, and disturbances. The fatigue-related content in MC90 was already quite complete, incorporating three methods for calculating different levels of refinement. Subsequently, *fib* released the Model Code 2010 version (hereafter referred to as MC10) [5], which introduced certain adjustments to the content structure. Although fatigue verification remained part of the design module, its distinct second-level section heading, namely, "Verification under Nonstatic Loading," was used to differentiate it from verification under static loading. Notably, although the fatigue verification methods in MC10 did not significantly differ from those in MC90, substantial modifications were made to some calculation parameters and formulas, such as those for the nominal compressive fatigue strength and the compressive fatigue life of the concrete. These differences must be carefully distinguished in practical applications.

The Eurocode (Eurocode 2: Design of Concrete Structures) [6,7] first released its official version in 2004. In the 2004 version of the Eurocode (hereinafter referred to as EN04) [6], fatigue design is presented as part of the ultimate limit state (ULS) design. The code provides requirements for fatigue load combinations, along with fatigue verification methods for conventional reinforcement, prestressing steel, and concrete materials. With respect to the fatigue verification approach, this version of the code primarily employs the concept of equivalent fatigue loading, which specifies corresponding stress limits for the respective materials.

The Eurocode (hereinafter referred to as EN23) [7] underwent a comprehensive update in 2023, introducing significant adjustments to the organization of its chapters. With respect to fatigue design, it not only dedicates a separate chapter within the main text but also provides additional fatigue verification methods in Annex E. The main chapter first outlines the applicability conditions for fatigue verification, followed by stipulations for fatigue load combinations. The key points for sectional stress analysis are explained, and simplified fatigue verification methods for conventional reinforcement, prestressing steel, and concrete materials are presented. Notably, for concrete materials, the main text provides only a method for verifying compressive fatigue and does not mention methods for testing tensile fatigue or combined tension–compression fatigue.

Annex E of EN23 further provides detailed calculation methods beyond the simplified approach, namely, the equivalent damage stress method and a detailed design method based on the fatigue load spectrum. Notably, since the design method based on the fatigue load spectrum requires calculating the fatigue life of the material, the code also provides S–N curves for concrete materials. The characteristics of these curves reveal that the fatigue life of the concrete is related to both the maximum and minimum stress levels of the stress cycle.

For wind turbine towers, the fatigue design methodology for structural steel sections is relatively well established. Therefore, this paper focuses specifically on the prestressed concrete sections of the tower. Given that wind turbine towers are predominantly subjected to dynamic loading—a key distinction from conventional structures—the prestressed concrete sections are typically designed as uncracked members. By tracing the evolution within the fib Model Code and the Eurocode, this paper elucidates the development of fatigue design approaches in these standards. It systematically examines and elucidates the applicability conditions and key calculation procedures for various fatigue design methods. Furthermore, a practical calculation example based on a real-world engineering project is provided. The overarching aim is to foster a deeper understanding and more effective application of the relevant code provisions.

2 Concrete Materials

Prior to calculating the compressive fatigue of the concrete, the key parameter—the nominal compressive fatigue strength of the concrete, $f_{cd,fat}$—must first be determined. This strength depends highly on parameters such as the age at loading, concrete grade, and type of cement. The calculation formulas provided in different codes [4-7] exhibit slight variations, as shown in Equations (1) to (4):

$$f_{cd,fat,MC90} = 0.85\beta_{cc}(t)[f_{ck}(1-\frac{f_{ck}}{25f_{cko}})]/\gamma_c \tag{1}$$

$$f_{cd,fat,EN04} = k_1\beta_{cc}(t_0)f_{cd}(1-\frac{f_{ck}}{250}) \tag{2}$$

$$f_{cd,fat,MC10} = 0.85\beta_{cc}(t)[f_{ck}(1-\frac{f_{ck}}{400})]/\gamma_{c,fat} \tag{3}$$

$$f_{cd,fat,EN23} = \beta_{cc}(t_0)\frac{f_{ck}}{\gamma_c}\eta_{cc,fat} \tag{4}$$

where:

β_{cc} is a coefficient related to the age of the concrete at the time of fatigue loading;
f_{ck} is the characteristic compressive cylinder strength of the concrete;
f_{cko} is the reference strength, taken as 10 MPa;
f_{cd} is the concrete compressive strength in the design;
γ_c is the partial factor for the concrete, taken as 1.5;
$\gamma_{c,fat}$ is the partial factor for the concrete under fatigue loading, taken as 1.5;
k_1 is a coefficient, typically taken as 0.85;
$\eta_{cc,fat}$ is a coefficient for the strength of the concrete under fatigue loading.

A comparison of the aforementioned formulas reveals that the expressions provided in MC90 and EN04 are virtually identical. In MC10, the modification factor for f_{ck} was adjusted from 1/250 to 1/400, yet the overall formulation remains largely consistent with those in MC90 and EN04. In contrast, the formula in EC23 has a more streamlined form, retaining only three primary influencing factors. Based on the formulas from these respective codes and when the parameter values are identical—specifically, s =0.2 (coefficient related to the cement type) and t_0=28 days (age of the concrete at the time of fatigue loading)—the nominal compressive fatigue strength

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$f_{cd,fat}$ was calculated for concrete of various grades. The results of these calculations are presented in Figure 1.

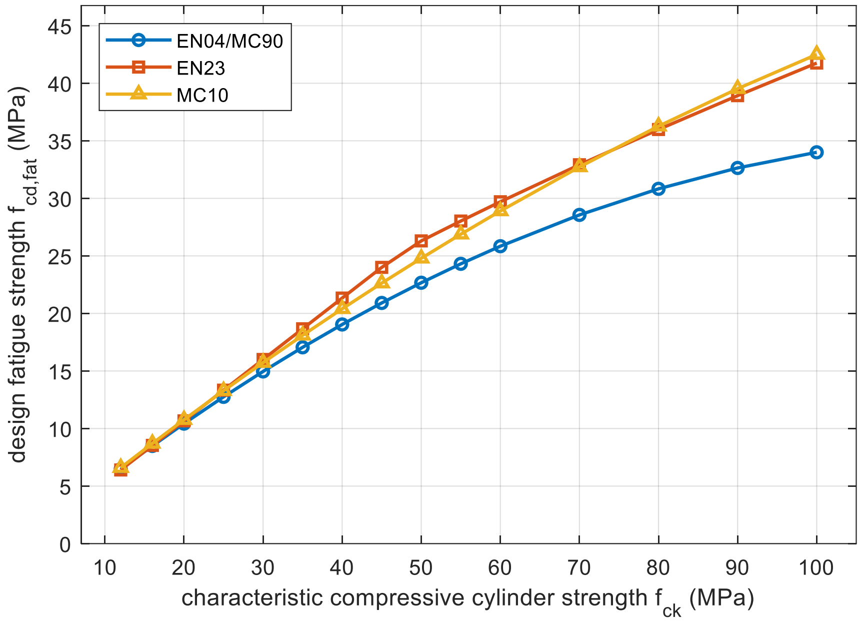

Figure 1 Comparison of the concrete fatigue strength values calculated according to different codes (s =0.2, t_0=28 days)

As shown in the figure, the results calculated using MC90/EN04 are generally significantly less than those obtained from the other two codes. For lower-grade concrete, specifically concrete with grades not exceeding the Chinese standard grade C45, the discrepancy is relatively minor, generally not exceeding 10%. However, as the concrete grade increases, the difference progressively increases, reaching a maximum of approximately 25%. These findings indicate that the earlier codes adopted a relatively conservative approach for determining the nominal compressive fatigue strength of concrete. In contrast, both MC10 and EN23 have increased this strength value, resulting in a notable increase. This adjustment reflects an increased utilization of the fatigue capacity of the material in the newer code provisions.

3 Structural Analysis

For wind turbine towers, the internal forces are typically calculated by the wind turbine manufacturer using specialized simulation software, which is based on site-specific wind data and turbine parameters. The output is usually organized according to the tower segment heights. For fatigue calculations, the results are generally provided as internal force components for each segment height. These results are often presented as Markov matrices for each internal force component, derived using the rainflow counting method, and sometimes converted into equivalent fatigue loads. For the analysis of compressive fatigue in concrete, the Markov matrix for the bending moment (M_y) component should be the primary focus, as shown in Figure 2. This is based on the premise that the M_y component is the dominant factor that induces compressive fatigue in the concrete.

For the M_y component, attention must be given to its sign convention. The windward and leeward sides must be clearly defined to ensure consistency in the stress analysis results. For wind turbine towers, the coordinate system is typically defined as follows (see Figure 3): the x-axis aligns with the prevailing wind direction, the y-axis is perpendicular to the prevailing wind direction, and the z-axis is vertical.

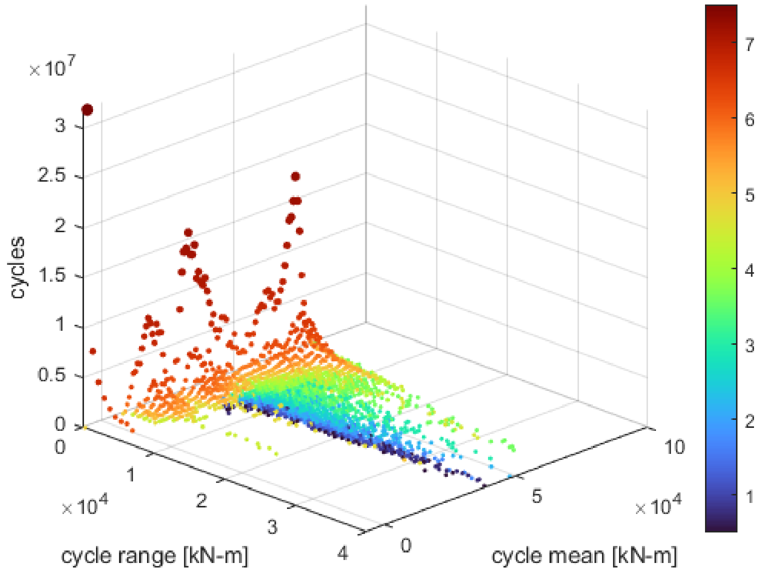

Figure 2 Bending moment component (M_y) of the fatigue load spectrum at a typical cross-section

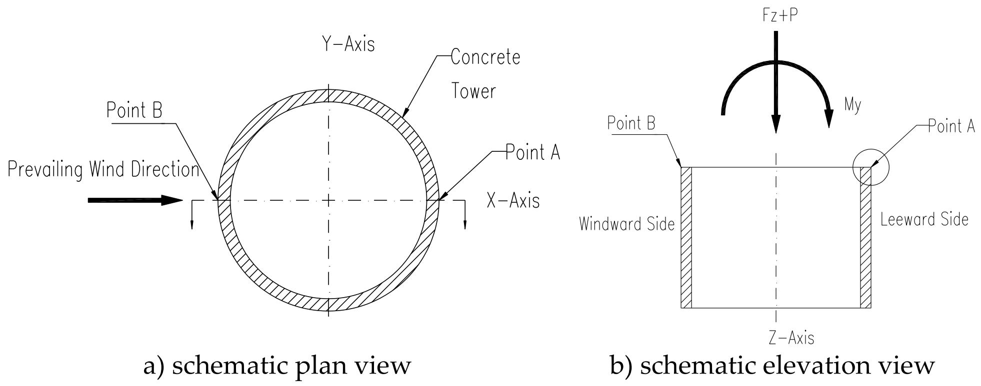

Figure 3 Schematic of the fatigue loading direction and calculation points

Consequently, for any tower cross-section, point B can be defined as the windward side, where a positive M_y moment generates tensile stress. Conversely, point A is defined as the leeward side, where a positive M_y moment induces compressive stress.

Typically, for the Markov matrix of any M_y component, the variation range for the majority of cyclic loads falls within the positive domain (see Figure 2). Therefore, the corresponding point A on the leeward side, subjected to greater compressive stress, often becomes the critical or governing location for the analysis.

Another critical aspect in sectional stress analysis is the selection of sectional geometric properties, which depend on whether the section is cracked. For conventional structures, sections are typically assumed to be cracked, and their geometric properties are calculated based on the transformed section. However, for wind turbine towers, particularly those of concrete or hybrid construction, the occurrence of cracking under cyclic fatigue loading leads to a significant increase in the compressive stress in the concrete and the stress range in both the conventional reinforcement and the prestressing tendons. This would result in substantial fatigue damage and must therefore be prevented. The common practice is to increase the prestressing level to ensure that the horizontal sections of the concrete tower remain uncracked under the combined action of prestressing and fatigue loading, thereby mitigating fatigue damage to both the concrete and the steel components.

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Another issue that requires clarification pertains to the load combinations for fatigue analysis. For wind turbine support structures, fatigue analysis should be conducted using fatigue load cases. In these cases, the structure's self-weight (Fz), prestress (P), fatigue load (predominantly M_y), and thermal effects must be considered. In practice, however, the influence of thermal effects is often neglected in design because of its relatively minor impact. The primary loads considered are typically the self-weight, prestress, and fatigue load. For these load combinations, all partial factors should be taken as 1.0 [8].

4 Design Methods

For the calculation of compressive fatigue in concrete, three primary calculation methods are typically employed, representing different levels of accuracy and applicability conditions: the simplified method, the equivalent load method, and the cumulative damage method (also known as the load spectrum method). For wind turbine towers, which are subjected to an extremely high number of load cycles, the cumulative damage method is generally adopted. The following sections provide an overview of each method.

4.1 Simplified Method

Both MC90 and MC10 specify the applicability conditions for this method, namely, that it is suitable for structures subjected to no more than 10^8 load cycles and relatively low cyclic stress levels. When this method is applied to the compressive fatigue verification of concrete, only the maximum compressive stress level for the relevant load combinations must be calculated. Fatigue verification is deemed satisfactory if this stress level does not exceed a specified limit, which is defined as a certain proportion of the nominal fatigue strength of the concrete (Equation 5).

$$\gamma_{Ed}\sigma_{c,\max}\eta_c \leq 0.45 f_{cd,fat} \tag{5}$$

where:

γ_{Ed} is the fatigue load factor;

$\sigma_{c,\max}$ is the maximum compressive stress;

η_c is the coefficient accounting for the stress gradient.

The primary advantage of this method lies in its simplicity, as it requires only the calculation of the load cycle that induces the maximum internal forces within the fatigue load spectrum. However, it suffers from significant drawbacks for wind turbine support structures. The number of load cycles these structures endure throughout their service life often exceeds 10^8, which calls the applicability of this method into question. Furthermore, the method imposes a stringent limit on the permissible stress level in the structure, allowing the applied stress to reach only 45% of the nominal compressive fatigue strength of the concrete. This severe restriction frequently leads to oversized cross-sections and a substantial increase in concrete material consumption. Consequently, the use of this method for the fatigue design of tower structures is not recommended.

4.2 Equivalent Load Method

The core of this method lies in simplifying the original fatigue load spectrum by transforming it into an equivalent fatigue load with a single constant amplitude while ensuring that the fatigue damage induced in the structure remains equivalent to that caused by the original spectrum. This approach is generally considered suitable for structural materials whose fatigue life is independent of the mean stress level, such as metallic materials. Nevertheless, both EN04 and EN23 still list this method as among the available options for fatigue verification. Specifically, the maximum and minimum equivalent fatigue stresses are calculated and verified using a simplified calculation formula, as shown in Equation 6.

$$\frac{|\sigma_{cd,max,equ}|}{f_{cd,fat}} + 0.43\sqrt{1 - \frac{|\sigma_{cd,min,equ}|}{|\sigma_{cd,max,equ}|}} \leq 1 \tag{6}$$

where:

$\sigma_{cd,max,equ}$ is the upper stress limit of the equivalent damage stress range for 10^6 cycles;

$\sigma_{cd,min,equ}$ is the lower stress limit of the equivalent damage stress range for 10^6 cycles.

Notably, the application of this calculation method presupposes the availability of a clearly defined equivalent fatigue load. While the Eurocode provides a method for calculating equivalent fatigue stresses for highway and railway bridges, it does not specify how this should be handled for wind turbine support structures. Consequently, this method is not recommended.

4.3 *Cumulative Damage Method*

The cumulative damage method considers the complete fatigue load spectrum. It involves performing separate calculations for each stress level within the spectrum: determining the corresponding fatigue life (i.e., the allowable number of cycles) at that specific stress level. The damage is defined as the ratio of the number of applied stress cycles to the allowable number of cycles at that stress level. The individual damage values for all the stress levels over the entire service life of the structure are then summed. The fatigue verification results are ultimately determined by comparing this total cumulative damage with the permissible damage limit. This method is widely regarded as the most accurate among the three fatigue calculation approaches.

$$\sum_{i=1}^{m} \frac{n_i}{N_i} \leq 1 \tag{7}$$

where:

n_i represents the number of applied stress cycles at a specified stress level;

N_i represents the allowable number of cycles at that specified stress level.

For wind turbine support structures, the cumulative damage method is recommended to verify concrete fatigue. However, notably, the formulas for calculating the fatigue life at a single stress level have been revised throughout the evolution process of the code. These changes provide a more in-depth understanding of the concrete fatigue behavior among code developers.

In general, the fatigue life under compressive fatigue loading is governed predominantly by the maximum and minimum stress levels $S_{cd,\max}$ and $S_{cd,\min}$, respectively). MC90, MC10, and EN23 provide corresponding calculation formulas. The core methodology involves comparing the calculated stress at the critical sectional fiber with the nominal compressive fatigue strength of the concrete. The allowable number of cycles (i.e., the fatigue life, N) is then determined based on this ratio.

Following the formulas from the respective codes, the relationship between $S_{cd,\max}$ and the fatigue life N was plotted for typical $S_{cd,\min}$ stress levels, as shown in Figure 4.

As shown in the figure, the MC90 curve exhibits a distinct three-stage characteristic, whereas the curves for MC10 and EN23 are smoother and notably closer to each other. Furthermore, at the minimum stress level of $S_{cd,\min} = 0.8$, the fatigue life corresponding to $S_{cd,\max} = 0.9$ is $10^{3.01}$ cycles for MC90, $10^{4.21}$ cycles for EN23, and $10^{5.05}$ cycles for MC10. Overall, for high-stress-level cycles, the calculated fatigue life according to different codes follows the trend MC10 > EN23 > MC90. The result from MC10 is slightly greater than that from EN23, but the difference is minor; both results are significantly greater than the result calculated using MC90.

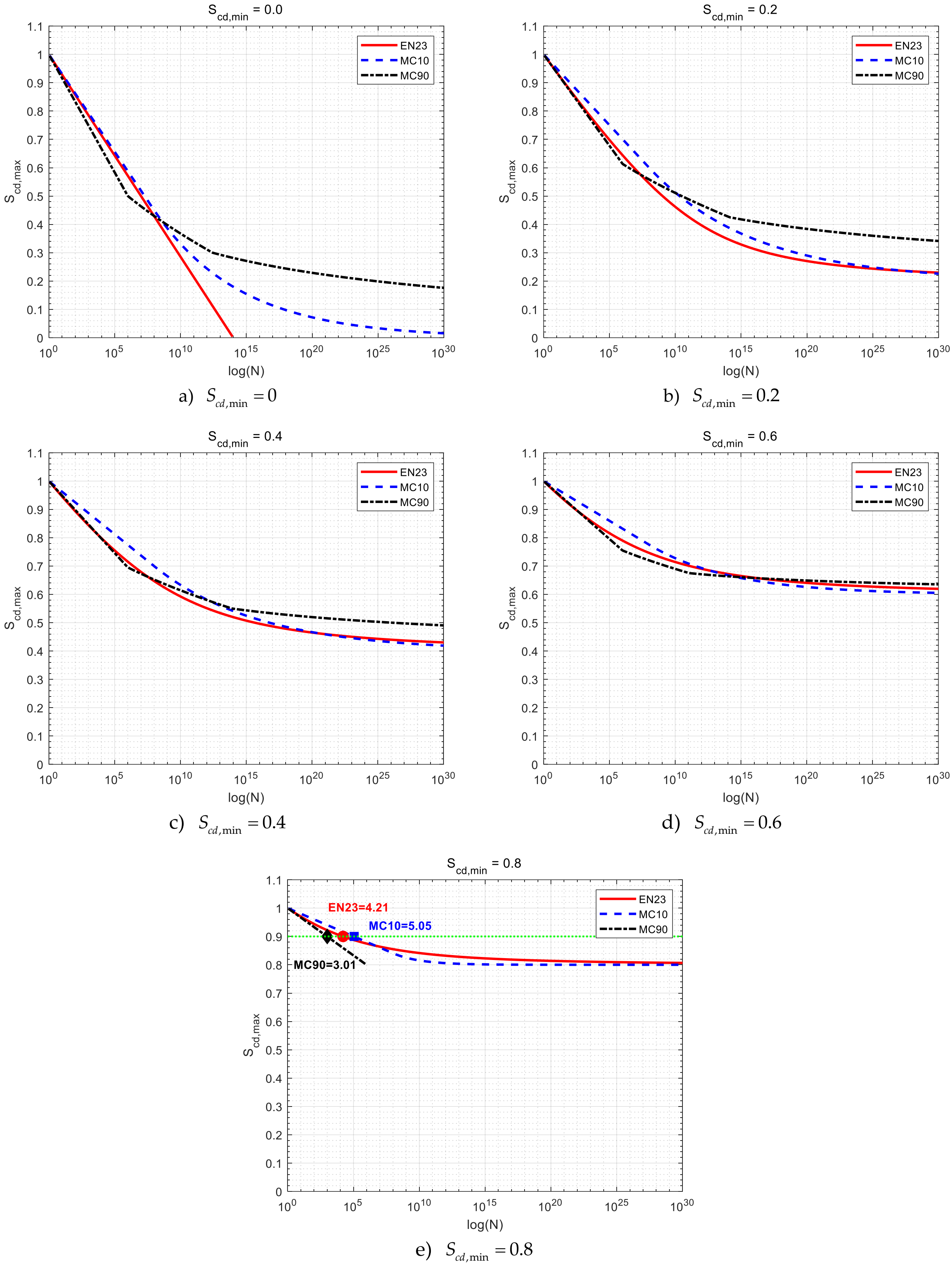

Figure 4 Relationships between the fatigue life and maximum stress level of the concrete at various minimum stress levels ($S_{cd,\min}$)

These findings indicate that even when the cumulative damage method is employed for analyzing concrete fatigue in wind turbine towers, the choice of design code significantly influences the results. For practical engineering applications, Annex A of IEC 61400-6 [9] lists MC90 and DNVGL-ST-0126 [10] as acceptable references for the calculating the fatigue of concrete and reinforcements. Notably, DNVGL-ST-0126 references MC10 and EN04 in its sections related to fatigue calculation. Since DNVGL-ST-0126 was published prior to EN23, the latter has not yet been included. This implies that within the wind energy industry, the use of MC90, EN04, or MC10 as the basis for the fatigue design of concrete sections is recognized. However, considering that codes evolve based on ongoing research and engineering practice, reflecting a progressively deepening understanding of concrete fatigue behavior, the design methods in MC10 should be adopted to achieve more economical design outcomes. EN23, which has been published relatively recently, has not yet undergone extensive validation through engineering practice and is not currently listed as a recognized design basis within the wind industry. Its role at this stage should primarily be for reference and comparative purposes.

5 Engineering Case Study

This section verifies the compressive fatigue of the concrete in a critical section of a concrete tower structure from a real-world project. Based on the cumulative damage method, this case study aims to illustrate the detailed calculation procedure and highlight key considerations.

This project utilizes a steel–concrete hybrid tower. The main structure consists of a prestressed, segmentally precast concrete tower. A steel transition piece and a tubular steel tower section are installed atop the concrete structure, supporting a 5-MW-class wind turbine. Prestressing was applied after the erection of the steel transition piece, integrating the precast segments into a monolithic unit.

The analyzed section is located in the upper-middle region of the concrete tower segment, with an outer diameter of 5400 mm, and is constructed using high-strength C80/95 concrete. In addition to fatigue loads (see Figure 2), this section is subjected to prestressing forces and the self-weight of the upper structure; therefore, the influence of these loads must be considered.

When the nominal compressive fatigue strength of concrete is calculated, two key parameters warrant particular attention: t_0, which is the age of the concrete at the onset of fatigue loading, and s, which is a parameter related to the strength and type of the concrete. t_0 should be determined based on the project's specific schedule, considering the entire process from concrete element production and transportation to installation. Generally, it should not be less than 28 days; in this case study, it is 90 days. The parameter s should be determined based on the concrete mix design, specifically the type of cement used and the strength grade of the concrete; a value of 0.2 is adopted in this case.

Another parameter that requires attention is the magnification factor γ_{Ed} for calculating the fatigue stress level. This factor can be taken as either 1.0 or 1.1. Owing to the high sensitivity of fatigue life to stress level, the choice of this parameter can lead to significant differences in the calculated cumulative damage. According to the Model Code, a value of 1.0 may be used when an accurate stress analysis method is employed; otherwise, 1.1 should be used. However, the code does not explicitly define what constitutes an “accurate stress analysis method”. A value of 1.1 is generally recommended unless additional stress/strain monitoring instruments are deployed to precisely calibrate the theoretical calculations with measured data.

In this section, selected stress cycles are analyzed using the calculation method specified in MC10 to demonstrate the procedure and present the results, as shown in Table 1. In the table, n represents the number of cycles in the load spectrum, while the "mean" and "amplitude" values characterize the variation in the bending moment component M_y. These values are derived from the Markov matrix of loads provided by the wind turbine manufacturer, as referenced in Figure 2.

Table 1 Calculated fatigue damage results for concrete (selected cycles)

n /	Mean (kN·m)	Amplitude (kN·m)	σ_{max} /MPa	σ_{min} /MPa	$S_{cd,max}$ /	$S_{cd,min}$ /	γ /	log$N1$ /	log$N2$ /	log$N$ /	N /	Damage /
23,189	53,200	4,400	-25.24	-24.48	0.701	0.680	0.803	12.13	16.80	16.80	6.4E+16	3.6E-13
18,008	53,200	5,200	-25.31	-24.41	0.703	0.678	0.802	12.02	16.08	16.08	1.2E+16	1.5E-12
11,588	53,200	6,000	-25.38	-24.34	0.705	0.676	0.801	11.90	15.47	15.47	2.9E+15	3.9E-12
5,373	53,200	6,800	-25.45	-24.27	0.706	0.674	0.801	11.79	14.93	14.93	8.6E+14	6.2E-12
2,887	53,200	7,600	-25.51	-24.21	0.708	0.672	0.800	11.67	14.46	14.46	2.9E+14	9.9E-12
3,582	53,200	8,400	-25.58	-24.14	0.710	0.670	0.800	11.56	14.04	14.04	1.1E+14	3.2E-11
5,761	53,200	9,200	-25.65	-24.07	0.712	0.668	0.799	11.45	13.66	13.66	4.6E+13	1.3E-10
2,689	53,200	10,000	-25.72	-24.00	0.714	0.666	0.798	11.34	13.31	13.31	2.1E+13	1.3E-10
2,687	53,200	10,800	-25.79	-23.93	0.716	0.664	0.798	11.23	12.99	12.99	9.9E+12	2.7E-10
1,791	53,200	11,600	-25.86	-23.86	0.718	0.662	0.797	11.12	12.70	12.70	5.0E+12	3.6E-10

In the actual calculation process, the upper and lower limits of the internal force component for a specific cycle must first be determined using the mean value and amplitude of the bending moment component (M_y). Subsequently, based on the sectional geometric properties and by superimposing the compressive stresses induced by the self-weight and prestressing, the upper and lower stress limits (σ_{max} and σ_{min}) at the calculation point (typically on the leeward side; see Figure 3) are derived.

Afterward, the nondimensional upper and lower stress levels, $S_{cd,max}$ and $S_{cd,min}$, are calculated by referencing the nominal compressive fatigue strength of the concrete. The nondimensional parameter γ is then computed. This is followed by the calculation of $\log N1$ and $\log N2$. Depending on whether the value of $\log N1$ is greater than 8, the final value of $\log N$ is determined (using either $\log N1$ or $\log N2$), from which the fatigue life N is obtained.

The damage contribution from this specific stress cycle is then calculated as the ratio of n (the number of applied stress cycles) to N (the allowable number of cycles at that stress level). This procedure is repeated for every stress level defined in the Markov matrix of the M_y component. The individual damage values are summed to obtain the total cumulative damage. This final cumulative damage value is compared with the permissible damage limit to assess whether the section satisfies the concrete compressive fatigue requirement.

The same set of stress cycles was analyzed using different design codes, with the results summarized in Table 2. The calculated damage results from MC10 and EN23 are quite comparable, whereas the damage calculated using MC90 is significantly greater.

This discrepancy is primarily attributed to differences in the nominal compressive fatigue strength specified for the concrete and the associated S–N curves. As referenced in Figure 1, for high-strength C80/95 concrete, the nominal compressive fatigue strength determined by MC90 is markedly less than that calculated by MC10 and EN23, with a difference of approximately 17%. This difference leads to variations in the calculated nondimensional stress level S_{cd}.

Specifically, for the selected cycles in this case study, the stress levels S_{cd} calculated using MC10 and EN23 fall within the range of 0.66 to 0.71. In contrast, the S_{cd} values derived from MC90 range from 0.78 to 0.84. Consequently, consistency in code selection is crucial in practical engineering design to ensure reliable and comparable design outcomes.

Table 2 Comparative analysis of the fatigue damage of the concrete samples calculated according to different codes (selected cycles)

n	Mean	Amplitude	MC10		MC90		EN23	
			N	Damage	*N*	Damage	*N*	Damage
/	(kN·m)	(kN·m)	/	/	/	/	/	/
23,189	53,200	4,400	6.38E+16	3.63E-13	1.79E+05	1.30E-01	1.43E+21	1.62E-17
18,008	53,200	5,200	1.21E+16	1.49E-12	1.49E+05	1.21E-01	2.17E+19	8.31E-16
11,588	53,200	6,000	2.94E+15	3.94E-12	1.25E+05	9.28E-02	7.68E+17	1.51E-14
5,373	53,200	6,800	8.61E+14	6.24E-12	1.04E+05	5.15E-02	4.92E+16	1.09E-13
2,887	53,200	7,600	2.91E+14	9.91E-12	8.73E+04	3.31E-02	4.85E+15	5.95E-13
3,582	53,200	8,400	1.10E+14	3.24E-11	7.30E+04	4.90E-02	6.63E+14	5.40E-12
5,761	53,200	9,200	4.59E+13	1.25E-10	6.12E+04	9.42E-02	1.17E+14	4.93E-11
2,689	53,200	10,000	2.06E+13	1.30E-10	5.13E+04	5.24E-02	2.53E+13	1.06E-10
2,687	53,200	10,800	9.88E+12	2.72E-10	4.30E+04	6.25E-02	6.44E+12	4.17E-10
1,791	53,200	11,600	5.00E+12	3.58E-10	3.61E+04	4.96E-02	1.88E+12	9.52E-10

6 Conclusions

This paper systematically reviews the evolution of relevant design codes concerning the fatigue of concrete in wind turbine towers. A comparative analysis was conducted on the methods specified in different codes for calculating the key material parameter—the nominal compressive fatigue strength of concrete. Furthermore, practical recommendations for structural analysis and fatigue design methodology are provided, drawing on a real-world engineering case study. The primary recommendations are summarized as follows:

(1) Level of prestressing: The selected prestressing level should ensure that all sections remain uncracked under fatigue loading. This is crucial for preventing a significant increase in the stress range resulting from section cracking.

(2) Material and section selection: The use of high-strength concrete is recommended to achieve a higher nominal compressive fatigue strength. Concurrently, the maximum stress level in the section should be limited to no more than 0.9 times the nominal compressive fatigue strength to avoid fatigue failure under high-stress conditions. Furthermore, consistency must be maintained between the values selected for the nominal compressive fatigue strength and the chosen fatigue verification method. Notably, the parameters for calculating this strength differ across codes, and these discrepancies are particularly pronounced for high-strength concrete.

(3) Selection of design method: Given the unique nature of the loads acting on wind turbine towers, the cumulative damage method should be adopted. The calculation formulas provided in the fib Model Code 2010 (MC10) are recommended. When these code formulas are applied, attention must be given to the influence of the concrete age at the onset of fatigue loading (t_0). A realistic value for t_0 should be determined based on the practical project schedule, encompassing production, transportation, and installation processes.

A correct understanding of the fatigue phenomenon is paramount for achieving reliable design outcomes in wind turbine tower structures. More importantly,

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selecting an appropriate methodology for fatigue verification tailored to the specific characteristics of the engineering project is essential. The objective is to strike an optimal balance: fully utilizing the material's strength capacity while ensuring structural safety and considering economic efficiency. This paper aims to provide practical design guidance to foster a deeper understanding of concrete fatigue among designers and the industry, thereby promoting the correct application of design methods.

Conflict of interest: All the authors disclosed no relevant relationships.

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