



D 2026

MONITORING AND ADVANCED NUMERICAL MODELLING TO SUPPORT ONSHORE WIND TURBINE OPERATION AND FOUNDATION DESIGN

MÁRIO JOÃO MOREIRA SOBRINHO
DOCTORAL THESIS IN CIVIL ENGINEERING
PRESENTED TO THE FACULTY OF ENGINEERING
OF THE UNIVERSITY OF PORTO



Monitoring And Advanced Numerical Modelling To Support Onshore Wind Turbine Operation And Foundation Design

Mário João Moreira Sobrinho

July 2026

Dissertation submitted to the Faculty of Engineering of the University of Porto in fulfilment
of the requirements for the degree of Doctoral of Civil Engineering

Supervisor: Mário Pimentel (Assistant Professor, FEUP)

Co-supervisor: Filipe Magalhães (Associate Professor, FEUP)

ABSTRACT

The rapid expansion of wind energy capacity, driven by the need to mitigate climate change, has led to a continuous upscaling of wind turbine dimensions and to a growing number of wind turbines approaching the end of their service lifetime. These trends impose increasing structural demands on foundations, making it necessary to reassess design methodologies and to develop robust strategies for the safety assessment of existing structures, particularly those that exhibit pathologies at the tower-foundation interface. This thesis addresses these challenges by investigating the structural behaviour of the two most prevalent onshore tower-foundation connection solutions: the prestressed Anchor Cage (AC) and the embedded Tower Foundation Section (TFS).

The research methodology combines extensive experimental campaigns with advanced numerical modelling. Structural health monitoring systems were implemented on operating wind turbines at Serra del Rei and Serra da Lousã wind farms with the aim of acquiring field data, including strains, rotations, and relative displacements between the foundation and the tower. These data were used to validate nonlinear finite element models developed to simulate the complex interaction between the steel tower and the concrete foundation.

The results demonstrate that AC foundations exhibit a stable and predominantly linear response, with high rotational stiffness, even under high load levels. It is shown that standard linear design models are generally adequate for this type of foundation, although advanced nonlinear analysis reveals significant potential for optimisation, specifically enabling a reduction in the reinforcement of the slab and concrete grade in the pedestal, as well as optimization in the AC components. In contrast, the study confirms that TFS foundations are highly susceptible to concrete degradation in the interface region. The formation of circumferential gaps between the embedded steel flange and the surrounding concrete leads to a nonlinear hysteretic moment-rotation response and to a substantial reduction in rotational stiffness. A critical operational threshold is identified: while gaps of 1 mm reduce stiffness without causing immediate failure, gaps of 2 mm trigger localised crushing of the pedestal concrete at the interface with the embedded steel ring at relatively low load levels, requiring the implementation of repair measures.

Finally, the incorporation of damaged TFS foundation models into aero-servo-elastic simulations indicates that, despite local stiffness degradation, the global operational behaviour of the wind turbine is not compromised under the examined conditions. Gaps up to 2 mm produce negligible changes in operating bending moments at the tower base and no measurable shifts in the first two natural frequencies, while tower-top accelerations and horizontal displacements exhibit only minor increases. Overall, the findings support continued operation with monitoring for gaps up to 2 mm, provided that the concrete foundation remains structurally intact.

Keywords: *Onshore wind turbine, Foundation design, Nonlinear finite element analysis, Structural health monitoring, Interface gaps.*

RESUMO

A rápida expansão da capacidade de energia eólica, impulsionada pela necessidade de mitigar as alterações climáticas, conduziu a um aumento contínuo da dimensão dos aerogeradores e a um número crescente de unidades a aproximar-se do fim da sua vida útil. Estas tendências impõem exigências estruturais crescentes às fundações, tornando necessário reavaliar metodologias de dimensionamento e desenvolver estratégias robustas para a avaliação de segurança de estruturas existentes, particularmente daquelas que apresentam patologias na interface torre-fundação. Esta tese aborda estes desafios, investigando o comportamento estrutural das duas ligações torre-fundação "onshore" mais prevalentes: a "Anchor Cage" (AC) pré-esforçada e a solução com secção embebida "Tower Foundation Section" (TFS).

A metodologia de investigação combina campanhas experimentais com modelação numérica avançada. Foram implementados sistemas de monitorização estrutural em turbinas eólicas em operação nos parques eólicos de Serra del Rei e de Serra da Lousã com vista à aquisição dados de campo, incluindo extensões, rotações e deslocamentos relativos entre a fundação e a torre. Estes dados foram utilizados para validar modelos de elementos finitos não lineares desenvolvidos para simular a interação complexa entre a torre de aço e a fundação de betão.

Os resultados demonstram que as fundações AC apresentam uma resposta estável e predominantemente linear, com elevada rigidez rotacional, mesmo para níveis de carregamento elevados. Verifica-se que os modelos lineares usuais de dimensionamento são, em geral, adequados para este tipo de fundação, embora a análise não linear avançada revele um potencial significativo de otimização, nomeadamente permitindo a redução da armadura da laje e da classe de resistência do betão no pedestal, bem como a otimização dos componentes da "anchor cage". Em contraste, o estudo confirma que as fundações TFS são altamente suscetíveis à degradação do betão na região da interface. A formação de folgas entre a flange de aço embebida e o betão envolvente conduz a uma resposta momento-rotação não linear e histerética e a uma redução substancial da rigidez rotacional. É identificado um limiar operacional crítico: enquanto folgas de 1 mm reduzem a rigidez sem provocar falha imediata, folgas de 2 mm desencadeiam esmagamento localizado do betão do pedestal na interface com o anel de aço embebido para níveis de carregamento relativamente baixos, exigindo a implementação de medidas de reparação.

Por fim, a incorporação de modelos de fundações TFS danificadas em simulações aero-servo-elásticas indica que, apesar da degradação local de rigidez, o comportamento operacional global do aerogerador não é comprometido, dentro das condições analisadas. Folgas até 2 mm originam alterações negligenciáveis nos momentos fletores de operação na base da torre e não produzem desvios mensuráveis nas duas primeiras frequências naturais, enquanto as acelerações e os deslocamentos horizontais no topo da torre apresentam apenas aumentos reduzidos. Globalmente, os resultados suportam a continuidade de operação com monitorização para folgas até 2 mm, desde que a fundação de betão permaneça estruturalmente intacta.

Palavras-chave: *Aerogeradore "onshore", Fundações, Análise não-linear por elementos finitos, Monitorização estrutural, Folgas na interface.*

ACKNOWLEDGEMENTS

The author would like to express sincere gratitude to all individuals and institutions whose support, guidance, and collaboration were essential to the completion of this doctoral thesis.

The author would like to express sincere gratitude to Professor Mário Pimental, supervisor of this doctoral research, for continuous guidance, scientific rigor, and constructive criticism throughout all stages of the work. The consistent availability, strategic direction, and encouragement provided were fundamental to the development and completion of the thesis.

The author is also grateful to Professor Filipe Magalhães, co-supervisor, for essential support in the development of the experimental campaigns, as well as for valuable technical discussions, insightful suggestions, and careful review of the research outcomes, which significantly strengthened the quality and clarity of the work.

The author would like to thank Nuno Pinto and Cláudio Ferraz for their kindness and prompt assistance during every experimental campaign, including the planning and execution of in situ tasks, as well as the practical support required to ensure reliable data acquisition.

The author acknowledges INEGI, in particular Eng.^a Silvina Guimarães and Luís Mendes, for the institutional support and operational collaboration that enabled the implementation of the experimental campaigns.

The author thanks EDPR, in particular Eng. Diogo Silva, for financial support for the monitoring campaign.

The author also acknowledges Nadara, in particular Dr. André Graça and Eng. Tiago Lipari, for the support provided during the monitoring campaigns performed at Serra da Lousã, including logistical coordination, technical collaboration, and the sharing of information.

The author is grateful to the Ashes team, in particular Dr. Loup Suja, for exceptional support in the use of the Ashes software, including guidance on model implementation and clarification of technical aspects that were essential for the numerical work.

The author would also like to thank Dr. Sérgio Pereira, Dr. Francisco Pimenta, and Dr. João Pacheco for their availability and generosity in sharing knowledge and experience at critical moments of the research.

À minha mãe, por tudo aquilo que representa para mim, pelo amor, pela dedicação e pelos sacrifícios ao longo da vida, bem como pelos valores e pela força que sempre me transmitiu, que constituíram uma base essencial ao longo de todo o meu percurso e me permitiram chegar até aqui.

Finalmente, e não menos importante, à minha esposa, Helena Felgueiras, pelo amor, pela força e pela dedicação diária, pela paciência e compreensão ao longo desta desafiante jornada, e por ter sido o meu principal apoio nos momentos mais exigentes.

CONTENTS

1	INTRODUCTION.....	29
1.1	Research context	29
1.2	Objectives	32
1.3	Organization.....	32
2	FUNDAMENTALS OF ONSHORE WIND TURBINE BEHAVIOUR AND FOUNDATIONS	35
2.1	General concepts of onshore wind turbines	35
2.2	Control and protection systems.....	38
2.3	Wind turbine aerodynamics	39
2.3.1	Axial momentum theory	39
2.3.2	Angular momentum theory	42
2.3.3	Blade element theory	44
2.3.4	Blade element momentum theory	45
2.4	Turbulence	46
2.5	Design loads and load cases.....	47
2.5.1	Wind conditions	48
2.5.1.1	Normal wind conditions.....	48
2.5.1.2	Extreme wind conditions.....	49
2.5.2	Loads.....	51
2.5.3	Design load cases (DLC)	52
2.6	Foundation typologies	57
2.7	Design criteria for onshore shallow foundations.....	59
3	STRUCTURAL MONITORING OF WIND TURBINES AND FOUNDATIONS.....	61
3.1	Introduction.....	61
3.2	Monitoring campaign at Serra del Rei wind farm.....	61
3.2.1	Description of the Monitoring System.....	62

3.2.2	Data processing	65
3.2.2.1	Strain gauges	65
3.2.2.2	Accelerometers	67
3.2.2.3	Load cells	67
3.2.2.4	Inclinometer	68
3.2.3	Results	69
3.2.3.1	Estimation of the bending moments	70
3.2.3.2	Natural frequencies	74
3.3	Experimental campaign at Serra da Lousã wind farm.....	75
3.3.1	Description of the experimental campaign	76
3.3.2	Data processing	79
3.3.2.1	Strain Gauges	79
3.3.2.2	Accelerometers	79
3.3.2.3	Inclinometers	80
3.3.3	Results	81
3.3.3.1	Phase 1	81
3.3.3.2	Phase 2	84
3.4	Conclusions	88
4	NONLINEAR NUMERICAL MODELLING OF ONSHORE WIND TURBINE FOUNDATIONS	
	91	
4.1	Introduction	91
4.2	Numerical models	93
4.2.1	Modelling strategy.....	93
4.2.2	Constitutive models and material properties	95
4.2.3	Interface properties	99
4.2.4	Loads	100
4.2.5	Solution techniques	100
4.3	Analysis for serviceability conditions	100
4.3.1	General.....	100
4.3.2	Results	102
4.4	Analysis up to failure	103
4.4.1	Crack patterns.....	103
4.4.2	Moment-rotation curves.....	105
4.5	Comparison with experimental evidence	106

4.5.1	Moment-rotation curve	106
4.5.2	Anchor bolt forces	107
4.6	Stress redistribution.....	109
4.6.1	Comparison of stress resultants	110
4.6.2	Reinforcement stresses.....	113
4.6.2.1	Bottom radial direction	114
4.6.2.2	Bottom circumferential direction.....	115
4.7	Conclusions.....	117
5	STRUCTURAL BEHAVIOUR OF DAMAGED FOUNDATIONS.....	119
5.1	Introduction.....	119
5.2	Numerical model.....	121
5.3	Interfaces - Parametric study	122
5.3.1	Moment-rotation curves.....	123
5.3.2	Comparison with experimental evidence	125
5.3.2.1	LVDT sensors.....	125
5.3.2.2	Inclinometers	127
5.4	Analysis for serviceability conditions	128
5.4.1	Crack patterns.....	128
5.4.2	Concrete compressive strains	129
5.4.3	Moment-rotation curves.....	130
5.4.4	Modification of the moment transfer mechanism.....	131
5.5	Analysis under extreme load conditions.....	133
5.5.1	Crack patterns.....	133
5.5.2	Stresses in the TFS	134
5.5.3	Moment-rotation curves.....	135
5.6	Conclusions.....	136
6	INFLUENCE OF GAPS IN TFS FOUNDATIONS ON WIND TURBINE OPERATIONAL RESPONSE.....	139
6.1	Introduction.....	139
6.2	Wind Turbine Modelling	140
6.2.1	WTG description and calibration	140
6.2.2	Wind turbine foundation interface.....	144
6.2.3	Comparison with experimental results.....	146
6.2.4	Parametric study	151

6.3	Gap effect in WTG behaviour under operation	156
6.3.1	Bending moment at the tower base.....	157
6.3.2	Tower accelerations.....	159
6.3.3	Horizontal displacements.....	159
6.3.4	Tower natural frequencies.....	160
6.3.5	Conclusions.....	161
7	CONCLUSIONS.....	163
7.1	Structural Monitoring and Data Interpretation.....	163
7.2	Modelling Strategies and Design Optimisation	164
7.3	Assessment of Damaged Foundations.....	164
7.4	Global Operational Impact	165

LIST OF FIGURES

Figure 1.1 – Wind power installation in GW between 2020 to 2024 (extracted from [2])	29
Figure 1.2 – Average power rating of installed wind turbines in Europe between 2015 to 2024 (adapted from [4])	30
Figure 1.3 – Evolution of onshore wind turbine size between 1980 to 2024 (adapted from [2])	30
Figure 1.4 – Evolution of concrete and reinforcement quantities and cost for onshore shallow wind turbine foundations with increasing rotor diameter. Estimates based on real projects.	31
Figure 2.1 – Schematic representation of: (a) wind turbine with horizontal axis; (b) wind turbine with vertical axis (extracted from [8]).....	35
Figure 2.2 – Rotor and nacelle components of a typical wind turbine (extracted from [9])	36
Figure 2.3 – Layout of a typical onshore wind turbine	36
Figure 2.4 – Terminology for the principal axes of translation and rotation of wind turbine top and blade deflection directions (extracted from [10]).....	37
Figure 2.5 – Blade cross-section geometry and stress resultant convention (extracted from [11])	37
Figure 2.6 – Typical power curve for wind turbines with 5-6 MW rated power.....	39
Figure 2.7 – Slipstream boundary with the actuator disc. Velocity and pressure behaviour along the slipstream boundary due to actuator disc influence (adapted from [13])	40
Figure 2.8 – Evaluation of C_P and C_T with axial induction factor a	42
Figure 2.9 – Stream tube with wake rotation: (a) stream tube model of flow; (b) annular stream tube (adapted from [11])	42
Figure 2.10 – Power coefficient evolution with tip speed ratio. Comparison with Betz limit (adapted from [11])	44
Figure 2.11 – Rotor annular ring and detailed blade element (extracted from [9])	44
Figure 2.12 – Blade element velocity and forces (adapted from [11]).....	45
Figure 2.13 – Turbulence intensity for the normal turbulence model [12]	47
Figure 2.14 – Onshore wind turbine foundation shape: (a) circular shape; (b) octagonal shape.....	57
Figure 2.15 – Connection between steel tower and concrete foundation using the TFS solution: (a) schematic drawing (b) onsite installation	57
Figure 2.16 – Connection between steel tower and concrete foundation using the AC solution: (a) schematic drawing (b) onsite installation	58
Figure 2.17 – Schematic drawings of different types of onshore wind turbine foundations: (a) shallow foundation; (b) pile foundation; (c) shallow foundation on soil reinforced with stone columns; (d) shallow foundation on soil reinforced with inclusions; (e) shallow foundation on exchanged soil; (f) anchored foundation.	58

Figure 3.1 – Serra del Rei Wind Farm. Wind turbine location and photo	62
Figure 3.2 – Monitoring system in the wind turbine of Serra del Rei.....	64
Figure 3.3 – Pictures of the monitoring devices: (a) load cell; (b) inclinometer; (c) accelerometers; (d) strain gauge and temperature sensor; (e) section S1 with strain gauges and temperature sensors; (f) cabinet with the data acquisition system.....	64
Figure 3.4 – Detail of the sinusoidal signal evaluated at the strain gauges as a consequence of the 360° yawing: (a) yaw angle; (b) deformation recorded by the strain gauges over the entire day; (c) detail of the deformation during the 360° yawing.....	66
Figure 3.5 – Deformation zeroed for the 360° yawing for the strain gauges. Comparison between the three 360° yawing occurrences: (a) at section S1; (b) at section S2.....	66
Figure 3.6 – Example of the RMS horizontal acceleration measured by the accelerometers during August 2021.....	67
Figure 3.7 – Post-tension force applied in the anchor bolts: (a) detailed view of the tensioner; (b) main view, including the datalogger.....	67
Figure 3.8 – Detail of the intervention in the anchor bolts detected by the load cells	68
Figure 3.9 – Pedestal rotation during the 360° yawing cycle, showing the yaw angle in the upper plot and the rotation in the x-direction (R_x), y-direction (R_y), and resultant rotation (R_{res}) in the lower plot, with rotation zeroed during yawing.....	69
Figure 3.10 – Complete time series with 10-minute resolution: (a) averaged strain gauges deformation; (b) accelerometers RMS; (c) averaged load cell forces; (d) averaged inclinometer resultant rotation	70
Figure 3.11 – Coordinate system of the tower cross-section with the strain gauges	70
Figure 3.12 – Direction of stress components	71
Figure 3.13 – Resultant bending moment evaluated during 360° yawing at section S1 and S2	72
Figure 3.14 – Maximum 10-minute results: top plot illustrates the wind speed, while the bottom plot illustrates the maximum bending moments at S1 and S2	73
Figure 3.15 – Top plot displays the maximum 10-minute bending moment at section S1, with the design bending moment for the combination SLS QP and GAP, marked by red and red dashed lines, respectively. Bottom plot shows the maximum 10-minute shear force and horizontal force for the combination SLS QP, marked by a red line.....	73
Figure 3.16 – Bending moment evolution with wind speed variation. Comparison between numerical (-) and experimental results (•): (a) evaluation at tower bottom section; (b) evaluation at section S1; (c) evaluation at section S2.....	74
Figure 3.17 – Relationship between normalised bending moment and rotation of the pedestal during WTG operation	74
Figure 3.18 – Campbell diagram of the fore-aft eigenfrequencies identified from accelerometer measurements at the Serra del Rei wind turbine generator: (a) August 2022; (b) April 2023.....	75
Figure 3.19 – Serra da Lousã Wind Farm. Wind turbines location and photo	76
Figure 3.20 – Monitoring system implemented during phases one and two: (a) plan view indicating the location of the four LVDT sensors (D1-D4) relative to the wind direction; (b) schematic vertical section illustrating the location of the LVDT and the measured displacement $\Delta disp$; (c) picture of one installed LVDT sensor (image courtesy of Nadara)	77
Figure 3.21 – Additional monitoring system implemented in phase two.....	78

Figure 3.22 – Pictures of the monitoring devices: (a) Inclinator in the pedestal (b) Inclinator in the pedestal and tower; (c) Strain gauge; (d) Accelerometer; (e) Cabinet with the data acquisition system.....	78
Figure 3.23 – Yaw direction during the event of 360° nacelle rotation and deformation of the tower base captured by the strain gauges	79
Figure 3.24 – Deformation of the strain gauges zeroed for the 360° yawing event	79
Figure 3.25 – RMS of the accelerations measured at the monitored wind turbine in Serra da Lousã .	80
Figure 3.26 – Rotation of (a) pedestal and (b) tower bottom section, during the 360° rotation of the nacelle.....	81
Figure 3.27 – Relative vertical displacement between the tower bottom section and the surface of the pedestal, recorded in 2020: (a) WTG 1; (b) WTG 9; (c) WTG 11	82
Figure 3.28 – Time series with 10-minute resolution for the entire month of December 2023 with averaged strain gauges deformation (top plot), accelerometers RMS (mid plot) and averaged inclinometers rotations (bottom chart) [P – pedestal; T – tower bottom].....	84
Figure 3.29 – Campbell diagram of the fore-aft eigenfrequencies identified from accelerometer measurements at WTG 8 in Serra da Lousã: (a) December 2023; (b) August 2024	85
Figure 3.30 – Coordinate system of the tower cross-section with the strain gauges (E _i) and LVDT sensors (D _i) position (the β angle depends on the rotor yaw)	85
Figure 3.31 – Comparison between bending moment and relative vertical displacement between the pedestal and the tower bottom section during the shutdown events occurred on: (a) 24/10/2023; (b) 11/10/2024.....	86
Figure 3.32 – Location of the LVDT sensors, relative vertical displacements and wind direction during the shutdown events in: (a) 10/2024; (b) 10/2023.....	86
Figure 3.33 – Moment-rotation curves recorded by the inclinometers during the shutdown events that occurred on: (a) 24/10/2023; (b) 11/10/2024.....	87
Figure 4.1 – Simple 2D strut-and-tie model for wind turbine foundation design subjected to bending moment. Dashed lines represent concrete struts and continuous lines the steel ties: (a) AC foundation; (b) TFS foundation (strut ---; tie —).....	91
Figure 4.2 – Wind turbine foundation dimensions (units in meters): (a) AC system; (b) TFS system (Units: m).....	93
Figure 4.3 – Finite element mesh using shell elements with variable thickness	93
Figure 4.4 – AC foundation: (a) finite element mesh; (b) embedded reinforcement finite elements; (c) interfaces between concrete and steel flanges highlighted in red; (d) detail of anchor bolts and flanges	94
Figure 4.5 – TFS foundation: (a) finite element mesh; (b) embedded reinforcement finite elements; (c) interfaces between concrete and steel flanges highlighted in red; (d) interface finite elements with the detail of the horizontal rebars crossing the TFS	95
Figure 4.6 – Stress-strain relationship curves: (a) concrete tensile behaviour (adapted from [38]); (b) concrete compressive behaviour (adapted from [38]); (c) reinforcement behaviour (adapted from [24]).....	96
Figure 4.7 – Reduction factor due to lateral cracking proposed by [41].....	97
Figure 4.8 – Bond stress-slip relationship for monotonic loading (extracted from [23]).....	99
Figure 4.9– Location points for rotational stiffness evaluation: (a) AC foundation; (b) TFS foundation	101

Figure 4.10 – Deformed shapes highlighting the detachment between concrete and steel parts: (a) AC foundation; (b) TFS foundation	101
Figure 4.11 – Comparative analysis of the moment-rotation curves in the AC foundation. Evaluation at: (a) tower base flange; (b) concrete pedestal; (c) slab bottom surface	103
Figure 4.12 – Comparative analysis of the moment-rotation curves in the TFS foundation. Evaluation at: (a) tower base flange; (b) concrete pedestal; (c) slab bottom surface (● detachment from the soil).....	103
Figure 4.13 – AC foundation results for a tower bottom section rotation of $\theta = 2 \times 10^{-3}$ rad ($M/M_{SLS}=2.5$): crack strain (Eknn) pattern; vertical stress (STZ) below slab foundation.....	104
Figure 4.14 – AC foundation results for the last converged solution [$M/M_{SLS}=3.4$]: (a) crack strain (Eknn) pattern; vertical stress (STZ) below slab foundation	104
Figure 4.15 – TFS foundation results for a tower bottom section rotation of $\theta = 2 \times 10^{-3}$ rad ($M/M_{SLS}=2.0$): (a) crack strain (Eknn) pattern; vertical stress (STZ) below slab foundation.....	104
Figure 4.16 – TFS foundation results for the last converged solution [$M/M_{SLS}=2.17$]: (a) crack strain (Eknn) pattern; vertical stress (STZ) below slab foundation	105
Figure 4.17 – AC foundation: Moment-rotation curves evaluated at the T-flange. Comparison between Linear, Interface and NL analyses	106
Figure 4.18 – TFS foundation: Moment-rotation curves evaluated at the steel shell. Comparison between Linear, Interface and NL analyses.....	106
Figure 4.19 – AC foundation rotation with increasing bending moment. Comparison between experimental and numerical results. Numerical model results are presented for: (a) ground E-modulus $E = 1000$ MPa; (b) ground E-modulus values $E = 200$ MPa, $E = 1000$ MPa and $E = 5000$ MPa;	107
Figure 4.20 – Simplified model of the ring flange with the preloaded bolts (adapted from [17]).....	108
Figure 4.21 – Forces in the anchor bolts (AC foundation). Comparison between experimental, numerical and analytical results: (a) force at P1; (b) force at P4	109
Figure 4.22 – Bending moment in the circumferential direction (M_{xx}) for SLS load combination: (a) Linear analysis; (b) Nonlinear analysis	111
Figure 4.23 – Bending moment in the radial direction (M_{yy}) for SLS load combination: (a) Linear analysis; (b) Nonlinear analysis	111
Figure 4.24 – Circumferential force (N_{xx}) for SLS load combination: (a) Linear analysis; (b) Nonlinear analysis.....	112
Figure 4.25 – Radial force (N_{yy}) for SLS load combination: (a) Linear analysis; (b) Nonlinear analysis	112
Figure 4.26 – Bending moment in the circumferential direction (M_{xx}) for ULS load combination: (a) Linear analysis; (b) Nonlinear analysis	112
Figure 4.27 – Bending moment in the radial direction (M_{yy}) for ULS load combination: (a) Linear analysis; (b) Nonlinear analysis	112
Figure 4.28 – Circumferential force (N_{xx}) for ULS load combination: (a) Linear analysis; (b) Nonlinear analysis.....	113
Figure 4.29 – Radial force (N_{yy}) for ULS load combination: (a) Linear analysis; (b) Nonlinear analysis	113
Figure 4.30 – Bottom reinforcement mesh of the foundation, showing the circumferential and radial reinforcement (blue lines). The dashed red lines indicate the paths along which the stresses in the	

reinforcement are evaluated for comparison between linear and nonlinear analyses: the circumferential path at $r = 3$ m (C-C'), and the radial paths at 100° (B-B') and 180° (A-A'). The bottom drawing schematically represents the foundation cross-section to support interpretation.	114
Figure 4.31 – Stresses in the bottom radial reinforcement evaluated along the circumferential path at $r = 3$ m. Comparison between linear analysis (solid lines) and nonlinear analysis (dashed lines) for the SLS and ULS load combinations. The nonlinear results are shown for the reference reinforcement area (100%) and a reduced reinforcement area (50%)	115
Figure 4.32 – Colourmaps of the stresses in the bottom radial reinforcement for the ULS load combination: (a) reference reinforcement area (100%); (b) reduced reinforcement area (50%)	115
Figure 4.33 – Stresses in the bottom circumferential reinforcement. Comparison between linear analysis (solid lines) and nonlinear analysis (dashed lines) for the SLS and ULS load combinations. The nonlinear results are shown for: (a) the radial alignment at $\theta = 100^\circ$ (B-B') and; (b) the radial alignment at $\theta = 180^\circ$ (A-A'), considering the reference reinforcement area (100%) and a reduced reinforcement area (50%)	116
Figure 4.34 – Colourmaps of the stresses in the bottom circumferential reinforcement for the ULS load combination: (a) reference reinforcement area (100%); (b) reduced reinforcement area (50%)	116
Figure 4.35 – (a) Vertical deformation of the foundation for ULS load combination; (b) crack pattern (bottom view) with red arrows illustrating the circumferential traction	117
Figure 5.1 – Concrete spalling at the interface between the steel tower and concrete pedestal (photo on the right is courtesy from Kay Bode)	119
Figure 5.2 – Gap between the embedded T-flange and the concrete above: (a) 1 mm thickness; (b) > 50 mm thickness (extracted from [49])	120
Figure 5.3 – Detail of the gap location beneath the TFS flange	120
Figure 5.4 – Relationship between normal displacement and normal stress used to represent the gap	121
Figure 5.5 – Location of the interface where the bilinear gap law is applied (highlighted in red)	121
Figure 5.6 – TFS foundation mesh	122
Figure 5.7 – Moment-rotation curve of the tower bottom section with varying gap thickness, assuming frictionless and cohesionless interface	123
Figure 5.8 – Moment-rotation curve of the tower bottom section with varying friction angle, assuming cohesionless interface and gap thickness of 1 mm	123
Figure 5.9 – Moment-rotation curve of the tower bottom section with varying cohesion, assuming friction angle of 30° and gap thickness of 1 mm	123
Figure 5.10 – Comparison of the moment-rotation curve at: (1) TFS; (2) pedestal. The analysis considers a cohesion of 80 kPa, a friction angle of 30° , and a gap thickness of 1 mm.	123
Figure 5.11 – Moment-rotation curves obtained from the numerical simulations with decreasing loading cycles, assuming a cohesion of 80 kPa and a friction angle of 30° : (a) gap with 1 mm located at the T-flange top surface; (b) gap with 1 mm located at the T-flange bottom surface; (c) gap with 1 mm distributed as 0.5 mm at the top surface and 0.5 mm at the bottom surface of the T-flange	125
Figure 5.12 – Relative vertical displacements between the tower base section and the concrete pedestal surface obtained from the numerical simulations for two scenarios: one without friction between	

the TFS and concrete, and another with a friction angle of 45° . Results are compared with the experimental measurements from LVDT sensors D1 and D4	126
Figure 5.13 – Location of the measurement points in the numerical model: (a) top view; (b) lateral view	127
Figure 5.14 – Correlation between bending moment and rotation in the pedestal and TFS. Comparison between numerical simulations and experimental observations for the shutdown events on: (a) 24/10/2023; (b) 11/10/2024	127
Figure 5.15 – Crack strain (E_{knn}) pattern: (a) no gap; (b) gap = 1 mm; (c) gap = 2 mm	129
Figure 5.16 – Concrete compressive strain (E_3): (a) gap = 1 mm; (b) gap = 2 mm	130
Figure 5.17 – Moment-rotation curves obtained from the numerical simulations, assuming a cohesion of 80 kPa and a friction angle of 30° between the TFS and the concrete. Comparison between scenarios with no gap, 1 mm gap, and 2 mm gap with the analysis conducted up to 20% above SLS-QP (♦ establishment of contact between the upper surface of the T-flange and the concrete)	131
Figure 5.18 – Concrete compressive stress (S_3): (a) no gap; (b) gap = 2 mm	132
Figure 5.19 – Schematic bending moment transfer mechanisms in the TFS foundation: (a) undamaged foundation (idealised); (b) undamaged foundation (most representative, including shear transfer along the vertical joints T_1 and T_2); (c) damaged foundation (extreme scenario). Schemes are simplified and consider bending moment only	133
Figure 5.20 – Crack strain (E_{knn}) pattern: (a) no gap; (b) gap = 1 mm	134
Figure 5.21 – Stress in the TFS for 2xSLS-QP load combination (Von Mises equivalent stress): (a) no gap; (b) gap = 1 mm	135
Figure 5.22 – Moment-rotation curves obtained from the numerical simulations, assuming a cohesion of 80 kPa and a friction angle of 30° between the TFS and the concrete. Comparison between scenarios with no gap and 1 mm gap, with the analysis conducted up to, approximately, 2xSLS-QP (♦ establishment of contact between the upper surface of the T-flange and the concrete)	136
Figure 6.1 – Wind turbine main view	141
Figure 6.2 – Torque/Speed curve for simple variable-speed control: (a) theoretical curve adapted from [60]; (b) data from SCADA	142
Figure 6.3 – Blade pitch angle function of the average wind speed	142
Figure 6.4 – Bending moment evolution with the wind speed variation. Comparison between the theoretical data and the numerical results	143
Figure 6.5 – Overview of the foundation model incorporated into the software Ashes	144
Figure 6.6 – Comparison of the simulation results from the model in Chapter 5 and the Python-based script, considering a damaged foundation with 1 mm gap located at the top surface of the T-flange, a friction coefficient of 30° , and a cohesion of 80 kPa between the embedded steel tower and the surrounding concrete.	146
Figure 6.7 – Relationship between bending moment and rotation of the tower bottom section. Comparison between numerical simulations and experimental results (inclinometers from 24/10/2023)	148
Figure 6.8 – Relationship between bending moment and rotation of the tower bottom section. Comparison between numerical simulations and experimental results (inclinometers from 11/10/2024)	149

Figure 6.9 – Normalised power spectral density (PSD) estimates of tower acceleration for time windows immediately before shutdown (top row) and during shutdown (bottom row): (a) spectra derived from accelerometer measurements for the 2024 shutdown event; (b) spectra obtained from the corresponding numerical simulation. In each case, the spectral ordinates are normalised by the maximum value of the respective spectrum. Vertical dashed lines indicate the identified first and second natural frequencies, f_1 and f_2	150
Figure 6.10 – Evaluation of the natural frequencies (1 st and 2 nd modes) of the WTG as a function of the foundation interface stiffness.....	152
Figure 6.11 – Evaluation of the horizontal displacement at the tower top as a function of the foundation interface stiffness.....	152
Figure 6.12 – Comparison between Scenario 1 (no gap) and Scenario 2 (1 mm gap with frictionless and cohesionless interface): Moment–rotation relationships (left plot) and time histories of bending moment and rotation at the tower base during the shutdown (right plots)	154
Figure 6.13 – Comparison between Scenario 2 (1 mm gap with frictionless and cohesionless interface) and Scenario 3 (1 mm gap with interface cohesion of 80 kPa and friction angle of 30°): Moment–rotation relationships (left plot) and time histories of bending moment and rotation at the tower base during the shutdown (right plots)	154
Figure 6.14 – Comparison between Scenario 3 (1 mm gap with interface cohesion of 80 kPa and friction angle of 30°) and Scenario 4 (1 mm gap with interface cohesion of 120 kPa and friction angle of 30°): Moment–rotation relationships (left plot) and time histories of bending moment and rotation at the tower base during the shutdown (right plots).....	155
Figure 6.15 – Numerical time histories at tower top during an emergency shutdown, comparing simulations with and without an interface gap: (a) horizontal displacement and (b) horizontal acceleration. Results are shown for the model with gap (W/ GAP) and without gap (W/o GAP)	156
Figure 6.16 – Numerical time histories at tower top during an emergency shutdown for the model including an interface gap, comparing cases with and without interface friction: (a) horizontal displacement and (b) horizontal acceleration. Results are shown for $\phi = 30^\circ$ (with friction) and $\phi = 0^\circ$ (frictionless interface).....	156
Figure 6.17 – Relationship between bending moment and rotation of the tower bottom section. Comparison between undamaged (no gaps) and damaged foundations (gaps with 1 mm and 2 mm)	158
Figure 6.18 – RMS of the accelerations at the tower top section. Comparison between undamaged (no gaps) and damaged foundations (gaps with 1 mm and 2 mm).....	159
Figure 6.19 – Horizontal displacements at the tower top section. Comparison between damaged and undamaged foundation	160
Figure 6.20 – Normalized power spectral density (PSD) estimates of tower acceleration for 10 minute time windows: (a) undamaged foundation; (b) foundation with 1 mm circumferential gaps; and (c) foundation with 2 mm circumferential gaps. In each case, the spectrum is normalized by its maximum ordinate. Vertical dashed lines indicate the identified first and second natural frequencies, f_1 and f_2	161

LIST OF TABLES

Table 2.1 – Basic parameters for wind turbine classes [12]	48
Table 2.2 – Design load cases (extracted from [12]).....	53
Table 2.3 – Partial safety factors for ULS and fatigue (extracted from [12]).....	56
Table 3.1 – Dates of the main installation and system improvement tasks	63
Table 3.2 – Readings provided by the load cells before and after applying the post-tension force in the anchor bolts.....	68
Table 3.3 – Natural frequencies (1 st mode FA) of the wind turbines 01, 09 and 11 monitored with LVDT sensors in 2020.....	83
Table 3.4 – Natural frequencies (1 st mode FA) of the WTG 8 monitored with LVDT sensors in 2023 and 2024	88
Table 4.1 – Characteristic properties of the concrete.....	98
Table 4.2 – Characteristic properties of the steel.....	98
Table 4.3 – Properties for the bond-slip relation between concrete and reinforcement	99
Table 4.4 – Soil interface properties for AC foundation	99
Table 5.1 – Properties of the horizontal and vertical interface between steel and concrete	122
Table 5.2 – Concrete properties.....	128
Table 5.3 – Steel properties	128
Table 6.1 – Tower-foundation interface parameters and corresponding hardening parameters adjusted to the experimental results derived from the inclinometers for the shutdown events occurred in 24/10/2023 and 11/10/2024	147
Table 6.2 – Wind conditions for the simulations.....	148
Table 6.3 – Foundation stiffness according to the friction angle of the interface.....	151
Table 6.4 – Tower-foundation interface parameters and corresponding hardening parameters.....	152
Table 6.5 – Wind conditions adopted in the simulation.....	152
Table 6.6 – Tower-foundation interface parameters and corresponding hardening parameters used in the simulations	157

NOMENCLATURE

Latin symbols

A	Rotor swept area / Cross-sectional area
a	Axial induction coefficient
a'	Tangential induction coefficient
B	Number of blades
c	Chord length
C_d	Drag coefficient
C_l	Lift coefficient
C_p	Power coefficient
C_t	Thrust coefficient
D	Rotor diameter
E	Young modulus
F	Force / Thrust force
f_{ck}	Characteristic compressive cylinder strength of concrete
f_{yk}	Characteristic yield strength of reinforcement
G_c	Compressive fracture energy
G_f	Fracture energy
I	Turbulence intensity
I_{ref}	Expected turbulence intensity
K_φ	Dynamic rotational stiffness
M	Bending moment
P	Power
p	Pressure

Q	Volumetric flow rate / Torque
r	Radius / Radial distance
T	Period
v	Wind speed
V_{ave}	Average wind speed
V_{hub}	Wind speed at hub height
V_{ref}	Reference wind speed
z	Height above ground

Greek symbols

α	Angle of attack
β	Shear retention factor / Yaw angle relative to tower
γ_f	Partial safety factor for loads
γ_{SLS}	Partial safety factor for Serviceability Limit State
θ	Pitch angle
λ	Tip speed ratio
A_l	Longitudinal turbulence scale parameter
ρ	Air density
σ	Standard deviation of wind speed
σ'	Local solidity
σ_l	Turbulence standard deviation
ϕ	Flow angle (angle between relative velocity and plane of rotation)
ω	Angular velocity
Ω	Rotational frequency of the rotor

Abbreviations

AC	Anchor Cage
BEM	Blade Element Momentum
CDZ	Compressive Damage Zone
DLC	Design Load Case
ECD	Extreme Coherent Gust with Direction Change

EDC	Extreme Direction Change
EOG	Extreme Operating Gust
ETM	Extreme Turbulence Model
EWM	Extreme Wind Speed Model
EWS	Extreme Wind Shear
FCM	Fictitious Crack Model
FEM	Finite Element Model
FLS	Fatigue Limit State
FPZ	Fracture Process Zone
IEC	International Electrotechnical Commission
IRENA	International Renewable Energy Agency
LVDT	Linear Variable Differential Transformer
MEMS	Micro Electro-Mechanical Systems
NLFEA	Nonlinear Finite Element Analysis
NREL	National Renewable Energy Laboratory
NTM	Normal Turbulence Model
NWP	Normal Wind Profile
RMS	Root Mean Square
SCADA	Supervisory Control and Data Acquisition
SLS	Serviceability Limit State
TFS	Tower Foundation Section
ULS	Ultimate Limit State
WTG	Wind Turbine Generator

1 INTRODUCTION

1.1 RESEARCH CONTEXT

The imperative to address climate change has accelerated the global transition toward renewable energy sources. The Paris Agreement set a clear target: to limit global temperature rise to below 1.5°C compared to pre-industrial levels. In response, the European Union has committed to reducing greenhouse gas emissions by 55% by 2030 and achieving climate neutrality by 2050. In line with the global efforts, wind technology remains a cornerstone of this transformation, with onshore playing a key role in the capacity growth.

Over the last two decades, wind energy has emerged as one of the leading technologies in the decarbonisation of the power sector. Global wind capacity passed the symbolic threshold of 1 TW in 2023, after a year in which approximately 117 GW of new capacity was installed worldwide, more than 100 GW of which was onshore [1]. Continued strong growth in 2024 added a further 117 GW, bringing cumulative wind capacity to about 1.14 TW by the end of that year [2]. The International Energy Agency reports a similar cumulative figure of around 1.015 TW at the end of 2023 and notes that roughly 93% of this capacity is installed onshore, confirming the dominant role of onshore wind in the current fleet [3]. Figure 1.1 illustrates the wind power installation between 2020 and 2024.

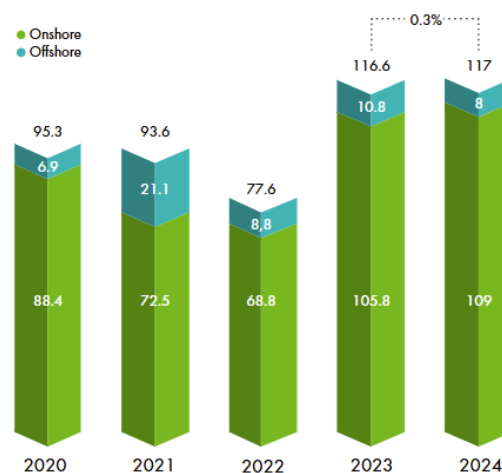


Figure 1.1 – Wind power installation in GW between 2020 to 2024 (extracted from [2])

Within Europe, wind power has become a central pillar of the electricity system. By the end of 2024, Europe had approximately 285 GW of installed wind capacity, of which 248 GW onshore and 37 GW offshore; the EU-27 accounted for about 231 GW, with 210 GW onshore and 21 GW offshore [4]. Projections indicate that the EU-27 is expected to add around 140 GW of new wind capacity between 2025 and 2030, an average of about 23 GW per year, which would raise total EU wind capacity to approximately 351 GW by 2030 [4]. At the global scale, the Global Wind Energy Council estimates that, in order to align with international climate objectives, annual installations of wind power will need to rise from the present level of around 117 GW per year to about 320 GW per year by 2030, with cumulative wind capacity reaching roughly 3 TW by that date [1], [2].

In parallel with this rapid expansion of installed capacity, modern onshore wind turbines have undergone a sustained process of upscaling. In Europe, where the average power rating of new onshore turbines installed in 2023 was about 4.5 MW, up from 4.1 MW in 2022 and from around 2.5 MW a decade earlier (Figure 1.2), the average rating of onshore turbines ordered in 2023 was even higher, about 5.5 MW, indicating that this upscaling trend will continue as these machines are commissioned [4].

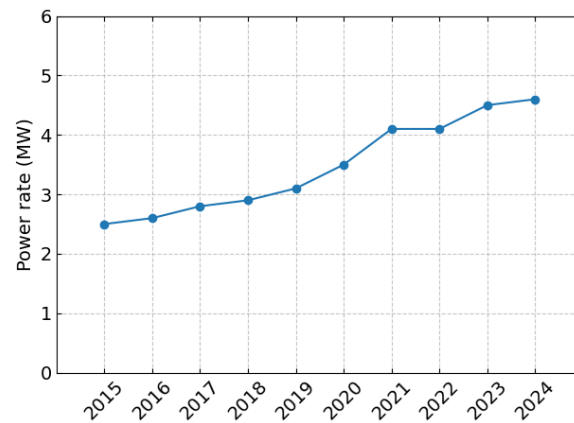


Figure 1.2 – Average power rating of installed wind turbines in Europe between 2015 to 2024 (adapted from [4])

Recent market data show that rotors with diameters greater than 180 m have already become normal, accounting for nearly 60% of global turbine orders in 2024, compared with about 43% in 2023 [2]. Figure 1.3 schematically illustrates the evolution of wind turbine size from 1980, including projections up to 2030.

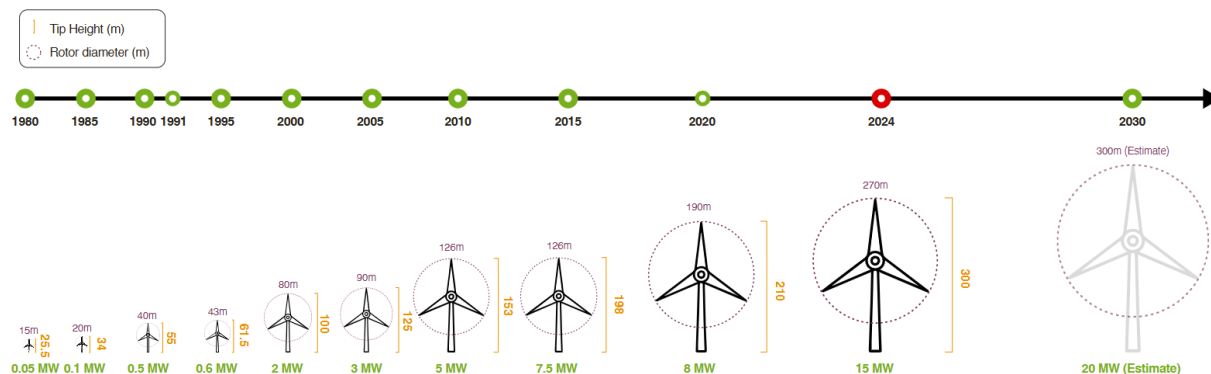


Figure 1.3 – Evolution of onshore wind turbine size between 1980 to 2024 (adapted from [2])

The combination of rapid growth in installed capacity and continued upscaling of turbine size has direct implications for the design of foundations. Taller towers and larger rotors increase the design wind loads, fatigue demands and dynamic effects that must be resisted by the foundation. At the same time, reinforced concrete foundations represent a significant fraction of the material consumption, cost and embodied carbon of onshore wind projects, creating a strong incentive to refine design methodologies and to exploit more fully the structural efficiency of foundation systems [5].

Figure 1.4 illustrates the evolution in the amount of concrete and steel and associated cost for conventional onshore wind turbine shallow foundations, with the rotor diameter increasing, considering the experience of the author on foundation design.

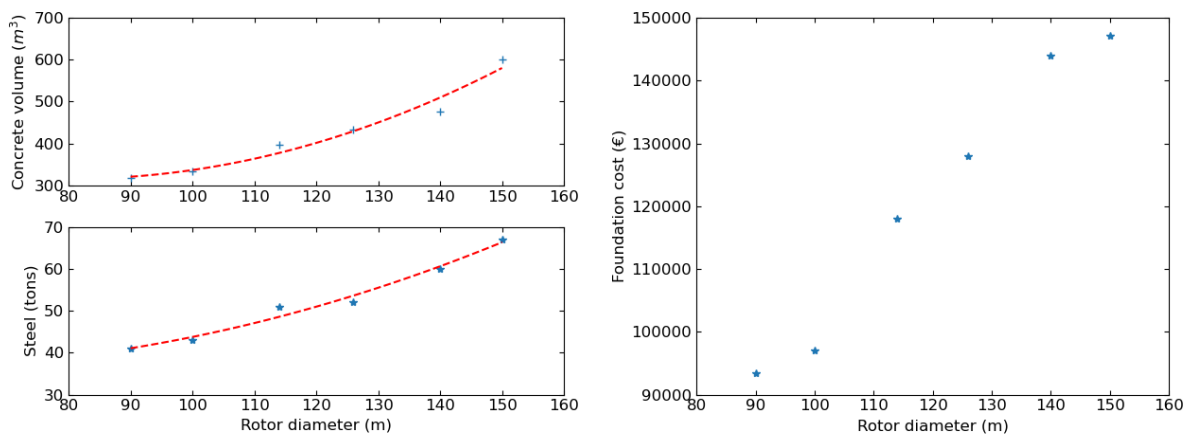


Figure 1.4 – Evolution of concrete and reinforcement quantities and cost for onshore shallow wind turbine foundations with increasing rotor diameter. Estimates based on real projects.

Despite continuous technological advancements in the wind turbines with consequent increase demand at the foundations, the foundation typologies remain essentially unchanged with respect to those adopted more than 20 years ago. Also, the design methodologies generally employed rely on linear elastic analysis based on shell elements with the foundation resting on an elastic bedding. No benefits from force redistribution are taken into account, which could lead to optimization of the flexural and shear reinforcement. Preliminary assessments indicate that potential savings may reach up to 30% in the amount of reinforcement with corresponding reductions of up to 16% in cost and up to 40 t of CO₂ emissions per foundation, assuming the price of the reinforced steel of 1.73€/kg and the CO₂ emissions of 2.49 tCO₂/t [6].

Simultaneously, a significant amount of wind turbines are reaching the end of their lifetime which brings the discussion of the extension of its lifespan. This operation requires the risk assessment of the wind turbine and, of course, the foundation. For the specific case of foundations, the most used solution in the past proved to be problematic, with relative movements between tower and concrete foundation, only justified by the formation of gaps in the connection. Those gaps induce nonlinear behaviour in the rotational stiffness at the tower base, with the amplification of the stress resultants, whose effects in the structural behaviour of the turbine and foundation are not clear. One of the visible effects of these gaps are cracks developed at the concrete pedestal in the interface with the steel tower.

At the same time, there remains limited clarity and no broad consensus regarding appropriate strategies for the safety assessment of existing foundations, particularly in cases where gaps develop between the

embedded steel flange and the concrete. Given the high costs associated with repair works, it is essential to define objective criteria for deciding when a repair intervention should be initiated.

1.2 OBJECTIVES

The continuous growth in wind turbine dimensions imposes increasing structural demands on foundations, rendering some past technical solutions obsolete and necessitating a reassessment of current design methodologies. Concurrently, the ageing of the fleet requires robust risk assessment strategies for extending the operational life of existing assets, particularly those exhibiting interface pathologies.

In this context, the primary aim of this thesis is to enhance the understanding of the structural behaviour of onshore wind turbine foundations through advanced numerical modelling and experimental validation. The specific objectives are defined as follows:

- Development and implementation of structural health monitoring systems: To design and deploy comprehensive instrumentation campaigns on operating wind turbines with different foundation typologies (implemented in Serra del Rei and Serra da Lousã wind farms). This includes the acquisition of reliable field data, such as strains, accelerations, rotations, and relative displacements, to characterise the real-world behaviour of both undamaged and damaged foundations and to provide an empirical basis for model calibration.
- Advanced numerical modelling and design optimisation: To develop and validate nonlinear finite element models using 3D solid elements for the two most common foundation connections: the Anchor Cage (AC) and the Tower Foundation Section (TFS). A key objective is to demonstrate the limitations of standard shell-element design approaches and to quantify the potential for reinforcement optimisation (cost and CO₂ reduction) by accounting for nonlinear stress redistribution mechanisms.
- Characterisation of foundation pathology and damage thresholds: To investigate the structural mechanics of the TFS connection when affected by circumferential gaps between the embedded steel flange and the surrounding concrete. This involves performing parametric studies to define the role of interface properties (friction, cohesion) and establishing objective damage thresholds (e.g., gap thickness limits) that trigger the onset of concrete crushing, thereby defining criteria for repair interventions.
- Assessment of global operational impact: To evaluate the influence of local foundation damage on the global performance of the wind turbine. By integrating nonlinear interface models into aero-servo-elastic simulations, the objective is to quantify the impact of foundation gaps on critical parameters such as natural frequencies, damping ratios, and tower vibrations, ensuring that operational safety limits are maintained.

1.3 ORGANIZATION

This thesis is structured into seven chapters, progressively advancing from fundamental theoretical concepts and experimental monitoring to advanced numerical simulations and global dynamic analyses of wind turbine systems.

Chapter 1 introduces the research context, emphasising the challenges imposed by the continuous upscaling of wind turbines and the management of ageing assets. It outlines the primary objectives of the study and presents the structural organisation of the document.

Chapter 2 establishes the theoretical framework regarding onshore wind turbine behaviour. It provides a comprehensive review of wind turbine aerodynamics, control and protection systems, and the design load cases (DLC) mandated by international standards such as the IEC. Furthermore, this chapter details the prevailing typologies of onshore foundations and the specific design criteria concerning Ultimate Limit State (ULS), Serviceability Limit State (SLS), and Fatigue Limit State (FLS).

Chapter 3 presents the structural monitoring campaigns conducted at Serra del Rei and Serra da Lousã wind farms. This chapter details the design and implementation of the monitoring systems, which include strain gauges, accelerometers, inclinometers, and LVDT sensors. The processing and analysis of the experimental data are discussed, providing the empirical basis used to validate and calibrate the numerical models developed in subsequent chapters.

Chapter 4 focuses on the nonlinear numerical modelling of onshore wind turbine foundations under undamaged conditions. It investigates the structural behaviour of two distinct connection solutions: the Anchor Cage (AC) and the Tower Foundation Section (TFS). The chapter compares standard design methodologies using shell elements against advanced 3D solid element simulations, validating the results with experimental evidence gathered in Chapter 3. Additionally, it addresses stress redistribution mechanisms and their implications for reinforcement design.

Chapter 5 investigates the structural behaviour of damaged foundations, specifically addressing the formation of gaps at the interface between the embedded steel flange and the surrounding concrete in TFS solutions. A parametric study is conducted to analyse the influence of interface properties, such as gap thickness, friction, and cohesion, on the rotational stiffness and crack propagation within the concrete pedestal. This chapter defines the mechanical properties of the damaged interface to be utilised in global analyses of the wind turbine behaviour.

Chapter 6 evaluates the influence of damaged foundations on the global behaviour of the wind turbine under operational conditions. It integrates the nonlinear foundation interface models developed in the previous chapters into a global aero-servo-elastic simulation. This analysis assesses the impact of foundation gaps on critical performance indicators, including tower natural frequencies, displacements, and acceleration responses during operation.

Chapter 7 summarises the main conclusions of the research, integrating the findings from the experimental campaigns and from the numerical simulations. Finally, it provides recommendations for the assessment of existing foundations and identifies promising directions for future research.

2 FUNDAMENTALS OF ONSHORE WIND TURBINE BEHAVIOUR AND FOUNDATIONS

2.1 GENERAL CONCEPTS OF ONSHORE WIND TURBINES

Wind turbines are designed to transform kinetic energy from the wind into electrical energy. The most fundamental feature in the classification of wind turbines is the orientation of the rotor axis (Figure 2.1). In the modern wind turbines, the rotor axis is horizontal (or close to it). The main advantages are:

- Lower cost in energy production due to lower rotor solidity (lower ratio between rotor blade mass and swept area).
- Rotor swept area possible at higher height, which increases the energy production, since the wind speed increases with the increase in height.

Wind turbines with vertical rotor axis, developed in the decades of 1970 and 1980, are rarely used in modern large wind turbines. Despite of some advantages, negative aspects as low tip-speed ratio, the inability of self-start and the difficulty to control the rotor speed [7], led to low interest in this type of solutions. For that reason, only wind turbines with horizontal axis will be discussed in this work.

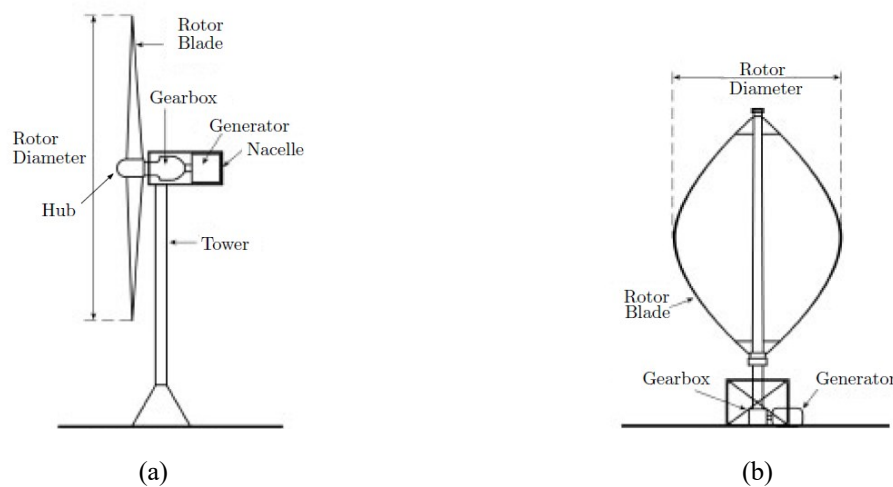


Figure 2.1 – Schematic representation of: (a) wind turbine with horizontal axis; (b) wind turbine with vertical axis (extracted from [8])

The main components responsible to transform kinetic energy into electrical energy are the rotor blades, (usually 3 blades) fixed to the hub that is connected to the gearbox by a low-speed shaft that rotates usually up to 20 rpm. The gearbox increases the rotation speed up to 1800 rpm and is linked to the generator that converts the kinetic energy into electrical energy. Figure 2.2 illustrates the position of the main components described above.

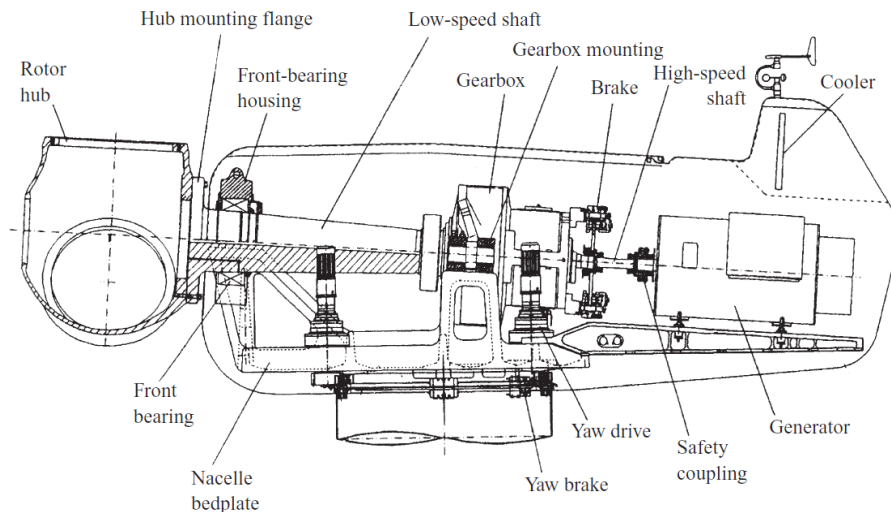


Figure 2.2 – Rotor and nacelle components of a typical wind turbine (extracted from [9])

The nacelle, responsible for housing the shaft, gearbox, generator and yaw system is placed on top of the tower made of steel, reinforced concrete or a combination of both materials. To provide stability to the wind turbine, the tower bottom is linked to the foundation, usually with a circular shape, in reinforced concrete, buried at the ground. Special attention to onshore wind turbine foundations will be given in Section 4. The complete picture of a typical onshore wind turbine is illustrated in Figure 2.3.

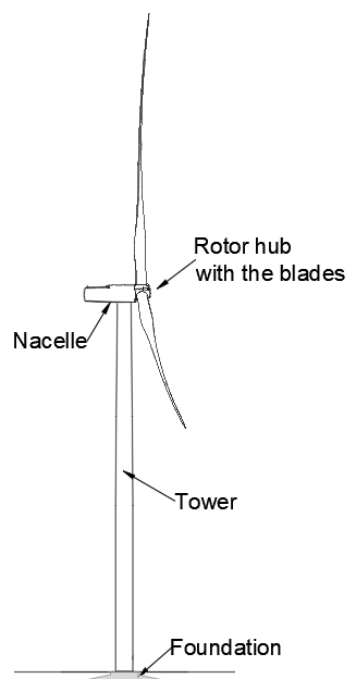


Figure 2.3 – Layout of a typical onshore wind turbine

To better understand the aerodynamics of a wind turbine, some fundamental concepts need to be introduced. Figure 2.4 and Figure 2.5 illustrate, the terminology used to characterize the principal axes of translation and rotation of the wind turbine top, the blade deflection directions, the blade geometry and the stress resultants due to the interaction with the air flow.

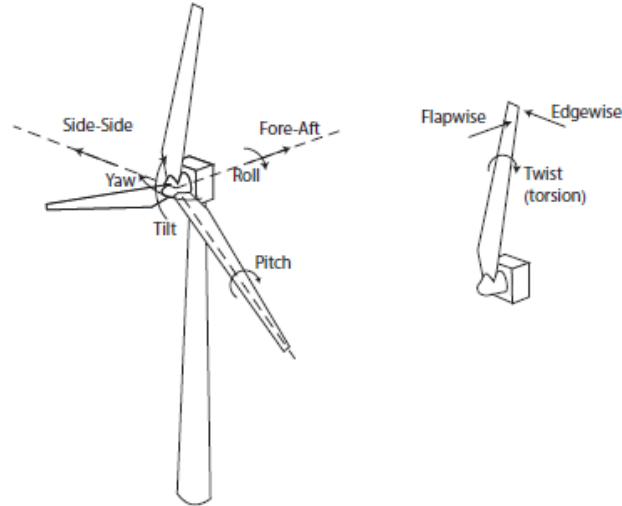


Figure 2.4 – Terminology for the principal axes of translation and rotation of wind turbine top and blade deflection directions (extracted from [10])

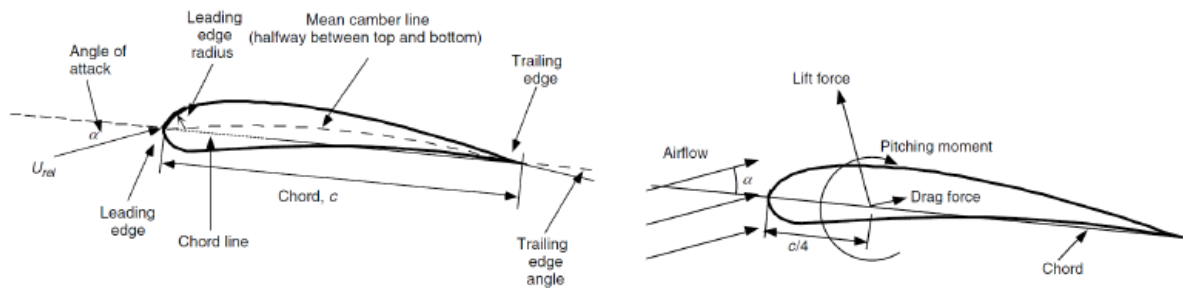


Figure 2.5 – Blade cross-section geometry and stress resultant convention (extracted from [11])

The structural behaviour of wind turbines is driven by dynamics. For that reason, the evaluation of the natural frequencies of the turbines are of extreme importance to avoid resonance phenomena in the structure. During their lifetime, wind turbines experience millions of excitation cycles. The rotor spinning at a given velocity may induce mass imbalance, causing a frequency known as 1P. The effect of the blades passing through the tower induces further excitation, known as 3P (in case of wind turbines with 3 blades). In that sense, wind turbine towers are classified according to the relationship between the tower natural frequency and the exiting frequencies as:

- Soft-soft tower, where the first tower natural frequency is lower than 1P. This results in very flexible structures;
- Soft-stiff tower, with the first tower natural frequency lying between 1P and 3P;
- Stiff-stiff tower, with the first tower natural frequency larger than 3P. This results in rigid structures with consequent cost increase.

2.2 CONTROL AND PROTECTION SYSTEMS

Control and protection systems shall be incorporated within the wind turbine for proper operation and safety.

Control functions

The purpose of the control system is to maximize the power production, while minimizing the loads acting on the wind turbine. Usually, these functions are divided in three main types of controls: (1) control system responsible for the management of the wind turbines within the same wind farm; (2) control system to manage each wind turbine, individually; (3) specific control systems that acts at different wind turbine elements.

One example of control system at the wind farm level is present when the power output to be delivered to the grid connection point is limited and below the wind farm power production capability. In this situation, the control system acts, at the wind farm level, to ensure that the power production will fit the grid requirements, by reducing the power output of all the wind turbines or in specific wind turbines.

At individual level, the control systems are required to govern:

- Power output
- Rotor speed
- Connection of the electrical load
- Cut-in and cut-out wind speed
- Cable twisting due to nacelle yawing
- Alignment to the wind direction

An example of power production improvement in modern wind turbines are the advanced independent pitch controls at each blade that allows the continuously adjustment of the blade orientation. With this feature, in combination with the torque control at the generator, it is possible to adjust the rotor speed as a function of the wind speed. This means that for low wind speed, the production can be maximized and for higher wind speed the loads acting at the turbine can be reduced, maintaining the power production and, reducing the stress at the wind turbine. In Figure 2.6 a typical power curve for a wind turbine within the range 5-6 MW is presented, along with the introduction of the cut-in and cut-out wind speed concepts. The first refers to the lowest wind speed at hub high at which the wind turbine starts to produce energy, and the second one represents the highest wind speed at hub high at which the wind turbine must be shut down to avoid damaging the structure.

Another important control feature responsible for the power production optimization is the yaw control. It allows that the nacelle is continuously oriented with the main wind flow direction. The yaw system is equipped with sensors that detect the wind direction, a rotatable connection between the nacelle and the tower, responsible for the rotation of the nacelle and a braking system to restrict the rotation of the nacelle.

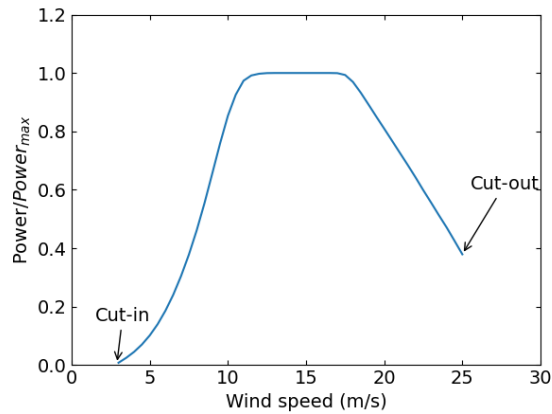


Figure 2.6 – Typical power curve for wind turbines with 5-6 MW rated power

Protection functions

In case of failure, protection systems shall be activated in order to maintain the wind turbine in a safe condition. The protection functions described in [12] that shall be activated are:

- Overspeed
- Generator overload or fault
- Excessive vibration
- Abnormal cable twisting due to yawing of the nacelle.

Detailed description of each individual control and protection system can be found in [9] and [11].

2.3 WIND TURBINE AERODYNAMICS

A wind turbine is a structure used to convert kinetic energy from the wind flow into electrical energy. To understand the result of the extracted kinetic energy, several theories may be applied. In this section, it will be described the axial momentum theory that considers only the energy extraction process to represent the aerodynamic behaviour of the turbine, neglecting the influence of the major components of the turbine. Subsequently, it will be described the angular momentum theory that includes the principal features of the rotor disc in the analysis of the air mass flow. Finally, blade element theory will be described. In this theory, the forces acting on the blades are functions of their shape and orientation.

2.3.1 AXIAL MOMENTUM THEORY

The energy extraction is quantified using the *actuator disc* concept. Figure 2.7 shows the boundary surface (slipstream boundary) and the *actuator disc* that represents the turbine rotor disc assuming the same diameter. Because the mass flow rate is the same everywhere (principle of mass conservation), the presence of the actuator disc leads the stream-tube to expand and the air flow to decrease. One can say that the disc induces change in velocity that must be superimposed on the free stream velocity.

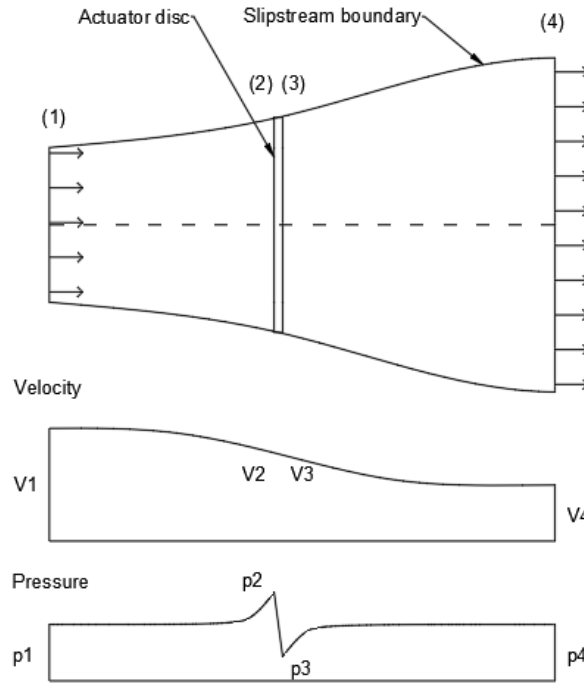


Figure 2.7 – Slipstream boundary with the actuator disc. Velocity and pressure behaviour along the slipstream boundary due to actuator disc influence (adapted from [13])

Assuming that the conditions over each side of the disc are uniform, all the elements of fluid passing through the disc undergo an equal change of pressure. Far upstream, at section 1 the flow is undisturbed with velocity v_1 and pressure p_1 . Immediately before the actuator disc, at section 2, the flow velocity decreases to v_2 and pressure rises to p_2 . Across the actuator disc, the pressure decreases to p_3 , while the velocity is assumed to be kept constant. Downstream, the flow velocity continues to decrease, stabilizing at section 4, where pressure p_4 equals pressure p_1 .

Applying Newton 2.^o law to the described flow behaviour, the force that acts in the *actuator disc* can be determined with the expression (2.1).

$$F = \rho \cdot Q \cdot (v_1 - v_4) = \rho \cdot A_2 \cdot v_2 \cdot (v_1 - v_4) \quad (2.1)$$

where ρ is the air density, Q is the volumetric flow rate through the stream tube and A_2 is the cross-section of the *actuator disc*.

Applying the principle of energy conservation on both sides of the *actuator disc*,

$$p_1 - p_2 = \frac{1}{2} \cdot \rho \cdot (v_2^2 - v_1^2) \quad (2.2)$$

$$p_3 - p_4 = \frac{1}{2} \cdot \rho \cdot (v_4^2 - v_3^2) \quad (2.3)$$

The continuity principle states that v_2 equals v_3 . Combining the expression (2.2) with the expression (2.3), yields the expression (2.4) for the velocity v_2 .

$$v_2 = \frac{(v_1 + v_4)}{2} \quad (2.4)$$

The effect of the actuator disc imposes velocity variation, therefore, at the disc, the velocity can be described as:

$$v_2 = v_1 \cdot (1 - a) \quad (2.5)$$

where a , defined as the induction factor:

$$a = \frac{v_1 - v_2}{v_1} \quad (2.6)$$

At far downstream, the flow velocity is represented by the expression (2.7).

$$v_4 = v_1 \cdot (1 - 2a) \quad (2.7)$$

Combining the expressions (2.1), (2.5) and (2.7) the force that acts in the disc can be calculated using the expression (2.8).

$$F = 2 \cdot \rho \cdot A \cdot v_1^2 \cdot a \cdot (1 - a) \quad (2.8)$$

The power generated is equal to the rate of change of work, resulting in the expression (2.9).

$$P = 2 \cdot \rho \cdot dA \cdot v_1^3 \cdot a \cdot (1 - a)^2 \quad (2.9)$$

Usually, the wind turbine performance is characterized by the power coefficient, C_P and thrust coefficient, C_T . These non-dimensional coefficients are described as:

$$C_P = \frac{\text{Power Extracted}}{\text{Power Available}} = \frac{2 \cdot \rho \cdot dA \cdot v_1^3 \cdot a \cdot (1 - a)^2}{\frac{1}{2} \cdot \rho \cdot A \cdot v_1^3} = 4 \cdot a \cdot (1 - a)^2 \quad (2.10)$$

$$C_T = \frac{\text{Thrust Force}}{\text{Dynamic Force}} = \frac{2 \cdot \rho \cdot A \cdot v_1^2 \cdot a \cdot (1 - a)}{\frac{1}{2} \cdot \rho \cdot A \cdot v_1^2} = 4 \cdot a \cdot (1 - a) \quad (2.11)$$

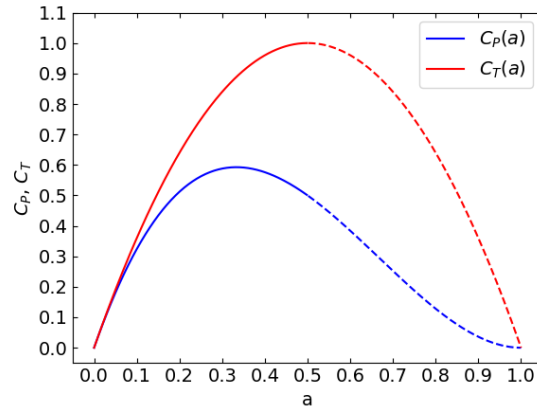
The maximum value of C_P and C_T are determined by differentiating it related to a and set the result equal to zero:

$$\frac{dC_P}{da} = 0 \rightarrow a = \frac{1}{3} \rightarrow C_P = \frac{16}{27} \quad (2.12)$$

$$\frac{dC_T}{da} = 0 \rightarrow a = \frac{1}{2} \rightarrow C_T = 1 \quad (2.13)$$

Figure 2.8 shows the evaluation of both coefficients with the axial induction factor. The maximum achievable power coefficient C_P , known as Betz limit, establishes the upper limit of power that can be extracted from the flow, no matter the wind turbine efficiency.

Expression (2.7) is valid for values of $a < 0.5$, otherwise the wake velocity (velocity in the downstream region) becomes zero or even negative. Under these circumstances, the momentum theory cannot be applied, as no flow mechanism would be possible.

Figure 2.8 – Evaluation of C_P and C_T with axial induction factor a

2.3.2 ANGULAR MOMENTUM THEORY

The previous theory assumes that no rotation was imparted to the flow crossing the rotor. However, air passing through the rotor disc exerts a torque on the rotor with opposite direction of the air in the wake region. Because of the interaction with the rotor, the air downstream rotates, gaining angular momentum and so the air particles have velocity in the tangential direction as well as in the axial direction of the air flow. The acquisition of tangential velocity of the air flow means an increase of kinetic energy that is compensated by a decrease of pressure in the wake, in addition to the one described in Section 2.3.1.

The major difference of this model, when compared with the linear momentum theory, is that the energy extracted from the air flow is lower.

The rotated flow in the wake region can be compared to infinitesimal annular stream tubes with radius r and thickness dr , as illustrated in (Figure 2.9).

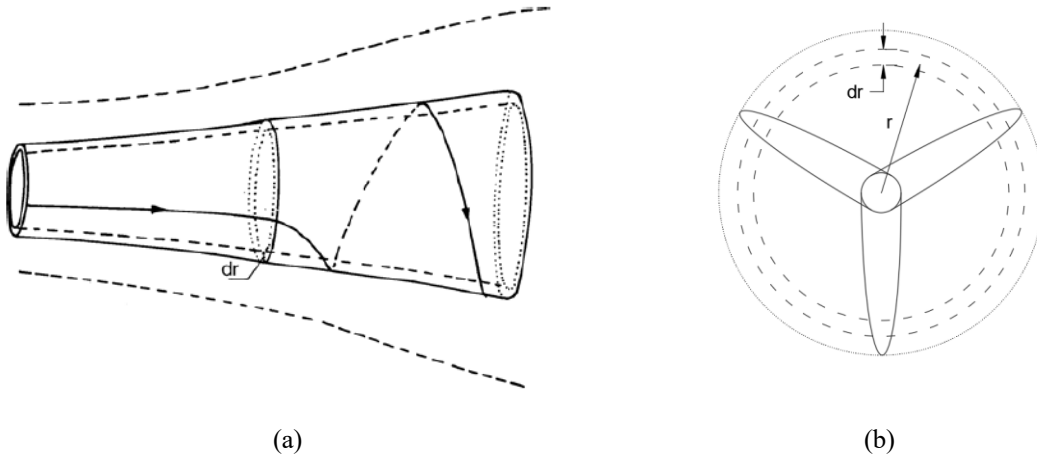


Figure 2.9 – Stream tube with wake rotation: (a) stream tube model of flow; (b) annular stream tube (adapted from [11])

Using the control volume (Figure 2.9) that moves with the angular velocity of the rotor blades (Ω) the energy equation can be applied in the sections before and after the rotor blades to derive an expression

for the pressure difference across the rotor blades [11]. Across the flow disc, the angular velocity of the air flow (ω), relative to the angular velocity of the rotor blades increases from Ω to $\Omega + \omega$.

The pressure difference at the rotor blades is determined according to the expression (2.14).

$$p_2 - p_3 = \rho \cdot r^2 \cdot \omega \cdot \left(\Omega + \frac{\omega}{2}\right) \quad (2.14)$$

Defining the angular induction factor that expresses the change in tangential velocity as:

$$a' = \frac{\omega}{2 \cdot \Omega} \quad (2.15)$$

The resulting thrust at the rotor blades can be determined according to the following set of expressions:

$$\begin{aligned} dT &= (p_2 - p_3) \cdot dA \\ dT &= 4 \cdot a' \cdot (1 + a') \cdot \pi \cdot \Omega^2 \cdot \rho \cdot r^3 \cdot dr \end{aligned} \quad (2.16)$$

with the torque generated in the ring given by expression (2.17).

$$dQ = 4 \cdot a' \cdot v_1 \cdot (1 - a) \cdot \rho \cdot \pi \cdot \Omega \cdot r^3 \cdot dr \quad (2.17)$$

The power output increment at each ring can be described as:

$$dP = dQ \cdot \Omega = 4 \cdot a' \cdot (1 - a) \cdot \lambda_r^2 \cdot (\rho \cdot v_1^3 \cdot \pi \cdot r \cdot dr) \quad (2.18)$$

with the tip speed ratio, λ_r defined as the ratio between the tangential speed, at a given position, and the free stream wind speed:

$$\lambda_r = \sqrt{\frac{\Omega^2 \cdot r^2}{v_1^2}} = \frac{\Omega \cdot r}{v_1} \quad (2.19)$$

The power output extracted from the flow depends not only on a and a' but also on λ_r . The incremental contribution from each annular ring to the power coefficient can be described as:

$$dC_p = \frac{dP}{\frac{1}{2} \cdot \rho \cdot A \cdot v_1^3} = \frac{8}{\lambda^2} \int_0^\lambda a' \cdot (1 - a) \cdot \lambda_r^3 \cdot d\lambda_r \quad (2.20)$$

The integral calculation from the previous expression is graphically represented in Figure 2.10 which also shows the Betz limit. The results shows that the power coefficient increases with the tip speed ratio and tends to the Betz limit.

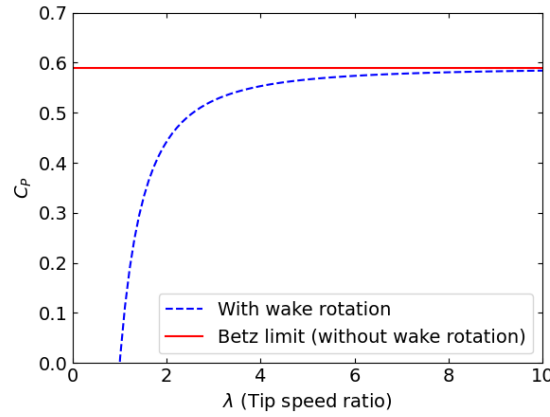


Figure 2.10 – Power coefficient evolution with tip speed ratio. Comparison with Betz limit (adapted from [11])

2.3.3 BLADE ELEMENT THEORY

In the blade element theory, it is assumed that the aerodynamic lift forces are responsible for the rate of change of axial and angular momentum of all the air that passes through the annulus swept. It is also assumed that there is no radial flow and three-dimensional effects are neglected. As a result, the forces acting on the blades can be calculated by dividing the blades in multiple elements, compared to two-dimensional airfoils. Figure 2.11 illustrates the rotor annular ring and details the velocities of the blade element.

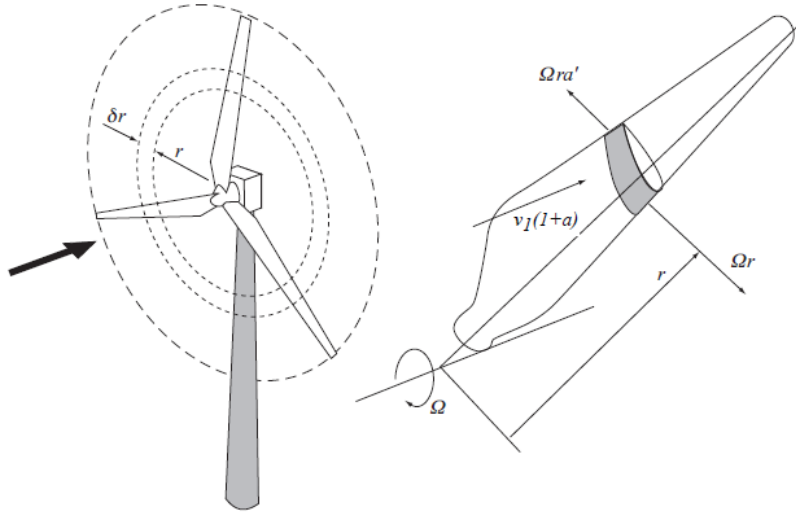


Figure 2.11 – Rotor annular ring and detailed blade element (extracted from [9])

Air flow acting on an airfoil produces a distribution of forces over its surface. The flow velocity increases in the convex surface, resulting in a decrease of pressure. On the concave surface the pressure increases. This pressure difference results in a lift force. Figure 2.12 details the velocities and forces acting on an airfoil section. The lift force is orthogonal to the airflow direction, and the drag force is parallel to the airfoil direction. θ is the pitch angle, α the angle of attack and ϕ the angle between the resultant relative velocity and the plane of rotation of the blade.

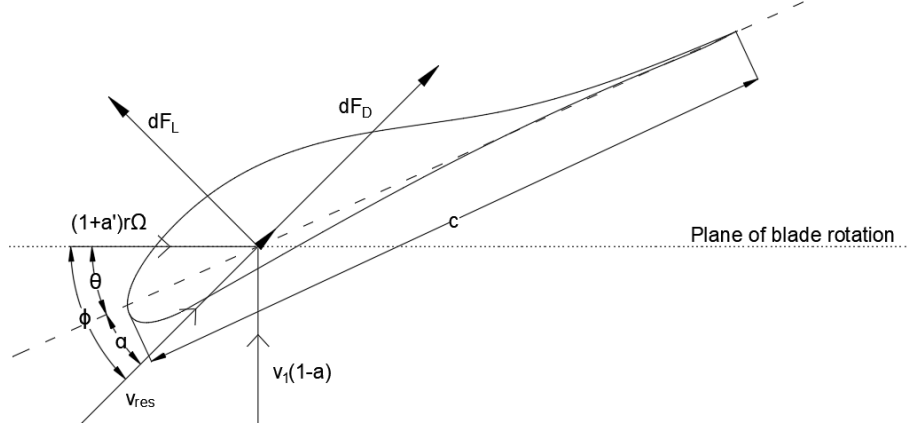


Figure 2.12 – Blade element velocity and forces (adapted from [11])

The net tangential velocity experienced by the blade element is characterized by the expression (2.21) [9].

$$(1 + a') \cdot r \cdot \Omega \quad (2.21)$$

The resultant velocity of the blade element is:

$$v_{res} = \sqrt{v_1 \cdot (1 - a)^2 + [\Omega \cdot r \cdot (1 + a')]^2} \quad (2.22)$$

The lift and drag forces acting at the blade element with length dr are:

$$dF_L = \frac{1}{2} \cdot \rho \cdot v_{res}^2 \cdot c \cdot C_l \cdot dr \quad (2.23)$$

$$dF_D = \frac{1}{2} \cdot \rho \cdot v_{res}^2 \cdot c \cdot C_d \cdot dr \quad (2.24)$$

with the c as the chord length and C_l and C_d as lift coefficient and drag coefficient, respectively.

Finally, the axial thrust and torque on an annular ring are:

$$dT = dF_L \cdot \cos\phi + dF_D \cdot \sin\phi = \frac{1}{2} \cdot \rho \cdot v_{res}^2 \cdot c \cdot B \cdot (C_l \cdot \cos\phi + C_d \cdot \sin\phi) \cdot dr \quad (2.25)$$

$$dQ = (dF_L \cdot \sin\phi + dF_D \cdot \cos\phi) \cdot r = \frac{1}{2} \cdot \rho \cdot v_{res}^2 \cdot c \cdot B \cdot (C_l \cdot \sin\phi + C_d \cdot \cos\phi) \cdot r \cdot dr \quad (2.26)$$

where B is the number of blades.

2.3.4 BLADE ELEMENT MOMENTUM THEORY

The blade element momentum (BEM) theory combines the momentum principle described in sections 2.3.1 and 2.3.2 with the blade element theory described in section 2.3.3, relating the rotor blade geometry with the ability of extracting power from the wind flow.

It was assumed that there is no radial flow interaction between contiguous annuli. Experimental examinations have shown that the radial independence is acceptable [14].

The rate of change of axial momentum resulting from the air passing through the elements that swept each annulus ring is equal to the axial thrust on all elements.

$$\begin{aligned} dT &= 2 \cdot \rho \cdot v_1^2 \cdot a \cdot (1 - a) \cdot 2 \cdot \pi \cdot r \cdot dr = \\ &= \frac{1}{2} \cdot \rho \cdot v_{res}^2 \cdot c \cdot B \cdot (C_l \cdot \cos \phi + C_d \cdot \sin \phi) \cdot dr \end{aligned} \quad (2.27)$$

Applying the same principle to torque, considering the rate of change of angular momentum of the air passing through the elements that swept each annulus ring is equal to the torque on the elements:

$$\begin{aligned} dQ &= 4 \cdot a' \cdot v_1 \cdot (1 - a) \cdot \rho \cdot \pi \cdot \Omega \cdot r^3 \cdot dr \\ &= \frac{1}{2} \cdot \rho \cdot v_{res}^2 \cdot c \cdot B \cdot (C_l \cdot \sin \phi + C_d \cdot \cos \phi) \cdot r \cdot dr \end{aligned} \quad (2.28)$$

Some authors [15] adopted C_d equal to zero in order to calculate the induction factors a and a' . Airfoils with low drag coefficients reveal negligible errors by assuming this simplification [11]. If this assumption is not adopted, the induction factors can be obtained with an iterative process, as described in [16].

Considering C_d equal to zero and after some equation manipulation it is possible to get the following relationships (for more details see [11]):

$$C_l = 4 \cdot \sin \phi \cdot \frac{(\cos \phi - \lambda_r \cdot \sin \phi)}{\sigma' \cdot (\sin \phi + \lambda_r \cdot \cos \phi)} \quad (2.29)$$

$$a = 1 / (1 + 4 \cdot \sin^2 \phi / (\sigma' \cdot C_l \cdot \cos \phi)) \quad (2.30)$$

$$a' = \frac{1}{4 \cdot \frac{\cos \phi}{\sigma' \cdot C_l} - 1} \quad (2.31)$$

with σ' defined as the local solidity:

$$\sigma' = \frac{B \cdot c}{2 \cdot \pi \cdot r} \quad (2.32)$$

2.4 TURBULENCE

In addition to the wind velocity, turbulence plays an important role in the design of wind turbines. Reference [9] defines turbulence as fluctuations in wind speed on a relatively fast timescale, typically less than 10 minutes. Two causes are the main responsible for the turbulence generation: (1) ‘friction’ of the wind with the earth’s surface mainly due to disturbances caused by the topographical features, such as mountains, vegetation and buildings; (2) thermal effects that may cause air masses to move vertically as a result of variation in temperature and therefore in the density of the air.

The turbulence phenomenon is a complex process that can be described by the laws of conservation of mass, momentum and energy, but variables such as temperature, humidity and the motion of the air in three dimensions are also required. With this it is possible to develop a set of differential equations that can predict the turbulence by integrating in time, starting from the initial conditions and subject to certain boundary conditions. Small changes in the definition of the initial conditions or boundary conditions

may result in large differences in the final prediction after relatively short period of time. For that reason, it is useful to develop turbulence models in terms of its statistical properties.

In the bibliography, several statistical descriptors of turbulence can be found, depending on the application, varying from simple turbulence intensities, gust factors, to more detailed descriptions in which the three components of the turbulence vary in space and time, as a function of frequency. Those three components are defined as:

- Longitudinal: along the direction of the mean wind velocity
- Lateral: horizontal and normal to the longitudinal direction
- Upwards: normal to both, the longitudinal and lateral directions

The turbulence intensity is a measure of the overall level of turbulence and is defined by the expression (2.33).

$$I = \frac{\sigma}{\bar{U}} \quad (2.33)$$

where σ is the standard deviation of the wind speed variation about the mean wind speed $\bar{U}$, usually defined over 10 minutes or 1 hour. As the wind speed increases, the turbulence tends to decrease with the wind flow becoming laminar. Figure 2.13 shows the relation between turbulence intensity and wind speed at the hub, considering a normal turbulence model for different wind turbine classes (defined in the next section).

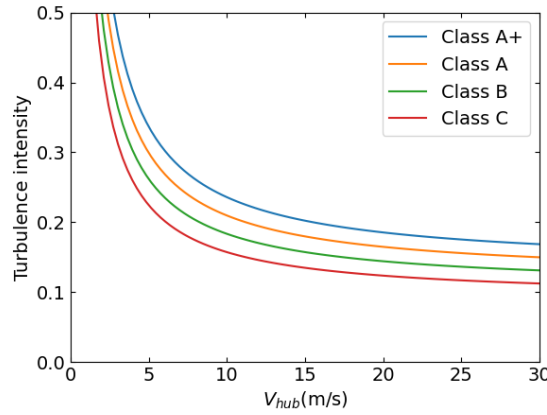


Figure 2.13 – Turbulence intensity for the normal turbulence model [12]

2.5 DESIGN LOADS AND LOAD CASES

Wind resource is highly variable, both geographically and temporally. On a large scale, it depends on the climatic conditions of the globe while in a smaller scale the topography, type of vegetation and buildings have a significant impact in the wind velocity and turbulence.

International regulations such as the IEC standard [12] classify the wind turbines in terms of wind speed and turbulence. Table 2.1 specifies the basic parameters for wind turbine classes at hub height.

Due to the variability of the wind resource, the classes aim to cover the most applications and not specific sites. However, depending on the business case, it may happen that the wind turbines are developed site specific (S class).

Table 2.1 – Basic parameters for wind turbine classes [12]

Wind turbine class		I	II	III	S
V_{ave}	(m/s)	10	8.5	7.5	
V_{ref}	(m/s)	50	42.5	37.5	
$V_{ref,T}$ tropical	(m/s)	57	57	57	
A^+	I_{ref}		0.18		Values specified by the designer
A	I_{ref}		0.16		
B	I_{ref}		0.14		
C	I_{ref}		0.12		

V_{ref} designates the reference wind speed average over 10 minutes, $V_{ref,T}$ the reference wind speed average over 10 min for areas subject to tropical cyclones and I_{ref} the expected turbulence intensity at 15 m/s.

2.5.1 WIND CONDITIONS

The wind conditions considered in the design of wind turbines are divided in normal wind conditions, which occur frequently during normal operation of a wind turbine and extreme wind conditions, defined as having 1-year or 50-year recurrence period. Those wind conditions are a combination of a constant mean flow with, either varying deterministic gust profile or with turbulence.

The following two subsections will introduce the different wind profile models described in the IEC-61400-1 [12] used to evaluate the different load combinations in the wind turbine design. The main goal is to get the reader familiar with the different models, as well as with the relevant parameters that define those models. Detailed description can be found in [9] and [11].

2.5.1.1 NORMAL WIND CONDITIONS

Normal wind conditions can be characterized by two models: (1) normal wind profile model (NWP); (2) normal turbulent model (NTM).

1- Normal wind profile model (NWP):

For standard wind turbines, the normal wind speed profile is given by the power law, defined by the expression (2.34)

$$V(z) = V_{hub} \cdot \left(\frac{z}{z_{hub}} \right)^{0.2} \quad (2.34)$$

where $V(z)$ is the horizontal wind speed at a specified height z and V_{hub} is the reference velocity at the hub height level z_{hub} .

2- Normal turbulence model (NTM)

In the normal turbulence model, the turbulence standard deviation, σ_1 , for standard wind turbine classes is given by the expression (2.35).

$$\sigma_1 = I_{ref} \cdot (0.75 \cdot V_{hub} + 5.6) \quad (2.35)$$

2.5.1.2 EXTREME WIND CONDITIONS

Extreme wind conditions include peak wind speed events due to storms and rapid changes in wind speed and directions as well as severe gust and wind shear events. The IEC-61400-1 [12] classifies the extreme wind conditions in 6 different models, as described below.

1- Extreme wind speed model (EWM)

EWM is classified either a steady or turbulent wind model based on the reference wind speed, V_{ref} , and a fixed turbulence standard deviation, σ_1 .

For the steady model, the extreme wind speed, for a recurrence period of 50 years and 1 year, respectively, shall be determined considering the following expressions:

$$V_{e50}(z) = 1.4 \cdot V_{ref} \cdot \left(\frac{z}{z_{hub}} \right)^{0.11} \quad (2.36)$$

$$V_{e1}(z) = 0.8 \cdot V_{50}(z) \quad (2.37)$$

For the turbulent model, the 10-minute average wind speed for a recurrence period of 50 years and 1 year, respectively, shall be given by the following expressions:

$$V_{50}(z) = \left(\frac{z}{z_{hub}} \right)^{0.11} \quad (2.38)$$

$$V_1(z) = 0.8 \cdot (z) \quad (2.39)$$

with the longitudinal standard deviation given by the expression (2.40).

$$\sigma_1 = 0.88 \cdot V_{hub} \quad (2.40)$$

2- Extreme operating gust (EOG)

The gust magnitude at the hub height is given by the expression (2.41).

$$V_{gust} = \min \left[1.35 \cdot (V_{e1} - V_{hub}); 3.3 \cdot \frac{\sigma_1}{1 + 0.1 \cdot \left(\frac{D}{\Lambda_1} \right)} \right] \quad (2.41)$$

where D is the rotor diameter and Λ_1 is the longitudinal turbulence scale parameter, given by:

$$\Lambda_1 = \begin{cases} 0.7 \cdot z & z \leq 60 \text{ m} \\ 42 \text{ m} & z > 60 \text{ m} \end{cases} \quad (2.42)$$

The wind speed shall be determined with the following expression:

$$V(z, t) = \begin{cases} V(z) - 0.37 \cdot V_{gust} \cdot \sin\left(3 \cdot \pi \cdot \frac{t}{T}\right) \cdot \left(1 - \cos\left(2 \cdot \pi \cdot \frac{t}{T}\right)\right) & \text{for } 0 \leq t \leq T \\ V(z) & \text{otherwise} \end{cases} \quad (2.43)$$

with $T = 10,5$ s.

3- Extreme turbulence model (ETM)

The ETM shall use the NWP and turbulence with the longitudinal component of the standard deviation given by the following expression:

$$\sigma_1 = 2 \cdot I_{ref} \cdot \left(0.072 \cdot \left(\frac{V_{ave}}{2} + 3\right) \cdot \left(\frac{V_{hub}}{2} - 4\right) + 10\right) \quad (2.44)$$

4- Extreme direction change (EDC)

The magnitude of the extreme direction change, θ_e , shall be calculated with the following expression:

$$\theta_e = \pm \tan^{-1} \left(\frac{\sigma_1}{V_{hub} \cdot \left(1 + 0.1 \cdot \left(\frac{D}{\Lambda_1}\right)\right)} \right) \quad (2.45)$$

where σ_1 is given by the expression (2.35) and θ_e is within the interval $\pm 180^\circ$.

The extreme direction change transient, θ_t , is given by:

$$V(z, t) = \begin{cases} 0^0 & \text{for } t < 0 \\ \pm 0.5 \cdot \theta_e \cdot \left(1 - \cos\left(\pi \cdot \frac{t}{T}\right)\right) & \text{for } 0 \leq t \leq T \\ \theta_e & \text{for } t > T \end{cases} \quad (2.46)$$

where $T = 6$ s represents the duration of the extreme direction change. The wind speed shall follow the NWP.

5- Extreme coherent gust with direction change (ECD)

[12] specifies an extreme coherent gust with direction change of

$$V_{cg} = 15 \text{ m/s} \quad (2.47)$$

The wind speed is defined according to the following expression:

$$V(z, t) = \begin{cases} V(z) & \text{for } t < 0 \\ V(z) + 0.5 \cdot V_{cg} \cdot \left(1 - \cos\left(\pi \cdot \frac{t}{T}\right)\right) & \text{for } 0 \leq t \leq T \\ V(z) + V_{cg} & \text{for } t > T \end{cases} \quad (2.48)$$

where $T = 10$ s represents the rise time.

6- Extreme wind shear (EWS)

In the EWS, the following wind speed transients shall be considered:

a) Transient vertical shear (positive and negative):

$$V(z, t) = \begin{cases} V_{hub} \cdot \left(\frac{z}{z_{hub}}\right)^{0.2} \pm \left(\frac{z - z_{hub}}{D}\right) \cdot \left(2.5 + 1.28 \cdot \sigma_1 \cdot \left(\frac{D}{\Lambda_1}\right)^{\frac{1}{4}}\right) \cdot \left(1 - \cos\left(2 \cdot \pi \cdot \frac{t}{T}\right)\right) & \text{for } 0 \leq t \leq T \\ V_{hub} \cdot \left(\frac{z}{z_{hub}}\right)^{0.2} & \text{otherwise} \end{cases} \quad (2.49)$$

b) Transient horizontal shear:

$$V(y, z, t) = \begin{cases} V_{hub} \cdot \left(\frac{z}{z_{hub}}\right)^{0.2} \pm \left(\frac{y}{D}\right) \cdot \left(2.5 + 1.28 \cdot \sigma_1 \cdot \left(\frac{D}{\Lambda_1}\right)^{\frac{1}{4}}\right) \cdot \left(1 - \cos\left(2 \cdot \pi \cdot \frac{t}{T}\right)\right) & \text{for } 0 \leq t \leq T \\ V_{hub} \cdot \left(\frac{z}{z_{hub}}\right)^{0.2} & \text{otherwise} \end{cases} \quad (2.50)$$

where σ_1 is given by the expression (2.35). The two extreme wind shears are not applied simultaneously.

2.5.2 LOADS

The IEC-61400-1[12] divides the loads to be considered in the design of wind turbines as follows:

1. Gravitational and inertial loads

Static and dynamic loads resulting from gravity, vibration and rotation of the wind turbine as well as seismic activity.

2. Aerodynamic loads

Static and dynamic loads resulting from the interaction between the airflow and the wind turbine, as described in section 2.3. These loads are function of the average wind speed, turbulence, wind turbine geometry, air density and interactive effects between airflow and wind turbine.

3. Actuation loads

Actuation loads are the result of control and operation of wind turbines. They result from mechanical faults, yaw and pitch actuation and torque control, resulting in loss of resistance from the generator due to grid losses.

4. Other loads

Ice loads, wake losses, impact loads, etc., that may impact in the design of the wind turbine.

2.5.3 DESIGN LOAD CASES (DLC)

The design load cases shall consider all the loads with a reasonable probability of occurrence, combined with the control and protection systems of the wind turbine. The calculations used to verify the structural integrity of the wind turbine shall consider normal design conditions with normal and extreme events, fault occurrence and transportation, installation and maintenance.

In Table 2.2 are represented the load cases defined in [12]. They are function of the wind conditions, electrical and other external situations that may occur. For every load case it is defined the type of analysis: Ultimate (U) or Fatigue (F), and the partial safety factor to be used in the design, i.e., Normal (N), Abnormal (A) Fatigue (F) or Transportation (T). Normal design loads are expected to occur frequently within the wind turbine lifetime. Abnormal load cases are less probable to occur and correspond to faults, resulting in the activation of the wind turbine protection system. The description of each individual load case is presented below.

Table 2.2 – Design load cases (extracted from [12])

Design situation	DLC	Wind condition	Other conditions	Type of analysis	Partial safety factors
1 - Power production	1.1	NTM $V_{in} < V_{hub} < V_{out}$	For extrapolation of extreme events	U	N
	1.2	NTM $V_{in} < V_{hub} < V_{out}$		F	F
	1.3	ETM $V_{in} < V_{hub} < V_{out}$		U	N
	1.4	ECD $V_{hub} = V_r - 2 \text{ m/s}$, V_r , $V_r + 2 \text{ m/s}$		U	N
	1.5	EWS $V_{in} < V_{hub} < V_{out}$		U	N
2 - Power production plus occurrence of fault	2.1	NTM $V_{in} < V_{hub} < V_{out}$	Control system fault or loss of electrical network	U	N
	2.2	NTM $V_{in} < V_{hub} < V_{out}$	Protection system or preceding internal electrical fault	U	A
	2.3	EOG $V_{hub} = V_r \pm 2 \text{ m/s}$ and V_{out}	External or internal electrical fault including loss of electrical network	U	A
	2.4	NTM $V_{in} < V_{hub} < V_{out}$	Control, protection or electrical system faults, including loss of electrical network	F	F
3 - Start up	3.1	NWP $V_{in} < V_{hub} < V_{out}$		U	F
	3.2	EOG $V_{hub} = V_r \pm 2 \text{ m/s}$ and V_{out}		U	N
	3.3	EDC $V_{hub} = V_r \pm 2 \text{ m/s}$ and V_{out}		F	N
4 - Normal shut down	4.1	NWP $V_{in} < V_{hub} < V_{out}$		U	F
	4.2	EOG $V_{hub} = V_r \pm 2 \text{ m/s}$ and V_{out}		U	N
5 - Emergency shut down	5.1	NTM $V_{hub} = V_r \pm 2 \text{ m/s}$ and V_{out}		U	N
6 - Parked (standstill or idling)	6.1	EWM 50-year recurrence period		U	N
	6.2	EWM 50-year recurrence period	Loss of electrical network connection	U	A
	6.3	EWM 1-year recurrence period	Extreme yaw misalignment	U	N
	6.4	NTM $V_{hub} < 0.7 V_{ref}$		F	F
7 - Parked and fault conditions	7.1	EWM 1-year recurrence period		U	A
8 - Transport, assembly, maintenance and repair	8.1	NTM V_{maint} to be stated by the manufacturer		U	T
	8.2	EWM 1-year recurrence period		U	A

with V_{in} indicating the cut-in wind speed, V_{out} the cut-out wind speed, V_{hub} the wind speed at the hub, V_{ref} the reference wind speed and $V_{r \pm 2}$ the sensitivity analysis of the rated wind speed within the range of ± 2 m/s.

Power production (DLC 1.1 to 1.5)

Power production load cases correspond to normal machine state where the wind turbine is under operation, connected to the grid.

DLC 1.1 is characterized by normal wind turbine operation, considering the normal turbulent wind model (NTM). The analysis of the stability of the structure is done for ultimate loads.

DLC 1.2 is characterized for the same operating conditions as for DLC 1.1. However, this analysis is used to assess the fatigue strength of the materials, considering the fatigue loads.

DLC 1.3 simulates the extreme turbulent conditions (ETM) for ultimate loads, using the normal wind profile model (NWP) and turbulence with longitudinal component standard deviation.

DLC 1.4 simulates gust and direction change (ECD). Intents to represent transient events as gust superimposed with wind direction change that are expected to occur during the wind turbine lifetime.

DLC 1.5 simulates extreme wind shear (EWS). Intents to represent transient wind shear superimposed with the normal wind profile model.

Power production plus occurrence of fault (DLC 2.1 to 2.4)

The load cases belonging to this group represents internal and external faults that may occur during normal power production, including grid losses. In this scenario, the torque generated from the wind flow will no longer be resisted by the generator, leading to rotor acceleration, until the braking systems are activated with consequent rotor load increase.

DLC 2.1 considers normal load events during operation in turbulent wind fields (NTM) with occurrence of control system faults or grid losses.

DLC 2.2 associate rare events as protection system faults or internal electrical faults to normal load events during operation in turbulent wind fields (NTM). Abnormal partial safety factors shall be used.

DLC 2.3 is a combination of extreme operating gust (EOG) with an internal or external electrical system fault, including grid losses. Those combinations are considered a rare event and so, an abnormal partial safety factor shall be used.

DLC 2.4 simulate normal operation conditions with turbulent wind fields (NTM) associated with a fault or grid loss without an immediate shutdown of the wind turbine with fatigue damage resulting from subsequent loads.

Start-up (DLC 3.1 to 3.3)

The load cases in this group represent the events that may occurs between standstill or idling to power production situation.

DLC 3.1 simulate normal load conditions based on the normal wind profile (NWP), during start-up, that may cause fatigue damage.

DLC 3.2 is a combination of extreme operating gust (EOG) with start-up.

DLC 3.3 is a combination of extreme direction change (EDC) with start-up.

Normal shut down (DLC 4.1 to 4.2)

The load cases in this group represent the events that may occur between power production situation to standstill or idling.

DLC 4.1 simulate normal load conditions based on the normal wind profile (NWP), during normal shut down, that may cause fatigue damage.

DLC 4.2 is a combination of extreme operating gust (EOG) with normal shut down.

Emergency shut down (DLC 5.1)

Simulate the emergency shut down when the wind turbine is operating in normal conditions with turbulent wind fields based on the normal turbulence model (NTM).

Parked (DLC 6.1 to 6.4)

In parked design situations, loads are evaluated considering the rotor as standstill or idling. Turbine dynamic response shall be accounted for, despite the wind model used in the analysis.

DLC 6.1 considers the extreme wind speed model (EWM) in the analysis with 50-year recurrence period. A yaw misalignment up to $\pm 15^\circ$, using steady extreme wind model or, mean yaw misalignment up to $\pm 8^\circ$, using turbulent extreme wind model, is permitted if the yaw system is provided with restraint against slippage.

DLC 6.2 assume extreme wind conditions based on the extreme wind speed model (EWM) with 50-year recurrence period combined with grid loss. Yaw misalignment due to wind direction change up to $\pm 180^\circ$ shall be considered in the analysis if no power back up is provided to the control system for a period of at least 6 hours.

DLC 6.3 combine extreme wind conditions based on the extreme wind speed model (EWM) with 1-year recurrence period with an extreme yaw misalignment up to $\pm 30^\circ$, using the steady extreme wind model, or a mean yaw misalignment up to $\pm 20^\circ$, using the turbulent wind model.

DLC 6.4 considers the fatigue damage that may occur to any component of the wind turbine resulting from the non-power production time at different fluctuating loads, considering the normal turbulence model (NTM).

Parked plus fault conditions (DLC 7.1)

Fault conditions combined with extreme wind speed model (EWM) with 1-year recurrence period. Yaw misalignment up to $\pm 180^\circ$ shall be considered, in case of yaw system fault. Abnormal partial safety factor to be used.

Transport, assembly, maintenance and repair (DLC 8.1 to 8.2)

In DLC 8.1 the stated wind conditions and design situations assumed for transport, assembly, maintenance and repair of a wind turbine, shall be compared with the wind conditions considered in the design and included if they can produce significant loading on the turbine. A safety margin between stated conditions and wind conditions considered in the design shall be used. IEC 61400-1 recommend adding 5 m/s to the stated wind condition as a sufficient safety margin.

The DLC 8.2 includes the loading effect on the turbine components during the installation stage that includes transport, assembly, maintenance and repair, for a period of time longer than one week. Shall be considered the loading effect in a partially or completed tower, tower with the nacelle and wind turbine without one or two blades. Electrical network shall be assumed to be disconnected.

Partial safety factors for loads:

The partial safety factors to be used for ultimate limit states (ULS) and fatigue calculations are expressed in Table 2.3.

Table 2.3 – Partial safety factors for ULS and fatigue (extracted from [12])

Unfavourable loads				Favourable loads
Type of design situation				
Normal (N)	Abnormal (A)	Transport and erection (T)	Fatigue (F)	All design situation
1.35*	1.10	1.50	1.00	0.90

* For design load case DLC 1.1, given that loads are determined using statistical load extrapolation at prescribed wind speeds between V_{in} and V_{out} , the partial load factor for normal design situations shall be $\gamma_f = 1.25$.

For Serviceability Limit State Analysis (SLS) of towers and foundations, [17] classifies the load levels such as:

1. S1: Characteristic load level, corresponding to the normal (N) extreme values from the complete set of cases, defined in Table 2.2.
2. S2: Frequent load level, corresponding to the 0.01 percentile values from the complete set of cases denoted by the use of the F partial safety factor, defined in Table 2.2.
3. S3: Quasi-permanent load level, with the probability of exceedance of 1% from the fatigue spectrum. The loads are derived from the S2 but using the 1 percentile values.

The partial safety factor to be used the SLS load levels is:

$$\gamma_{SLS} = 1.0 \quad (2.51)$$

2.6 FOUNDATION TYPOLOGIES

Onshore wind turbine foundations typically use reinforced concrete with circular or polygonal shapes, as illustrated in Figure 2.14, since these are cost-effective for supporting towers with variable overturning moment directions [18] [19]. The most common type of footing consists of a tapered cast in situ solid slab, although some proposals have been made to use ribbed solutions [20], or even building the footing as an assembly of precast elements [21].

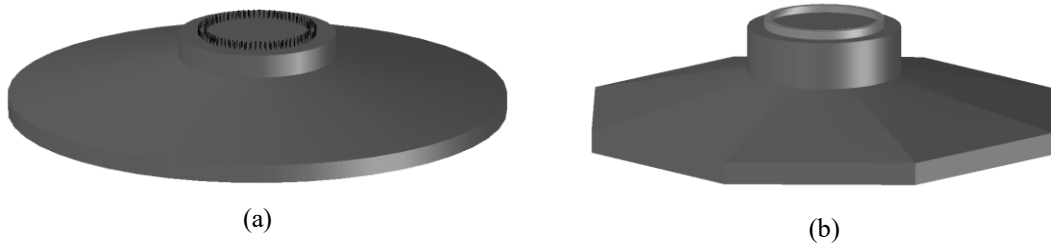


Figure 2.14 – Onshore wind turbine foundation shape: (a) circular shape; (b) octagonal shape

The connection between the steel tower and the concrete foundation is usually done considering one of the two solutions: (1) embedded steel tower foundation section (TFS); (2) prestressed anchor cage (AC).

As shown in Figure 2.15, the TFS corresponds to the first tower segment, with approximately 1.5 m to 2 m height, (depending on the manufacturer and wind turbine hub height) that is embedded in the concrete.



Figure 2.15 – Connection between steel tower and concrete foundation using the TFS solution: (a) schematic drawing (b) onsite installation

The tower segment features evenly distributed holes along the entire perimeter, allowing the radial rebars to pass through the steel shell. This reinforcement detail is required for internal equilibrium of the forces developing in the concrete pedestal.

The AC solution consists of post-tensioned vertical bolts fixing the bottom flange of the steel tower to the flange embedded in the concrete footing (Figure 2.16), enabling the transfer of the tensile forces to the bottom surface of the footing.



Figure 2.16 – Connection between steel tower and concrete foundation using the AC solution: (a) schematic drawing (b) onsite installation

The nature of soil beneath the foundation is one of the most important features responsible for the definition of the foundation typology. Figure 2.17 (a) to Figure 2.17 (f) identify, schematically, the solutions usually adopted, depending on the type of soil.

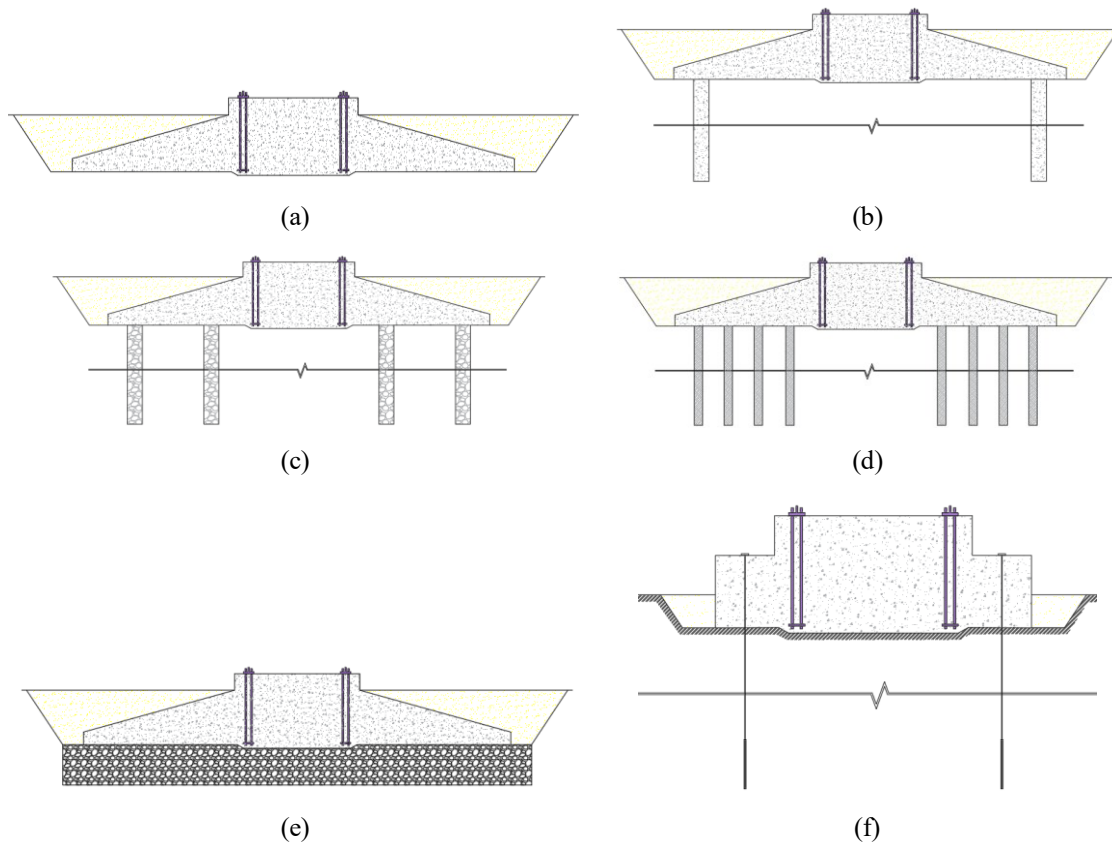


Figure 2.17 – Schematic drawings of different types of onshore wind turbine foundations: (a) shallow foundation; (b) pile foundation; (c) shallow foundation on soil reinforced with stone columns; (d) shallow foundation on soil reinforced with inclusions; (e) shallow foundation on exchanged soil; (f) anchored foundation.

2.7 DESIGN CRITERIA FOR ONSHORE SHALLOW FOUNDATIONS

The design of onshore wind turbine shallow foundations follows the recommendations in [17], [22], fib MC 2010 [23] and Eurocode 2 [24]. The foundations shall satisfy the Ultimate Limit State (ULS), Serviceability Limit State (SLS) and Fatigue Limit State (FLS), with checks covering both geotechnical and structural performance.

At ULS, the foundation is verified against loss of equilibrium and geotechnical failure. The principal checks typically include safety against overturning, bearing capacity of the soil, sliding resistance at the soil-foundation interface and structural resistance of the concrete and reinforcement. For overturning and bearing capacity, the combined effect of vertical, horizontal and torsional loads is usually treated through an equivalent eccentric loading at the foundation base, often under the simplifying assumption of rigid body behaviour. The checks against sliding are performed by comparing the design horizontal actions with the available frictional resistance along the foundation base. Structural verifications at ULS are carried out in accordance with Eurocode 2, ensuring adequate resistance to bending moments and shear forces, and employing strut-and-tie models in regions with highly non-linear strain fields, such as the pedestal.

At SLS, the emphasis is placed on operational performance and durability. The rotational stiffness of the foundation at the tower base shall be sufficient to meet the turbine manufacturers dynamic requirements. Inclination and long-term settlement of the foundation are checked against prescribed limits in order to avoid excessive tilt and associated second-order effects in the tower. The influence of cyclic loading on soil stiffness and strength is also considered, with particular attention to the potential development of gaps at the foundation-soil interface and the associated loss of contact area. In parallel, crack widths in the reinforced concrete slab and pedestal are controlled at SLS in accordance with Eurocode 2 procedures, using appropriate load combinations representative of long-term operation.

At FLS, the design addresses the cumulative damage induced by the large number of load cycles experienced over the turbine lifetime in the reinforcement and concrete. The total equivalent fatigue damage is determined using the Palmgren-Miner rule, satisfying the condition that the cumulative damage sum is less than unity. This assessment uses S-N curves for the reinforcing steel and load level spectra, typically in the form of Markov matrices, for the concrete elements, in accordance with fib Model Code recommendations [23].

3 STRUCTURAL MONITORING OF WIND TURBINES AND FOUNDATIONS

3.1 INTRODUCTION

The objective of this chapter is to present an experimental setup designed to provide reliable information regarding the wind forces acting on the tower structure and foundation of wind turbines, as well as the dynamic behaviour of the wind turbines during operation, including the impact of damaged foundations. The monitoring system must be capable of capturing the mechanical loads applied to the structure and the dynamic response of the wind turbines, while also considering the environmental and operational data that are already available through the SCADA system. Similar studies conducted by [25] and [26] were used to best decide on the strategy to be adopted.

This research is divided in two sections: (1) long-term measurements of an onshore wind turbine located in the wind farm of Serra del Rei in Portugal, and (2) long-term measurements of an onshore wind turbine with a damaged foundation, along with short-term measurements in four wind turbines, with and without damaged foundations, all located at Serra da Lousã, Portugal. The following sections provide a brief description of the key features of the wind turbines, along with details of their locations. The various instrumentation layouts adopted for the wind turbines are described, and the resulting experimental data is presented and analysed.

The data discussed in this chapter forms the foundation for the work presented in the subsequent chapters. These datasets, which include wind-induced strains, accelerations, forces, and rotations, will be used in Chapters 4, 5 and 6 to validate the numerical models, enabling accurate predictions of the behaviour of wind turbines and foundations under varying conditions.

3.2 MONITORING CAMPAIGN AT SERRA DEL REI WIND FARM

The firstly instrumented wind turbine is located in Serra del Rei wind farm, in the municipality of Óbidos, Portugal (Figure 3.1). It consists of one turbine with a rotor diameter of 100 meters and a hub high of 95 meters, with a rated power of 2 MW, developed for IEC 2B wind class.



Figure 3.1 – Serra del Rei Wind Farm. Wind turbine location and photo

The steel tower is made of 4 cylindrical segments with constant diameter in the first segment, and variable diameter in the remaining. The shallow foundation is made of reinforced concrete with circular shape and the connection between the steel tower and the foundation is made through the anchor cage system, consisting of post-tensioned vertical bolts fixing the steel tower bottom flange to the flange embedded in the concrete footing.

The instrumentation was designed and implemented during the installation of the wind turbine, in 2020. The main goal is to evaluate the forces at the tower base during the operation of the wind turbine, quantify the rotational stiffness of the foundation and the forces in the anchor cage bolts. This data is to be compared with the numerical simulation results presented in the following chapters. With this strategy, it will be possible to assess the suitability of the modelling procedures that may lead to optimized solutions when applied in the design stage.

3.2.1 DESCRIPTION OF THE MONITORING SYSTEM

The monitoring system comprises the following (Figure 3.2 and Figure 3.3):

- Four load cells with annular ring profile, incorporating bonded foil strain gauges, with a maximum capacity of 750 kN, to monitor the anchor bolt forces.
- Two cross-sections of the steel tower equipped with four linear electrical strain gauges each, allowing the estimation of the bending moments and shear forces. The first row of strain gauges is placed alongside with four temperature sensors for temperature correction. The type of equipment and installation followed the procedures implemented in the studies developed by [27] and [28].
- Four accelerometers at two different levels (two in each level), for continuous monitoring of the natural frequencies. Low noise ($5\mu\text{g}/\sqrt{\text{Hz}}$) MEMS sensor with a sensibility of 2V/g , measuring from -2g to $+2\text{g}$ with linear response in the frequency range $0\text{-}400\text{Hz}$. The equipment and location in the tower were decided based on investigations developed by [29] and [30].

- Inclinometer at the top surface of the foundation (pedestal) to calculate the foundation rotation. The solution adopted is a compact biaxial inclinometer with a range of $\pm 3^\circ$.

All the monitoring devices are connected to a central acquisition unit, recording the data with a sampling frequency of 50 Hz. The system operated continuously from 2021 until 2023. Additional information from the wind turbine operation and wind characteristics is supplied by the SCADA system.

The acquisition unit is a National Instruments CompactRIO with 4 modules responsible for reading the electrical signals of the sensors and generate hourly files with the recorded data. This unit is stored in a cabinet that also contains a power supply unit, a backup battery and a modem to send the data to a server located at FEUP.

Table 3.1 resumes the dates of the installation of the monitoring system components and site visits needed to improve the monitoring system performance. The first 3 visits were dictated by the works associated with the wind turbine construction and assembly. The operation of the system started after the visit at 19/02/2021. However, during this visit, it was not possible to conclude all the connections and validate all the sensors at the field. This was only possible at 04/05/2021, so some sensors readings are only valid after this date.

Table 3.1 – Dates of the main installation and system improvement tasks

Task	Date
Installation of the load cells	03/09/2020
Installation of the sensors in the tower (accelerometers, strain gauges and temperature sensors)	03/12/2020 – 04/12/2020
Installation of the inclinometer, data acquisition system and beginning of data acquisition	19/02/2021
Site visit to conclude and improve connections of the strain gauges, accelerometer at the top level and inclinometer	04/05/2021

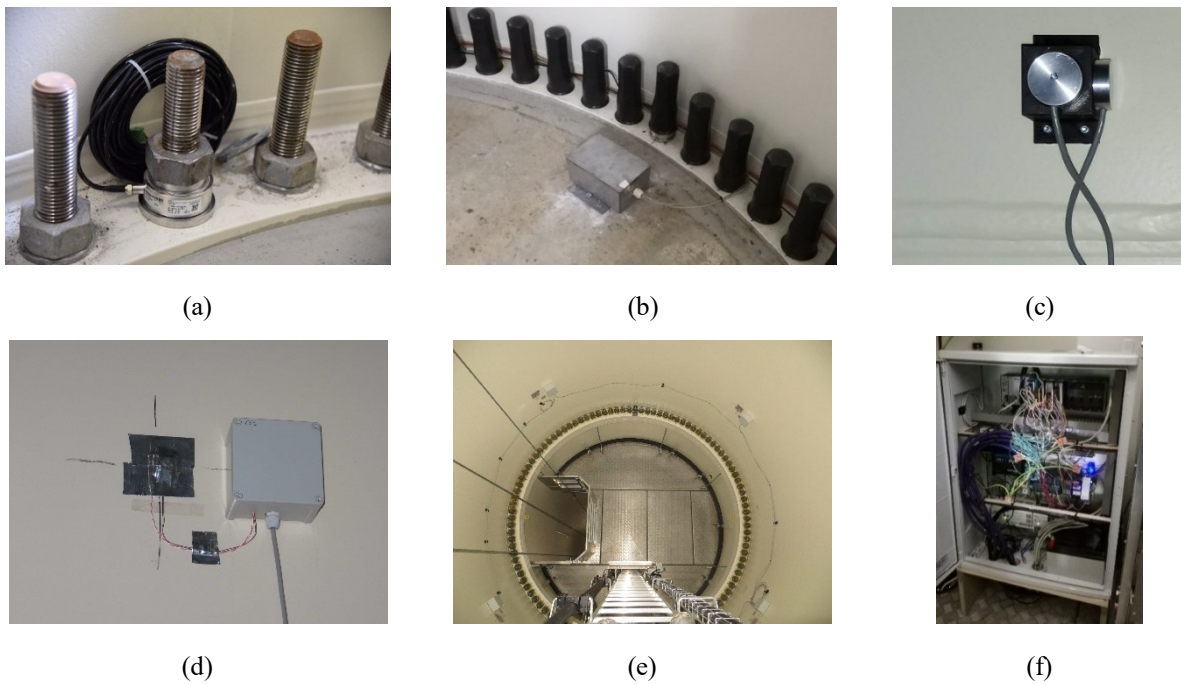
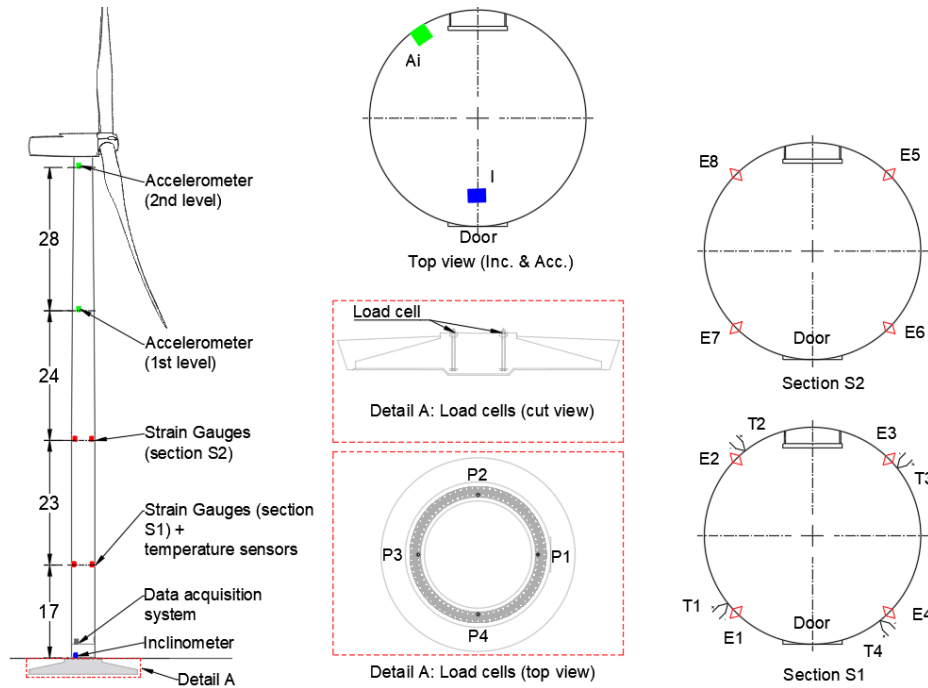


Figure 3.3 – Pictures of the monitoring devices: (a) load cell; (b) inclinometer; (c) accelerometers; (d) strain gauge and temperature sensor; (e) section S1 with strain gauges and temperature sensors; (f) cabinet with the data acquisition system

3.2.2 DATA PROCESSING

3.2.2.1 STRAIN GAUGES

Strain gauges are highly sensitive sensors susceptible to various factors that can significantly impact the measurement outcomes. Consequently, identifying all potential error sources and implementing appropriate methodologies to enhance the precision of the final results is crucial. Therefore, evaluating the bending moment and rotation necessitates post-processing of the raw data.

The bending moment is evaluated from the deformation measured by the strain gauges. The methodology to obtain a proper estimate requires the following steps.

- Temperature compensation effect: this step involves the use of the thermal output compensation curve (expression (3.1)), provided by the strain gauges supplier in conjunction with the recorded temperature of the steel tower surface;
- Deformation zeroing: this step entails removing the strain associated with the installation of the sensor and averaging signals from diametrically opposed strain gauges to mitigate parasitic strains induced by temperature gradients;
- Calibration check to counteract the drift effect, according to the guideline IEC 61400-13: this procedure involves rotating the nacelle 360° with a wind speed below the cut-in value. The resultant sinusoidal signal, generated due to the eccentricity of the rotor and nacelle mass during the full rotation, allows for the correction of drift. Disregarding the axial force effect, the mean value of the signal serves as the zero point in this corrective procedure.

Temperature compensation

In this study, the temperature compensation curve provided by strain-gauge supplier (Micro-Measurements) is given by equation (3.1). This strain is subtracted from the readings. The temperature values are those measured by the temperature sensors at section S1 in the vicinity of each strain gauge (it is assumed the same temperature for strain gauges in the same vertical alignment).

$$\varepsilon_{(T/0)} = -98.6 + 6.4 \cdot T - 1.04 \times 10^{-1} \cdot T^2 + 4.9 \times 10^{-4} \cdot T^3 - 5.94 \times 10^{-7} \cdot T^4 \quad (3.1)$$

Deformation zeroing

After applying the temperature correction, the signal of each strain gauge must be zeroed (since the goal is the monitoring of the bending moments, the strains are assumed to be zero when the resultant bending moment in the alignment of the sensors is zero). This procedure requires the yawing of the nacelle through 360° with wind velocities below cut-in wind speed. As the blades, nacelle and hub centre of gravity are eccentric in relation with the tower axis, the rotation of the nacelle generates a sinusoidal signal in each strain gauge. Figure 3.4 identifies the occurrence of one 360° yaw rotation event and the sinusoidal signal captured by the strain gauges. The average of a complete sinusoid recorded by each sensor is subtracted from the signal.

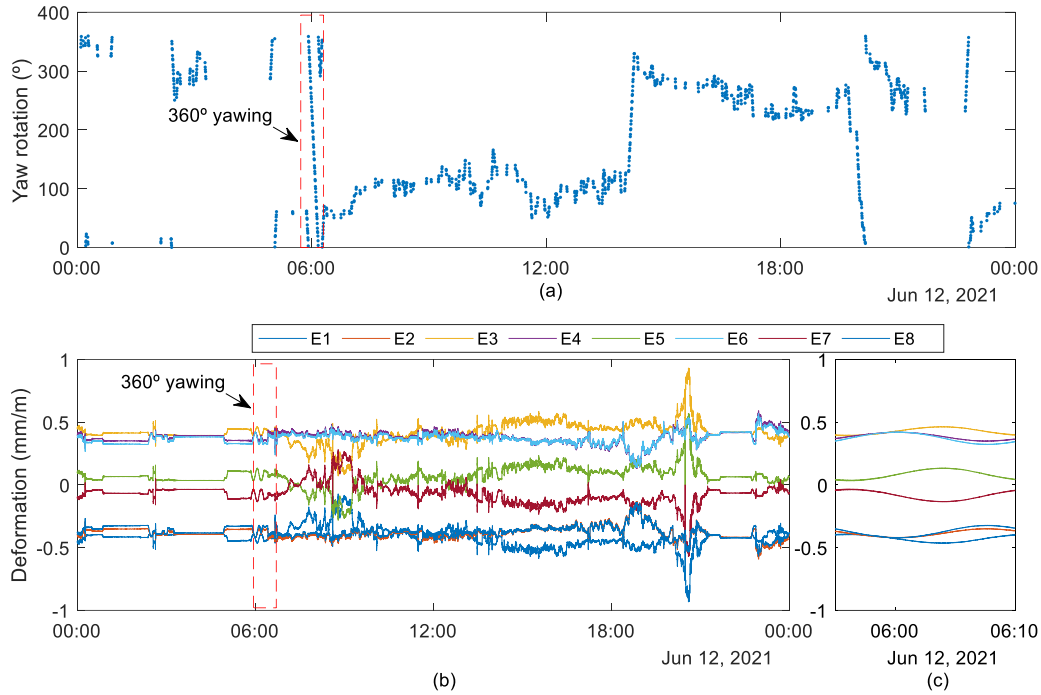


Figure 3.4 – Detail of the sinusoidal signal evaluated at the strain gauges as a consequence of the 360° yawing: (a) yaw angle; (b) deformation recorded by the strain gauges over the entire day; (c) detail of the deformation during the 360° yawing

Calibration check to counteract the drift effect

Another important source of error might be long term drifts of the signal in the strain gauges. This phenomenon can be captured by comparing the signal for different 360° yawing occurrences. If the deformation curves deviate between different 360° yawing occurrences, it means that the strain gauges are drifting, and corrections are needed. Figure 3.5 compares the recordings during three 360° yawing, occurred in 12, 15 and 24 of June 2021, captured with the strain gauges at section S1 and section S2, respectively. It is clearly visible the overlapping of the curves at each strain gauge which means that no drifting phenomena was observed during this period.

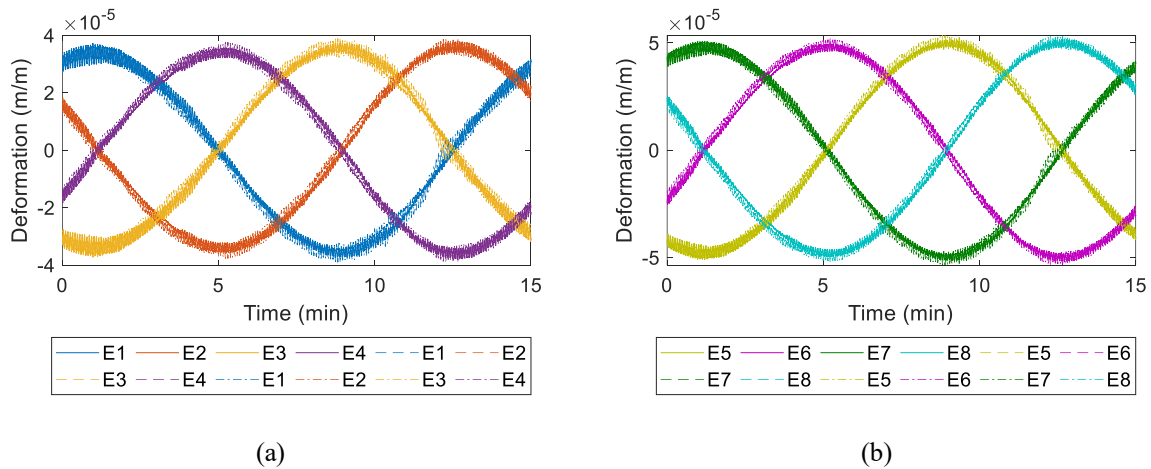


Figure 3.5 – Deformation zeroed for the 360° yawing for the strain gauges. Comparison between the three 360° yawing occurrences: (a) at section S1; (b) at section S2

3.2.2.2 ACCELEROMETERS

Data recorded by the accelerometers are converted to acceleration in m/s^2 by the relationship:

$$\text{Acceleration } [\text{m/s}^2] = \text{Sensor output} / 2 \cdot 9.81 \quad (3.2)$$

Figure 3.18 shows, for illustration purpose an example of the Root Mean Square (RMS) values of the horizontal acceleration time series recorded during the entire month of August 2021. The complete series is presented in Section 3.2.3. The four curves correspond to the two measurement levels and the two orthogonal horizontal directions of the accelerometers. The RMS values remain relatively low, typically below 1.0 m/s^2 , but exhibit clear temporal variability, with clusters of higher peaks that can be associated with periods of increased wind excitation or particular operating conditions of the turbine (for example, start-ups, shutdowns or changes in control regime). The similar order of magnitude of the four curves indicates that the dynamic response of the tower is comparable in both horizontal directions and at both instrumented levels.

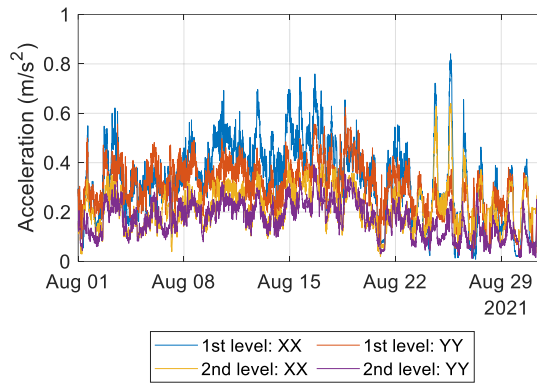
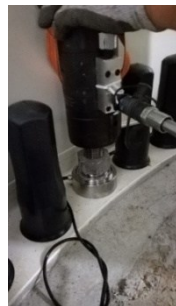


Figure 3.6 – Example of the RMS horizontal acceleration measured by the accelerometers during August 2021

3.2.2.3 LOAD CELLS

The wind turbine manufacturer specifies a post-tension force at each anchor bolt of 561 kN. During the installation, a force of 567 kN was applied at each anchor bolt (Figure 3.7) and the output was recorded in Table 3.2. This information was used to set the slope of the linear relation between the output (mV/V) and the force (kN). As a consequence, it is assumed that the force in the four load cells was the same immediately after tensioning.



(a)



(b)

Figure 3.7 – Post-tension force applied in the anchor bolts: (a) detailed view of the tensioner; (b) main view, including the datalogger

Table 3.2 – Readings provided by the load cells before and after applying the post-tension force in the anchor bolts

Load cell	Nominal post-tension force (kN)	Load cell output before applying post-tension (mV/V)	Load cell output after applying post-tension (mV/V)
P1	567	0.0141	1.0075
P2	567	0.0070	1.0862
P3	567	0.0075	0.7444
P4	567	0.0056	1.1140

Through the observation period, the readings during one regular maintenance operation were recorded, being observed that the monitored anchor bolts were unstressed and restressed. This observation, shown in Figure 3.8, proved that the load cells are working properly.

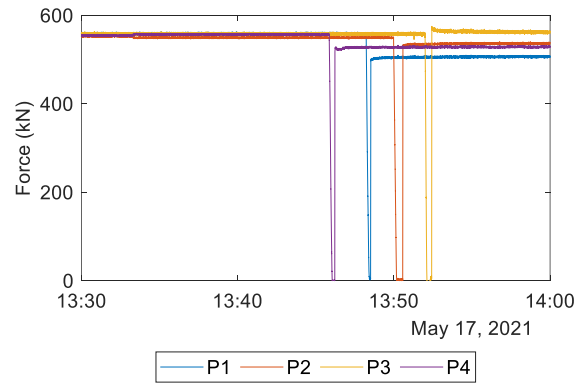


Figure 3.8 – Detail of the intervention in the anchor bolts detected by the load cells

3.2.2.4 INCLINOMETER

The inclinometer was installed in a flat surface, attached to the concrete pedestal. Since the horizontally of the surface cannot be ensured during the installation, a procedure for zeroing the inclinometer output was adopted, using the same principle used for the strain gauges.

The sensor output provides the inclination in the x and y directions. To evaluate the rotation of the pedestal is necessary to find the resultant rotation, which can be determined through the expression (3.3):

$$R_{res} = \sqrt{R_x^2 + R_y^2} \quad (3.3)$$

where R_x is the rotation along the x direction and R_y is the rotation along the y direction.

Figure 3.9 shows the rotations R_x and R_y zeroed for the 360° yawing. It is confirmed that R_x and R_y are 90° out of phase (their maxima are associated with a yaw angle difference of 90°), as expected. The resultant rotation during the yaw rotation event is not constant, which may indicate that the rotational stiffness of the footing is direction dependent.

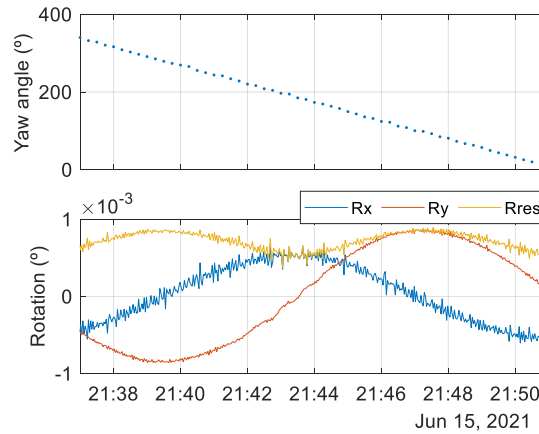


Figure 3.9 – Pedestal rotation during the 360° yawing cycle, showing the yaw angle in the upper plot and the rotation in the x-direction (R_x), y-direction (R_y), and resultant rotation (R_{res}) in the lower plot, with rotation zeroed during yawing.

3.2.3 RESULTS

The plots from Figure 3.10 shows data with 10-minute resolution, recorded until end of August 2023, for the complete monitoring system. The data recorded by the different sensors is consistent and synchronized, showing that the events are well reproduced by the entire system.

The first plot (a) presents the deformation data recorded by the strain gauges located at the two sections of the steel tower. The data, showing average deformation values, exhibits periodic fluctuations that correspond with the operational conditions and external loading. The consistent patterns of the deformation across both sections of the tower demonstrate that the monitoring system effectively captures reliable strain responses throughout the observed period, offering insight into the structural dynamics under varying loads and environmental conditions.

The second plot (b) presents the root mean square (RMS) values of the accelerometer data from the two sections of the tower. The RMS values, shown for both the first (blue and red) and second (orange and purple) measurement levels, illustrate the intensity of vibrations at different heights of the tower.

The third plot (c) displays the averaged forces recorded by the load cells located at four anchor bolts of the anchor cage. The data shows relatively stable force values with minor fluctuations, reflecting the steady distribution of loads during the monitored period. The stability and consistency of the force readings suggest that the anchor bolts are functioning within expected parameters under operational conditions.

Finally the plot (d) presents the resultant rotation data recorded by the inclinometer positioned on the top surface of the pedestal. The plot reveals gradual but small increases in rotation. The relatively smooth and consistent trend suggests that the foundation is responding as anticipated, with no significant or abrupt movements detected during the monitoring period, indicating a stable foundation under the given loading conditions.

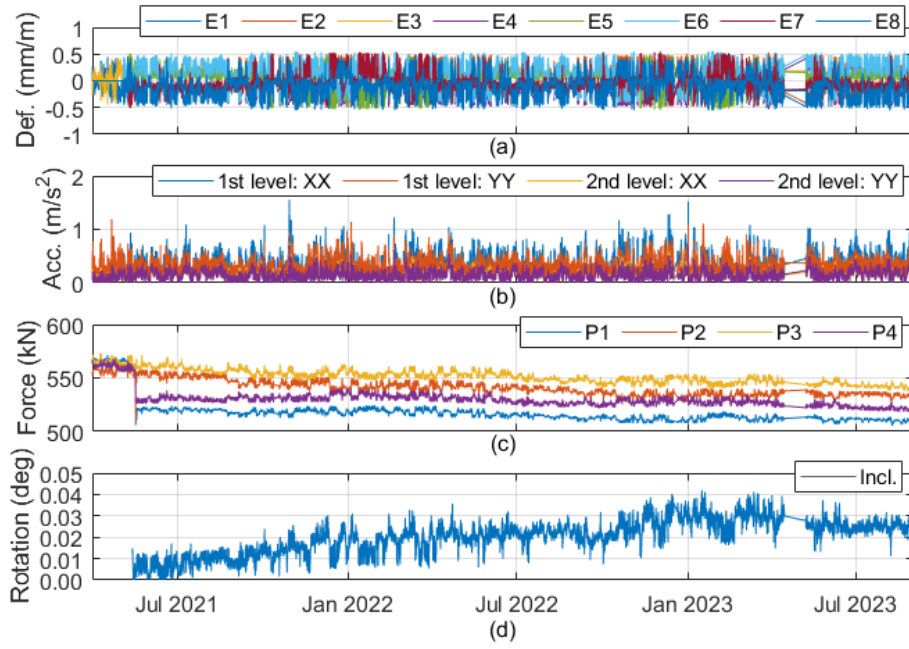


Figure 3.10 – Complete time series with 10-minute resolution: (a) averaged strain gauges deformation; (b) accelerometers RMS; (c) averaged load cell forces; (d) averaged inclinometer resultant rotation

3.2.3.1 ESTIMATION OF THE BENDING MOMENTS

With the longitudinal strain corrected signals, it is possible to evaluate the bending moments at the two instrumented tower sections and extrapolate the results to the tower bottom. Another advantage of having two monitored sections is the possibility to evaluate the tower shear force. Figure 3.11 shows the coordinate systems used to evaluate the bending moments in this study: M_x and M_y are defined by the position of the tower door, whereas M_{FA} and M_{SS} are defined as a function of the nacelle orientation, being the angle β determined from the yaw angle obtained from the SCADA data.

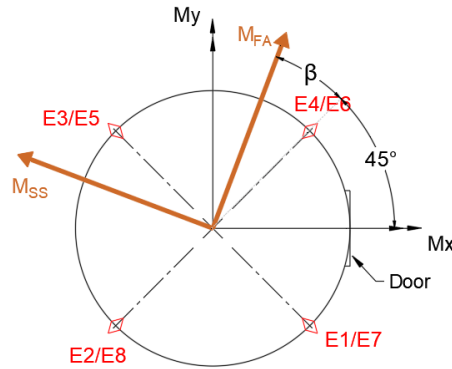


Figure 3.11 – Coordinate system of the tower cross-section with the strain gauges

The longitudinal strains (ϵ_{zz}) obtained from the strain gauges E4/E6 and E3/E5 are used to calculate the bending moment along the rotor axis (F-A), following the same procedure as described in [26].

Due to the small thickness of the tower shell, compared to the other dimensions, and according to the theory of linear elasticity, the perpendicular stresses can be considered negligible ($\sigma_{rr} = 0$). Therefore, the stress-strain relationship for isotropic materials in plane stress can be simplified, becoming:

$$\begin{bmatrix} \epsilon_{zz} \\ \epsilon_{\theta\theta} \\ \gamma_{\theta z} \end{bmatrix} = \frac{1}{E} \cdot \begin{bmatrix} 1 & -\nu & 0 \\ -\nu & 1 & 0 \\ 0 & 0 & 1 + \nu \end{bmatrix} \cdot \begin{bmatrix} \sigma_{zz} \\ \sigma_{\theta\theta} \\ \tau_{\theta z} \end{bmatrix} \quad (3.4)$$

with the direction of the stress components schematically represented in Figure 3.12.

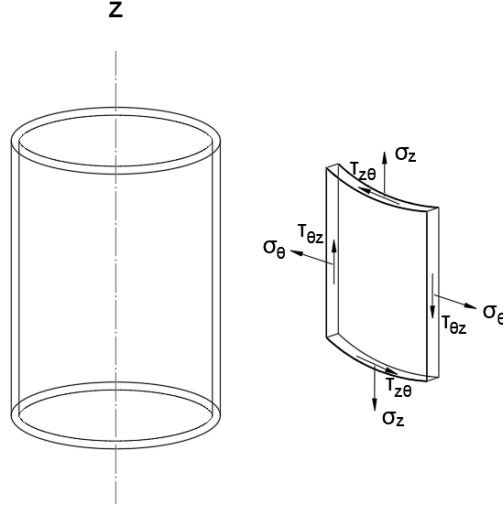


Figure 3.12 – Direction of stress components

Numerical simulations performed on towers have shown that the transverse deformations ($\epsilon_{\theta\theta}$) are negligible when compared to longitudinal deformations when subjected to thrust forces [26]. Therefore, the axial stress acting at the tower shell can be evaluated by means of the expression (3.5).

$$\sigma_{zz} = E \cdot \epsilon_{zz} \quad (3.5)$$

with E being the Young's modulus of the steel.

The stresses can also be calculated from the acting bending moment and axial force, assuming that the points under analysis are far away from any geometrical discontinuity and the plane sections remain plane after the deformation, resorting to the expression (3.6):

$$\sigma_{zz} = \frac{F_v}{A} \pm \frac{M}{I} \cdot R \quad (3.6)$$

where F_v is the axial force mainly due to the self-weight of the wind turbine, M the bending moment, A the cross-sectional area, I the 2nd moment of area of the tower and R the internal radius of the tower, where the stresses are evaluated. The axial force cannot be measured since the strain gauges can only provide variations from the stress state of the structure in the moment of installation.

Considering that the axial force variation can be neglected during the operation of the turbine, the acting bending moment can be determined from expression (3.5) and (3.6), leading to the expression (3.7) for the bending moment around the j -axis normal to the radius passing by the i^{th} strain gauge.

$$M_j = \frac{EI}{R} \cdot \epsilon_i \quad (3.7)$$

The corresponding fore-aft bending moment is given by:

$$M_{F-A} = \frac{E \cdot I}{R} \cdot [\varepsilon_{4/6} \cdot \cos(\beta + 45) + \varepsilon_{3/5} \cdot \sin(\beta + 45)] \quad (3.8)$$

where ε are the strains measured in the same cross-section and β is defined in Figure 3.11. The eccentricity of the rotor and nacelle is expected to generate a constant bending moment along the tower height during their rotation with low wind speeds. This can be assessed by evaluating the bending moments at the two cross-sections, where the strain gauges are installed. In Figure 3.13, the resultant bending moment evaluated at the two sections S1 and S2 are compared, during the 360° yawing event. As expected, the resultant moment is similar for the two sections during the full nacelle rotation. This is important to validate the calibration of the sensors and the data processing strategy.

Note that in Figure 3.13 the component bending moments M_i and M_j are shown with opposite sign between sections S1 and S2 solely to facilitate visual comparison. In reality, M_i and M_j at S1 and S2 have the same sign.

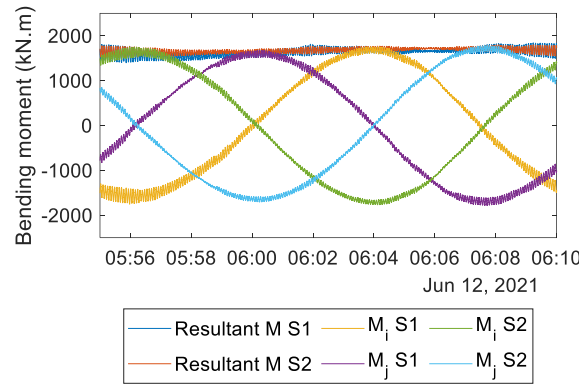


Figure 3.13 – Resultant bending moment evaluated during 360° yawing at section S1 and S2

Figure 3.14 illustrates the maximum 10-minute bending moment recorded at the two sections equipped with strain gauges and compares these values with the corresponding wind speeds. Figure 3.15 presents the bending moment and shear force determined at section S1, based on the assumption of a linear bending moment distribution along the height of the tower. These results are then compared with the theoretical bending moment and shear force at section S1, which are expected to create a gap between the foundation and the underlying soil. Additionally, the comparison includes the SLS QP load combination. These theoretical values are marked on the same plot by a horizontal red line and a horizontal red dashed line, respectively.

Based on the recorded data, no extreme events were detected during the period in question, and the stress resultants remained below the theoretical bending moment that could cause a gap between the foundation and the soil beneath.

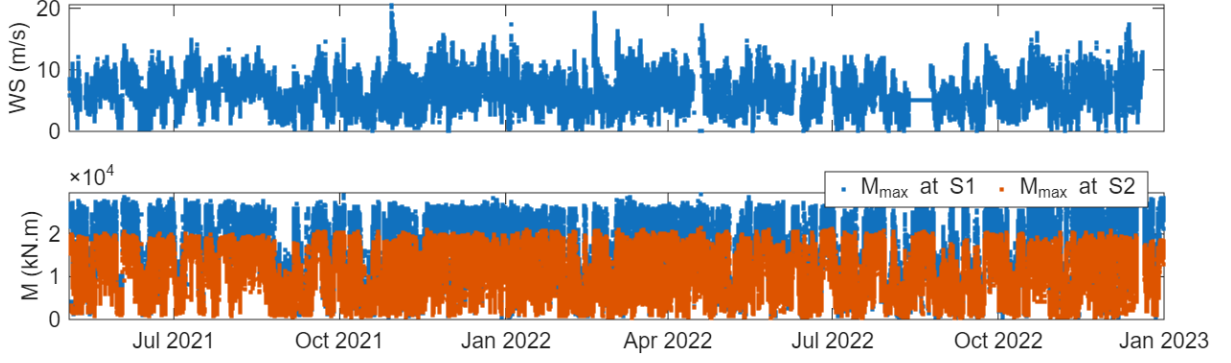


Figure 3.14 – Maximum 10-minute results: top plot illustrates the wind speed, while the bottom plot illustrates the maximum bending moments at S1 and S2

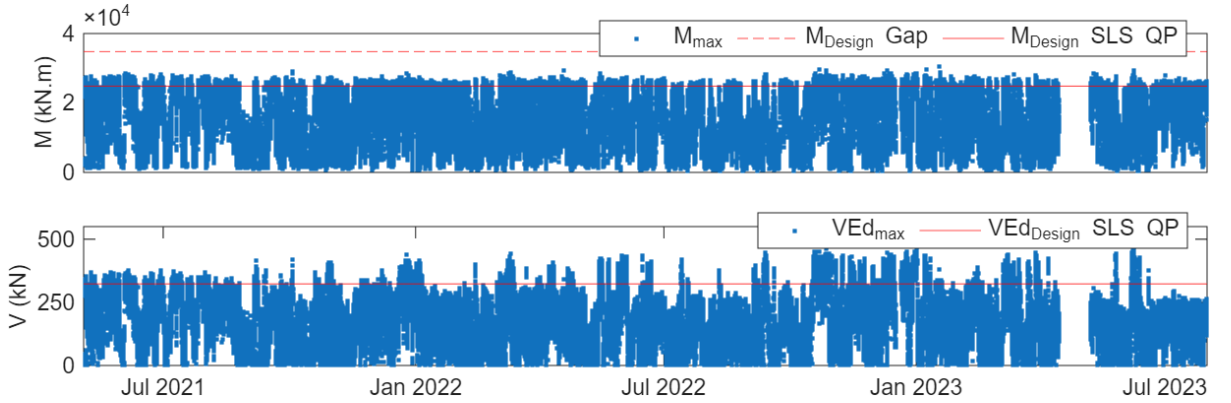


Figure 3.15 – Top plot displays the maximum 10-minute bending moment at section S1, with the design bending moment for the combination SLS QP and GAP, marked by red and red dashed lines, respectively. Bottom plot shows the maximum 10-minute shear force and horizontal force for the combination SLS QP, marked by a red line

The bending moment acting at the tower, derived from the experimental results, is compared with the bending moment curve generated from the blade element momentum theory, described in Section 2.3. For this exercise, the bending moment due to the thrust force is evaluated by the expression (3.9), considering the thrust coefficients (c_T) provided by the wind turbine owner and, assuming a standard air density ($\rho=1.225 \text{ kg/m}^3$).

$$M_i = \frac{1}{2} \cdot \rho \cdot A_{rotor} \cdot v_1^2 \cdot c_T \cdot H_i - M_{ecc} \quad (3.9)$$

where H_i denotes the vertical distance between the rotor centre and the tower section at which M_i is evaluated, and M_{ecc} represents the additional moment arising from the horizontal eccentricity of the rotor-nacelle assembly relative to the tower centre.

Figure 3.16 compares the bending moment derived from the experimental results, represented by blue points, with the theoretical bending moment due to the thrust force acting at the rotor, represented by the red line. The effects of the wind acting at the tower and the 2nd order moments due to the deflection of the tower were not considered. The results are evaluated at (1) tower bottom section, (2) section S1 and (3) section S2. The comparison between experimental and theoretical bending moments across the three sections of the tower reveals that, overall, the theoretical model provides a good approximation of the bending moments acting on the tower.

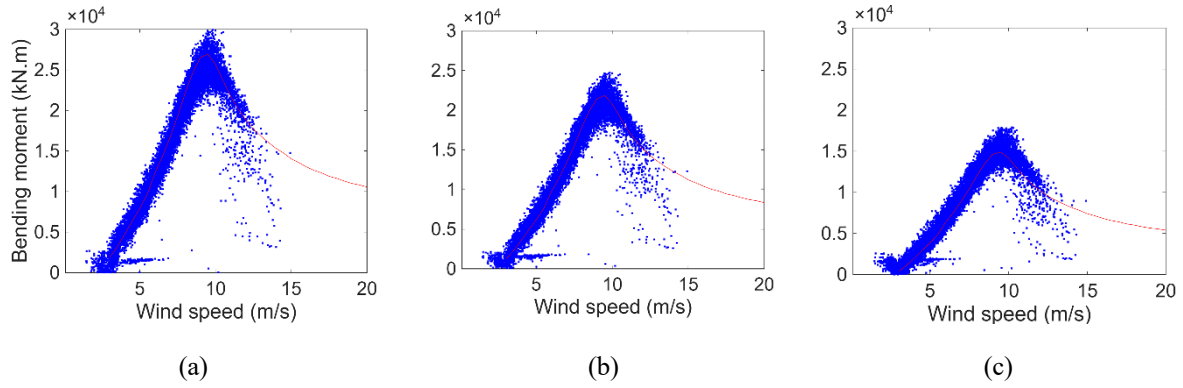


Figure 3.16 – Bending moment evolution with wind speed variation. Comparison between numerical (–) and experimental results (•): (a) evaluation at tower bottom section; (b) evaluation at section S1; (c) evaluation at section S2

Figure 3.17 compares the bending moment normalised by the bending moment at SLS, expressed as (M/M_{SLS}) with the rotation measured at the pedestal, obtained from the inclinometer installed on the horizontal surface of the concrete foundation. Despite the inclinometer drift previously identified and discussed, the scatter cloud exhibits a clear and approximately linear trend over the measured range, indicating a near-proportional relationship between the applied bending moment and the pedestal rotation. The observed linearity suggests that, for the analysed operational conditions, the pedestal response can be characterised by an essentially constant rotational stiffness.

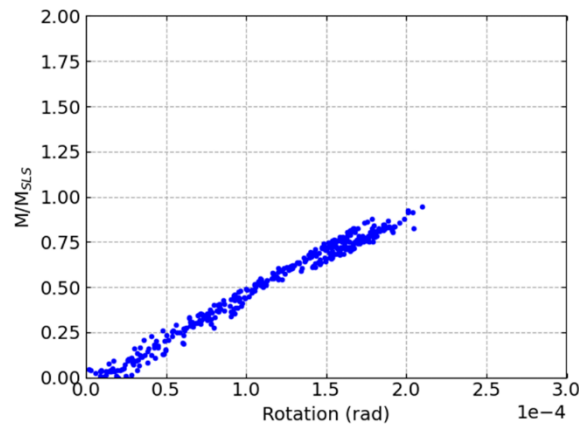


Figure 3.17 – Relationship between normalised bending moment and rotation of the pedestal during WTG operation

3.2.3.2 NATURAL FREQUENCIES

Figure 3.18 presents Campbell diagrams derived from the horizontal accelerometer measurements at the Serra del Rei wind turbine for two representative monitoring periods. Prior to spectral analysis, the acceleration records are linearly detrended and filtered using a fourth order Butterworth band pass filter in the range 0.05 to 10 Hz in order to suppress very low frequency drift and high frequency noise.

The Campbell diagrams are constructed from FFT based power spectral density (PSD) estimates computed using the Welch approach. The continuous time series are divided into segments of 60 seconds with 50% overlap, resulting in a frequency resolution of approximately $1/60 \approx 0.0167$ Hz.

To map the spectra against rotor speed, the rotor speed at the centre of each 60 seconds segment is obtained from the 10-minute SCADA records through time alignment and sample interpolation. Each segment PSD is then assigned to a rotor speed bin, with the rotor speed range divided into 30 equally spaced bins. For this project, the observed operating range is from 0 to 15 rpm, which corresponds to a bin width of approximately $15/30 = 0.5$ rpm. The PSDs associated with each bin are subsequently averaged, yielding the Campbell diagram as a colour map of PSD level versus frequency and rotor speed.

The oblique dashed lines superimposed on the diagrams represent the principal rotor harmonics, computed as $f = n \cdot \Omega / 60$ for $n = 1, 3$ and 6 , where Ω is the rotor speed in rpm. Persistent horizontal bands that remain approximately constant across rotor speed are interpreted as structural eigenfrequencies, whereas inclined bands aligned with the harmonic lines are associated with rotor induced excitation. For both monitoring periods, the first and second tower natural frequencies are consistently identified at $f_1 = 0.22$ Hz and $f_2 = 1.76$ Hz, respectively, in agreement with the persistent horizontal bands observed in Figure 3.18.

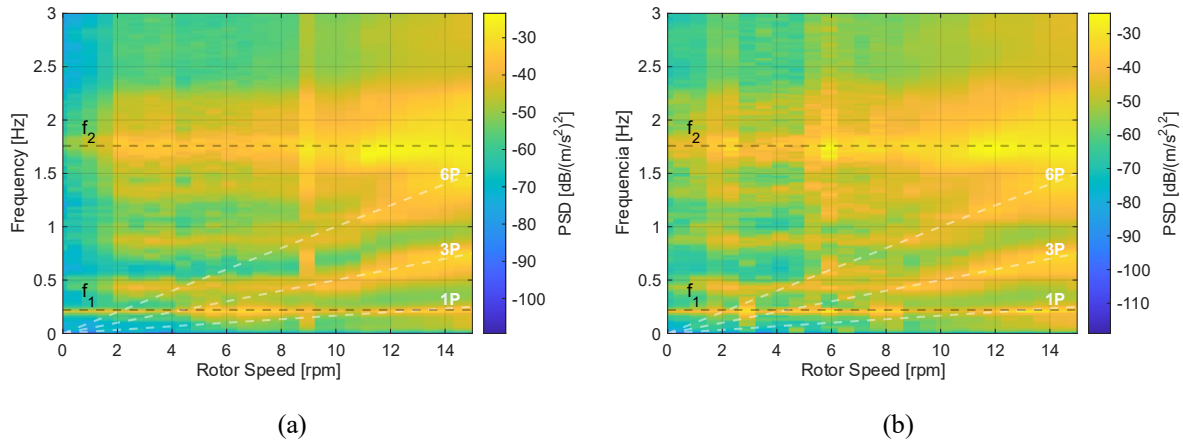


Figure 3.18 – Campbell diagram of the fore-aft eigenfrequencies identified from accelerometer measurements at the Serra del Rei wind turbine generator: (a) August 2022; (b) April 2023

3.3 EXPERIMENTAL CAMPAIGN AT SERRA DA LOUSÃ WIND FARM

A second experimental campaign was conducted on multiple wind turbines of Serra da Lousã wind farm. The primary objective was to evaluate the relative vertical displacement between the tower bottom section and the horizontal surface of the pedestal. The outcome of the measurements serves as indicators of potential gaps (voids) between the embedded T-flange of the tower bottom section and the surrounding concrete. In the absence of such gaps, the relative vertical displacement would solely result from the elastic deformation of the concrete and steel. Any measured displacements exceeding this threshold provides clear evidence of the presence of gaps.

The wind farm is located in Serra da Lousã, in the municipality of Lousã, Portugal (Figure 3.19). It consists of turbines with a rotor diameter of 88 meters and a hub high of 85 meters, with a rated power of 2.5 MW, developed for IEC 2A wind class.

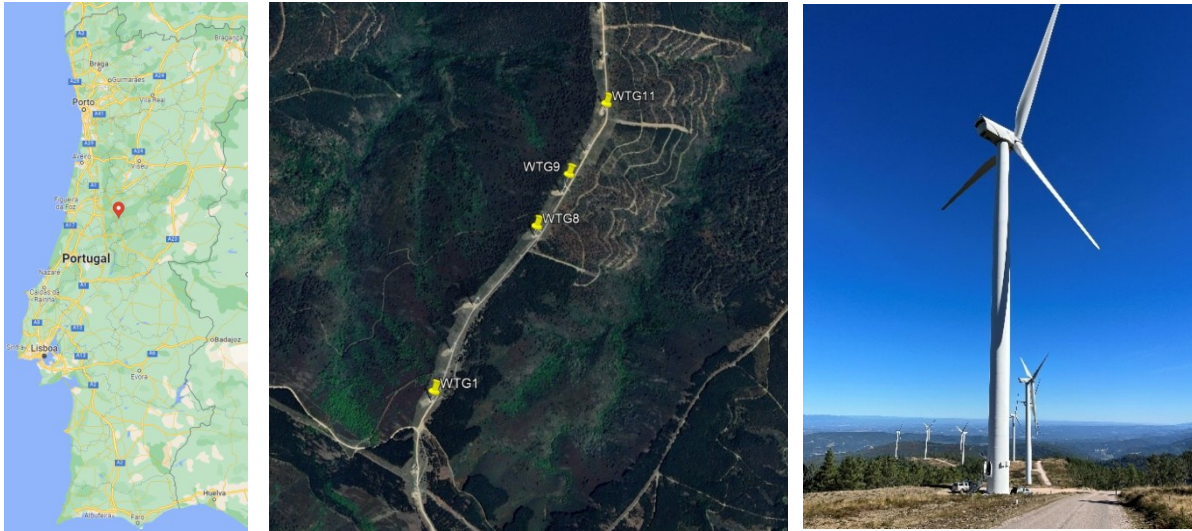


Figure 3.19 – Serra da Lousã Wind Farm. Wind turbines location and photo

The steel tower is made of 5 cylindrical segments (excluding the TFS) with constant diameter in the first four segments, and variable diameter in the last one. The shallow foundation is made of reinforced concrete with octagonal shape and the connection between the steel tower and the foundation is made through the tower foundation section (TFS) system, that corresponds to the first tower segment, with 1.98 m height that is partially embedded in the concrete.

3.3.1 DESCRIPTION OF THE EXPERIMENTAL CAMPAIGN

The experimental campaign comprised two distinct phases. The first phase involved measuring the relative vertical displacements between the base of the steel tower and the upper horizontal surface of the pedestal on 3 wind turbines (WTG 1, 9 and 11) during a free vibration event motivated by an emergency stop. These measurements were performed by the wind farm owner (NADARA) using Linear Variable Differential Transformer (LVDT) sensors. As illustrated in Figure 3.20, the sensors were installed on the outer side of the pedestal to measure the relative vertical displacement between the steel tower and the concrete pedestal. The vertical distance between the pedestal surface and the sensors, located in the tower, was approximately 20 cm. The sensors were connected to a datalogger that recorded the data for further analysis. The data collection was carried out in October 2020.

In the second phase, a comprehensive monitoring system was developed and implemented in a different wind turbine (WTG 8). The system was designed to be able to evaluate the bending moments and rotations at the base of the tower and the accelerations at two distinct levels along the tower height. To achieve these objectives, the monitoring system comprised the following equipment (Figure 3.21):

- One cross-section of the steel tower equipped with four linear electrical strain gauges, allowing the estimation of the bending moments. Along with the strain gauges, four temperature sensors for temperature correction are placed as well. The type of equipment and installation follow the procedures described in Section 3.2.

- Four accelerometers at two different levels, for continuous monitoring of the natural frequencies. Low noise ($40\mu\text{g}/\sqrt{\text{Hz}}$) MEMS sensor with a sensibility of 1 V/g , measuring from -2g to $+2\text{g}$ with linear response in the frequency range $0\text{--}400\text{Hz}$.
- One bi-axial inclinometer at the top surface of the foundation (pedestal) to calculate the foundation rotation and another bi-axial inclinometer at the tower bottom section to calculate the tower base rotation. The solution adopted is a compact biaxial inclinometer with a range of $\pm 3^\circ$.
- Four linear variable differential transformer (LVDT) sensors with an accuracy of 0.001 mm to measure the relative vertical displacement between the tower base section and the concrete pedestal (Figure 3.20).

The strain gauges, temperature sensors, accelerometers and inclinometers are connected to a central acquisition system configured to record the data continuously from September 2023 to December 2024 at a sampling frequency of 50 Hz , while the LVDT sensors, installed temporally to record particular events, are connected to an external data logger that stored the sensor readings in a laptop.

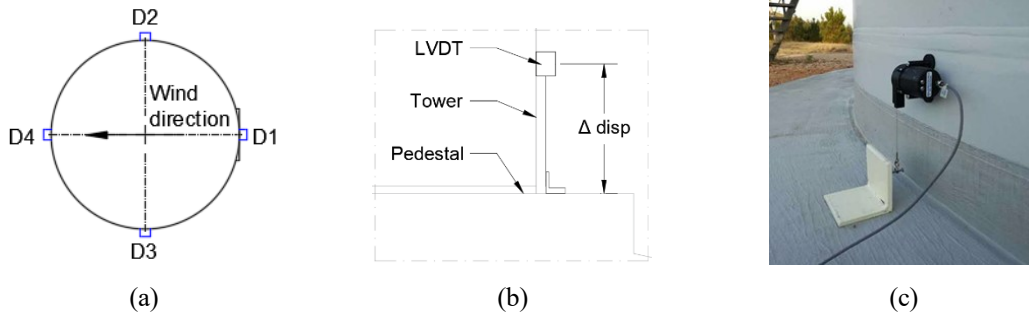


Figure 3.20 – Monitoring system implemented during phases one and two: (a) plan view indicating the location of the four LVDT sensors (D1-D4) relative to the wind direction; (b) schematic vertical section illustrating the location of the LVDT and the measured displacement $\Delta disp$; (c) picture of one installed LVDT sensor (image courtesy of Nadara)

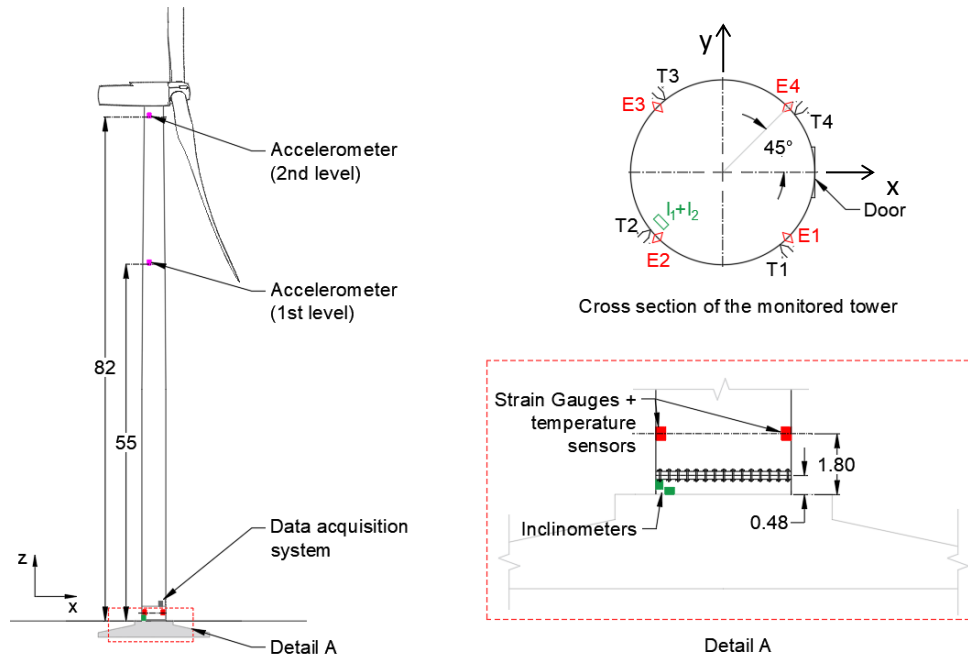


Figure 3.21 – Additional monitoring system implemented in phase two

The acquisition unit is a National Instruments CompactRIO with 3 modules responsible for reading the electrical signals of the sensors and generate hourly files with the recorded data. This unit is stored in a cabinet that also contains a power supply unit, a backup battery and a modem to send the data to a server located at FEUP. Figure 3.22 shows pictures of the monitoring system.

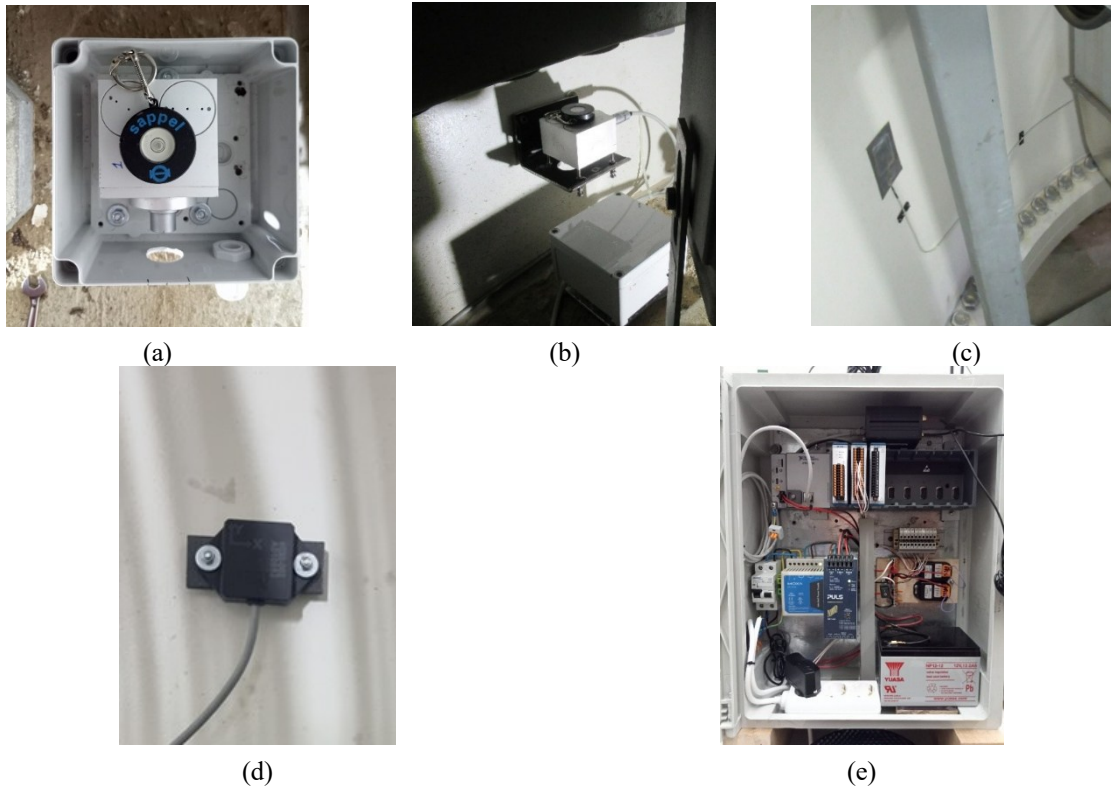


Figure 3.22 – Pictures of the monitoring devices: (a) Inclinometer in the pedestal (b) Inclinometers in the pedestal and tower; (c) Strain gauge; (d) Accelerometer; (e) Cabinet with the data acquisition system

3.3.2 DATA PROCESSING

3.3.2.1 STRAIN GAUGES

The data processing for the WTG 8 of Serra da Lousã, in which the continuous monitoring system was installed, followed the same procedures as for the WTG of Serra del Rei, described in Section 3.2. The only difference concerns the thermal output compensation curve adopted for WTG 8, given by:

$$\varepsilon_{(T/0)} = -95.5 + 6.33 \cdot T - 1.06 \times 10^{-1} \cdot T^2 + 5.05 \times 10^{-4} \cdot T^3 - 6.27 \times 10^{-7} \cdot T^4 \quad (3.10)$$

Figure 3.23 shows the 360° event occurred on October 10, 2023, while Figure 3.24 shows the deformation results at each strain gauge after applying the pre-processing procedures.

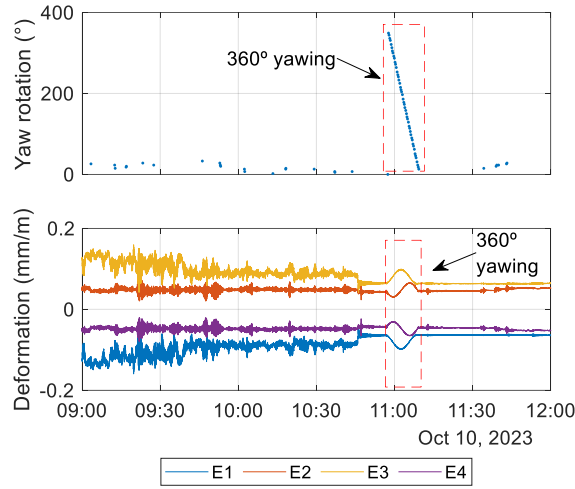


Figure 3.23 – Yaw direction during the event of 360° nacelle rotation and deformation of the tower base captured by the strain gauges

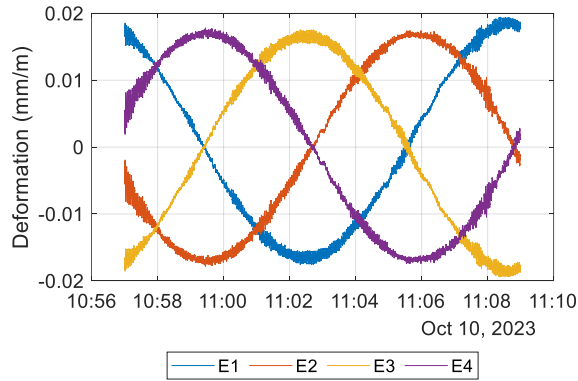


Figure 3.24 – Deformation of the strain gauges zeroed for the 360° yawing event

3.3.2.2 ACCELEROMETERS

In this case, the data recorded by the accelerometers is converted to acceleration in m/s^2 by the following expression:

$$\text{Acceleration } [\text{m/s}^2] = (\text{Sensor output} - 2.5) \cdot 9.81 \quad (3.11)$$

Figure 3.25 shows an example of the Root Mean Square (RMS) values of the time series recorded during the entire month of December 2023.

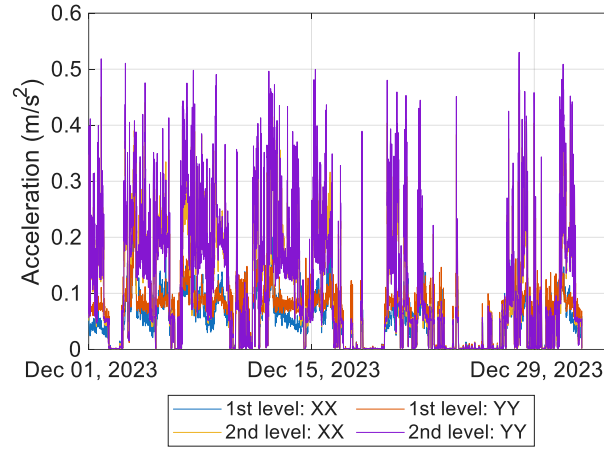


Figure 3.25 – RMS of the accelerations measured at the monitored wind turbine in Serra da Lousã

3.3.2.3 INCLINOMETERS

The assessment of the rotation involves the use of the data acquired from the inclinometers, applying the same calibration methodology performed for the strain gauges to establish a zero point for the data. Figure 3.26 illustrates the rotation of the pedestal and bottom tower section, during a 360° rotation of the nacelle. Figure 3.26 (a) shows the rotation of the pedestal about the global x and y axes (I_x and I_y). The recorded values remain very small, with amplitudes of the order of 10^{-3} degrees, indicating that the pedestal behaves almost rigidly under this operation and that the imposed yaw rotation does not induce significant tilting of the foundation block.

In contrast, Figure 3.26 (b) exhibits a clear cyclic pattern in the rotation measured at the tower bottom section, with amplitudes of approximately three times those recorded at the pedestal. A closer inspection of I_x and I_y indicates that the response is not axisymmetric. For an undamaged tower-foundation interface supported by approximately homogeneous ground stiffness, the two rotational components would be expected to behave as orthogonal principal responses, with the maximum of one component coinciding with the zero crossing of the other, reflecting an approximately uniform rotational stiffness around the circumference, as observed for the monitored undamaged foundation in Figure 3.9. In Figure 3.26 (b), however, the peak of I_x does not coincide with the zero of I_y .

This behaviour is consistent with a non-uniform rotational stiffness at the tower base. A plausible explanation is the presence of localized damage or stiffness degradation within a specific sector of the tower-foundation interface, for example due to cracking or the formation of gaps. Under such conditions, rotation about one global axis is accompanied by a non-negligible component about the orthogonal axis, and the measured time histories effectively capture the anisotropy induced by interface damage. This interpretation is further supported by the analyses presented in Chapters 5 and 6, where the presence of damage for this foundation typology is demonstrated to occur predominantly in the form of a gap developing at the tower foundation interface.

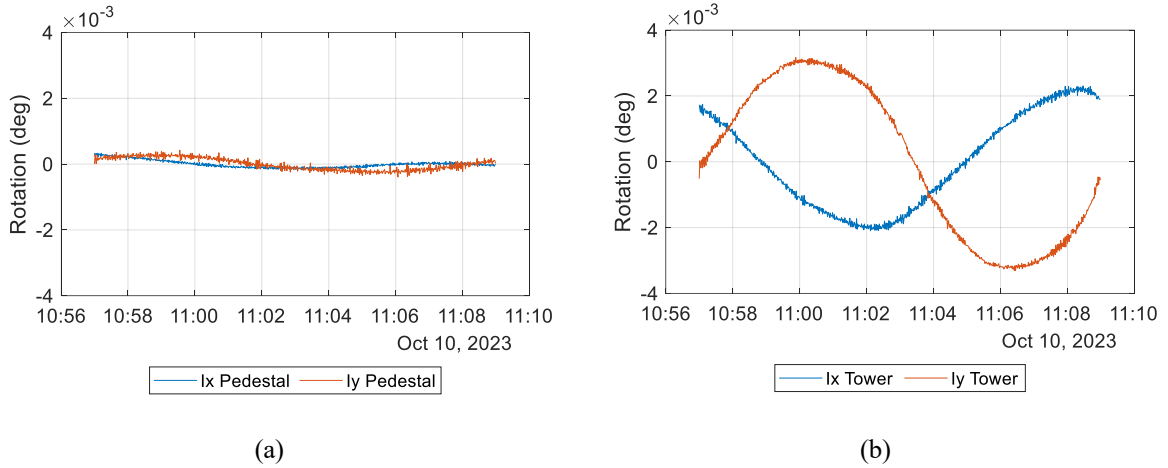


Figure 3.26 – Rotation of (a) pedestal and (b) tower bottom section, during the 360° rotation of the nacelle

3.3.3 RESULTS

3.3.3.1 PHASE 1

Figure 3.27 presents the measurements obtained during phase one of the experimental campaign, illustrating the relative vertical displacements between the bottom section of the tower and the surface of the pedestal for WTG 1, 9 and 11. These displacements were measured during forced shutdown events performed by the operator. Such shutdown events are a combination of feathering the blades with simultaneous activation of the mechanical brakes. The feathering of the blades consists of increasing the pitch angle by rotating the blades to reduce the thrust force applied by the wind in the rotor to a minimum close to zero. The mean wind speed during the shutdowns, recorded at each wind turbine, were respectively 15.2 m/s, 13.8 m/s and 12.6 m/s. The procedure for zeroing the data consists of shifting each signal by a constant value such that the post-event, stabilised plateau reached after the decay of the oscillations, but prior to the turbine resuming operation, was set to $\Delta Z = 0$.

A more detailed analysis of the recorded data reveals the following key phases:

- Wind turbines in operation (~ 0 -130 sec. in WTG 1, ~ 0 -120 sec. in WTG 9 and ~ 0 -100 sec. in WTG 11) with minimal variation in the vertical displacements that are primarily due to turbulence in the wind flow;
- Forced shutdown performed by the operator through the pitching of the blades and simultaneous activation of the mechanical brakes;
- Damped oscillation (until approximately 300 sec.) that follows the shutdown event, where the turbine entered a transient oscillatory phase, during which the amplitude of the vertical displacements gradually decreases due to damping effects.
- Standstill, with the blades feathered, keeping the rotor stationary.
- Start-up (~ 380 -530 sec. in WTG 1, ~ 450 -580 sec. in WTG 9 and ~ 380 -400 sec. in WTG 11) characterized by the initiation of the rotor rotation.

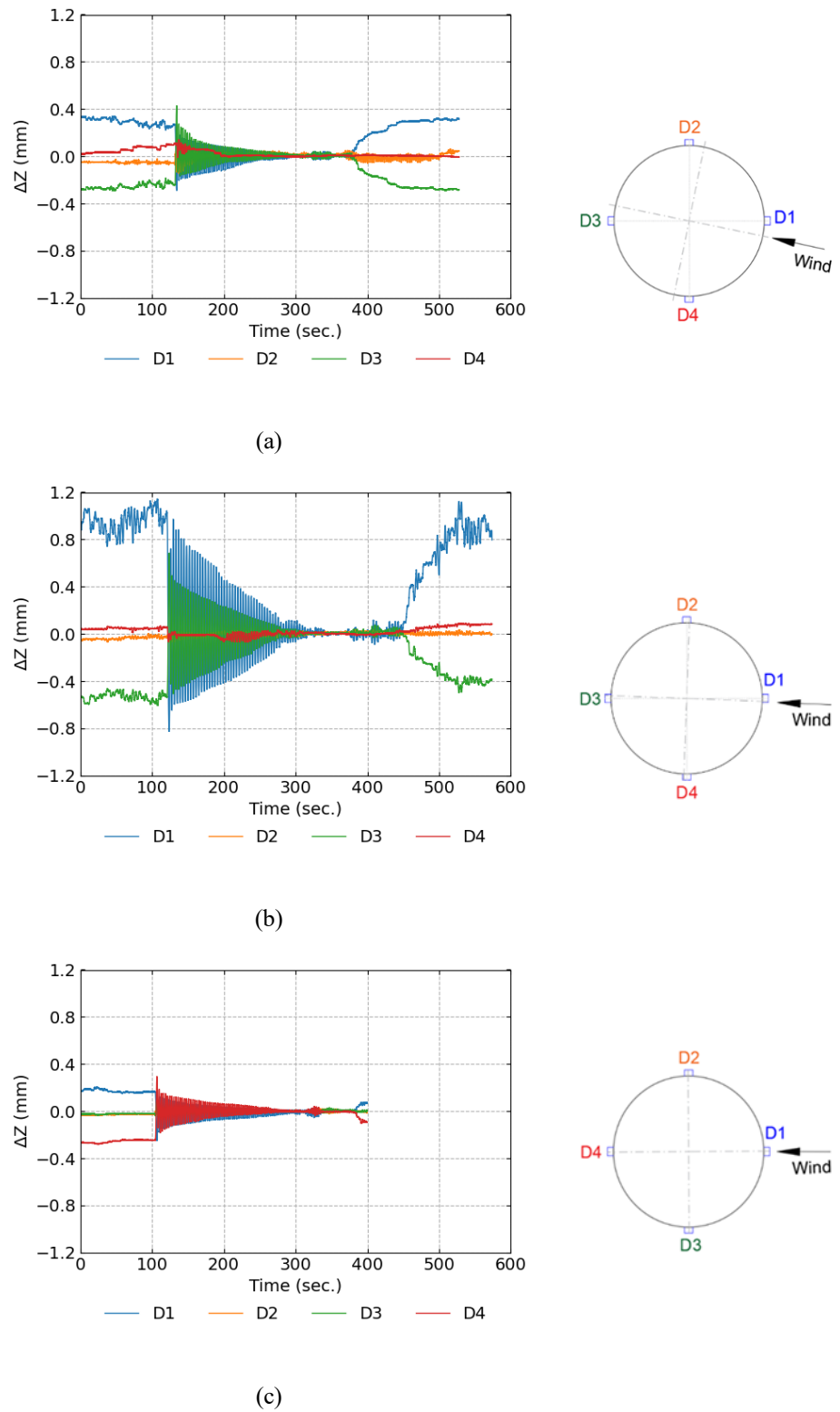


Figure 3.27 – Relative vertical displacement between the tower bottom section and the surface of the pedestal, recorded in 2020: (a) WTG 1; (b) WTG 9; (c) WTG 11

A comparative analysis of the forced shutdown events reveals distinct behavioural differences between the monitored turbines. WTG 9 exhibits a significantly higher maximum relative vertical displacement

of 1.80 mm, compared to 0.67 mm and 0.54 mm for WTG 1 and WTG 11, respectively. Furthermore, while the WTG 1 and WTG 11 display symmetric oscillations, WTG 9 exhibits a pronounced asymmetry, with displacements on the tensile side exceeding those on the compressive side. This asymmetry is indicative of potential gaps between the T-flange and the surrounding concrete. Under normal operation, the mean position of the tower relative to the pedestal remains approximately stable, and the interface response can be idealised as predominantly elastic about a quasi-stationary equilibrium. During a forced shutdown, the aerodynamic and inertial loads change rapidly, exciting the first fore-aft mode and inducing cyclic opening and closing at the interface (in case of presence of gaps). If a continuous contact condition exists around the circumference, the relative vertical displacements measured on opposite sides of the tower would be expected to be broadly symmetric in magnitude (assuming that the materials are in elastic regime). By contrast, if a gap is present, the relative displacement on one of the sides comprises both the elastic deformation of the materials and the additional displacement due to the existing gap. Consequently, the relative vertical displacement can increase markedly on the side where the gap is larger, while the opposite side remains governed mainly by elastic deformation and contact closure. If the gap is non-uniform circumferentially, this mechanism naturally produces an asymmetric displacement response as the one observed in WTG 9.

To examine whether this asymmetric interface behaviour is accompanied by measurable changes in the global dynamic properties during the same events, the tower natural frequencies were identified from spectral analysis of the measured signals. The power spectral density was estimated using an FFT-based approach (Welch method), in which the continuous records were divided into segments of 40 seconds with 75% overlap between consecutive segments, resulting in a frequency resolution of approximately $\Delta f = 1/40 \approx 0.025 \text{ Hz}$. Given the limited duration of the analysis windows, the resulting spectral resolution is insufficient to detect frequency variations smaller than 0.025 Hz. Consequently, a complementary time-domain approach was employed to validate the natural frequencies during the shutdown events. This method involved determining the instantaneous frequency based on the time intervals between consecutive local maxima (peak-to-peak periods) of the free-decay response.

The frequencies identified from the shutdown records are summarized in Table 3.3 for WTG's 1, 9 and 11. The first mode natural frequencies are closely grouped (0.33 - 0.34 Hz), with WTG's 1 and 11 presenting the same values (0.34 Hz), while WTG 9 exhibits a marginally lower frequency (0.33 Hz). Despite the displacement asymmetry observed for WTG 9, the present analysis did not detect any significant change in the first fore-aft natural frequency within the achieved spectral resolution and the adopted time-domain verification procedure.

Table 3.3 – Natural frequencies (1st mode FA) of the wind turbines 01, 09 and 11 monitored with LVDT sensors in 2020

WTG #	Frequency, f (Hz)
01	0.34
09	0.33
11	0.34

3.3.3.2 PHASE 2

Figure 3.28 presents the data with 10-minute resolution, collected throughout December 2023, to illustrate results obtained from the comprehensive monitoring system implemented during phase 2 in WTG 8. Only the plot for the month of December is shown here for illustration purposes. The data recorded by the different sensors is consistent, showing that the events are well reproduced by the entire system. The synchronization of the data suggests that the system operates effectively, and the calibration of the sensors is accurate, ensuring the integrity and reliability of the recorded measurements.

The first plot (top) presents the deformation data recorded by the strain gauges located at the bottom section of the steel tower. The data, showing average deformation values, exhibits periodic fluctuations that correspond with the operational conditions and external loading. The consistent patterns of the deformation demonstrate that the monitoring system effectively captures reliable strain responses throughout the observed period, offering insight into the structural dynamics under varying loads and environmental conditions.

The second plot (middle) presents the root mean square (RMS) values of the accelerometer data from the two sections of the tower. The RMS values, shown for both the first (blue and red) and second (yellow and purple) measurement levels, illustrate the intensity of vibrations at different heights of the tower.

Finally the plot (bottom) presents the rotation recorded by the inclinometers positioned on the top surface of the pedestal and at the tower bottom section. As expected, the rotations recorded by the inclinometer located in the tower are significantly higher than the rotations recorded by the inclinometer positioned in the pedestal.

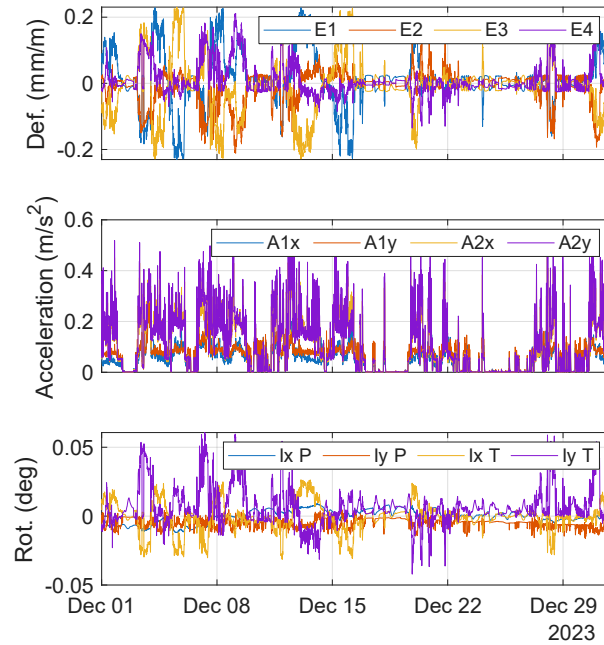


Figure 3.28 – Time series with 10-minute resolution for the entire month of December 2023 with averaged strain gauges deformation (top plot), accelerometers RMS (mid plot) and averaged inclinometers rotations (bottom chart) [P – pedestal; T – tower bottom]

Similarly to Serra del Rei (Section 3.2.3.2), Figure 3.29 presents Campbell diagrams obtained from fore-aft accelerometer measurements at WTG 8 in the Serra da Lousã wind farm for two representative

monitoring periods. In both cases, the first and second tower natural frequencies are consistently identified as $f_1 = 0.34 \text{ Hz}$ and $f_2 = 2.80 \text{ Hz}$, respectively. These experimentally identified frequencies are subsequently adopted as reference targets for the development and calibration of the numerical wind turbine model presented in Chapter 6.

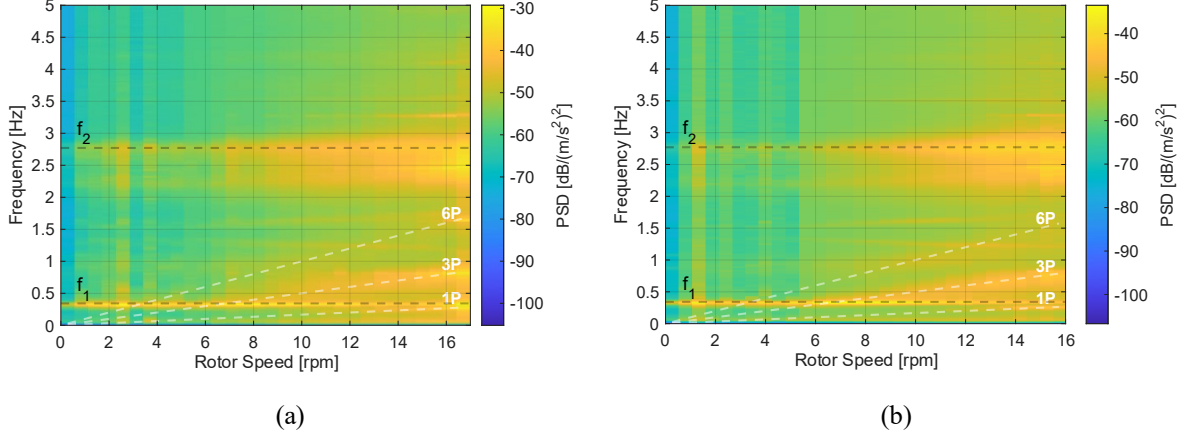


Figure 3.29 – Campbell diagram of the fore-aft eigenfrequencies identified from accelerometer measurements at WTG 8 in Serra da Lousã: (a) December 2023; (b) August 2024

3.3.3.2.1 SHUTDOWN EVENT

The installation of the strain gauges and temperature sensors at the bottom section of the tower enabled the evaluation of the stress resultants at the base of the tower, specifically the bending moment. The longitudinal strains obtained from the strain gauges E3 and E4 are used to calculate the bending moment along the rotor axis (F-A), following the same procedure as described in [10] and in Section 3.2 for the wind turbine located in Serra del Rei. Figure 3.30 shows the coordinate system used to evaluate the bending moments in this study.

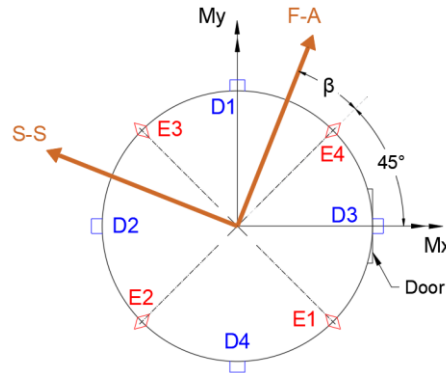


Figure 3.30 – Coordinate system of the tower cross-section with the strain gauges (E_i) and LVDT sensors (D_i) position (the β angle depends on the rotor yaw)

The corresponding fore-aft (F-A) bending moment is given by:

$$M_{F-A} = \frac{E \cdot I}{R} \cdot [\epsilon_4 \cdot \cos(\beta + 45) + \epsilon_3 \cdot \sin(\beta + 45)] \quad (3.12)$$

Figure 3.31 shows the bending moment and relative vertical displacements, evaluated by the LVDT sensors, between the tower and the pedestal during two forced shutdown events performed on WTG 8,

which occurred on 24/10/2023 and 11/10/2024, respectively. During these events, the average wind speed recorded was approximately 15 m/s.

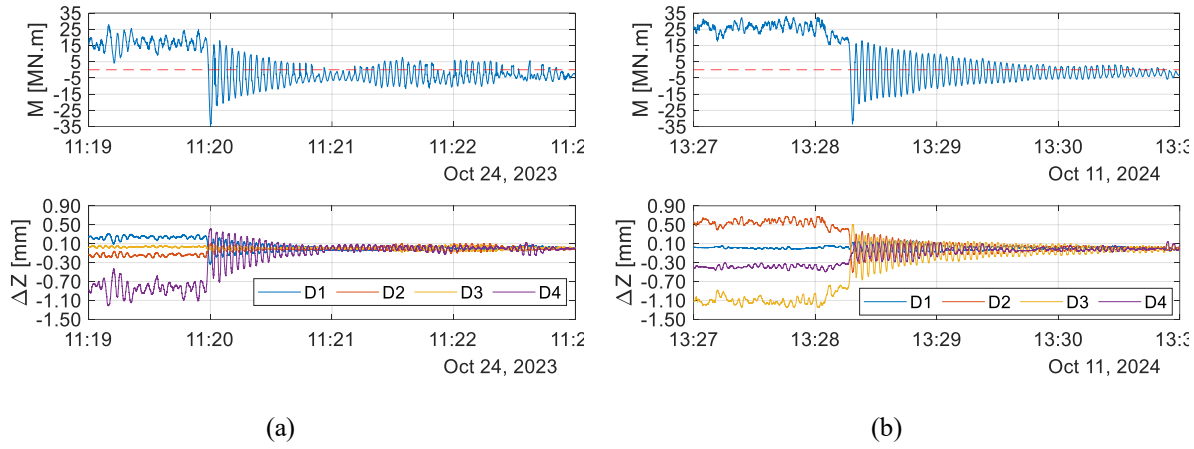


Figure 3.31 – Comparison between bending moment and relative vertical displacement between the pedestal and the tower bottom section during the shutdown events occurred on: (a) 24/10/2023; (b) 11/10/2024

The maximum amplitude variation of the relative vertical displacements recorded during the shutdown events occurred in 2023 and 2024 was 1.34 mm and 1.33 mm, respectively. For the event in 2023, the maximum displacement ranged from -0.92 mm to 0.42 mm, while for the event in 2024 it ranged from -0.83 mm to 0.50 mm. In both cases, the maximum amplitude variation of the bending moment was approximately 50 MN.m, decreasing from 17 MN.m, before the start of the blade pitching and mechanical braking, reaching a negative peak value of -33 MN.m.

To enable a direct comparison between monitoring periods, Figure 3.32 summarises the LVDT sensor layout together with the wind direction recorded during each event, and reports the corresponding maximum and minimum relative vertical displacements measured by the sensors aligned with the wind direction during the WTG 8 shutdowns in 2023 and 2024.

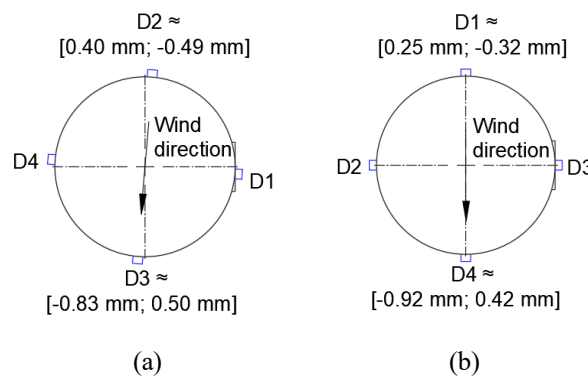


Figure 3.32 – Location of the LVDT sensors, relative vertical displacements and wind direction during the shutdown events in: (a) 10/2024; (b) 10/2023

An analysis of the recorded displacements over time reveals that the maximum oscillation amplitudes on the leeward side, measured by sensor D4 in 2023 and D3 in 2024, are approximately equal. In contrast, in the windward side, the sensors D1 in 2023 and D2 in 2024 show an increase in the maximum

vertical displacement, increasing from 0.57 mm to 0.89 mm. This observed increase in displacement amplitude on the windward side may indicate progressive degradation of the foundation, potentially due to an increase in the gap width at the interface between the T-flange and the surrounding concrete. Alternatively, this phenomenon may indicate that the tower has settled into an equilibrium position that differs from the state observed prior to the shutdown, thereby changing the baseline of the measured oscillations.

In addition to the relative vertical displacements discussed previously, the rotational behaviour at the steel tower base section and at the concrete pedestal was also examined using the data from the inclinometers. Figure 3.33 shows the relationship between the rotation of the pedestal and tower base section with the applied bending moment during the shutdown events of 24/10/2023 and 11/10/2024. The estimation of the moment-rotation curves is essential for characterising the rotational behaviour of the base of the tower under cyclic loading. These curves provide the knowledge for defining realistic boundary conditions to be incorporated in the numerical model of the wind turbine. Specifically, the observed rotational response reflects the combined effects of elastic deformation, interface friction, and potential gap development, which cannot be accurately captured through conventional rigid or linear-elastic assumptions, commonly used in the design of wind turbines.

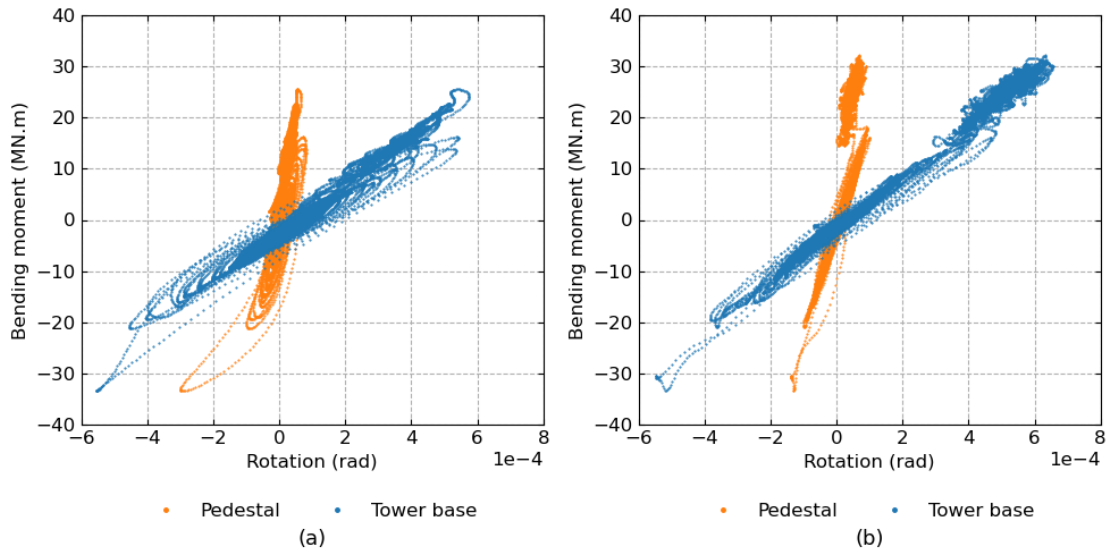


Figure 3.33 – Moment-rotation curves recorded by the inclinometers during the shutdown events that occurred on: (a) 24/10/2023; (b) 11/10/2024

The relationship between rotation and bending moment presented in Figure 3.33 revealed a hysteretic behaviour with energy dissipation under cyclic loading. As it will be discussed in detail in Chapter 5, this energy dissipation can be attributed to the friction between the concrete and the steel surfaces, especially in the presence of gaps. Examining the curves, the pedestal and the tower base exhibit different rotational stiffness. For a given bending moment, the pedestal response is characterised by very small rotations, whereas the tower base undergoes substantially larger rotations, indicating relative rotation between the tower and foundation, consistent with the presence of gaps.

The data from both shutdown events (Figure 3.33 (a) and Figure 3.33 (b)) illustrate consistent hysteretic behaviour. Nevertheless, some differences can be observed between the two events, which may be

associated with variations in wind conditions during the tests, differences in the shutdown procedures, different initial position of the tower in the two events or even changes in the gap configuration.

A particularly relevant feature is the offset observed in the cloud of points for the tower base in Figure 3.33 (b). For higher bending moments, the points tend to align along a well-defined branch, which may be interpreted as the response of the system in one quasi-stable contact configuration. When the emergency shutdown is triggered, the rapid change in aerodynamic loading causes the tower to oscillate freely. During this transient phase, the frictional resistance at the tower-foundation interface is likely to be exceeded locally, allowing slip and possible rocking over the uneven contact surface. As a result, the tower base settles into a different contact pattern, associated with a modified distribution of gaps and contact pressures. In the moment-rotation plane, this manifests as a second cluster of points with a similar slope but shifted in rotation, that is, an apparent offset of the cloud of points relative to the pre-shutdown branch. This interpretation is plausible for a foundation that already presents evidence of gap formation at the tower-foundation interface.

The verification of the first-mode natural frequencies was extended to WTG 8. The values obtained from the shutdown events recorded in 2023 and 2024 are summarised in Table 3.4. The identified frequencies are consistent with those obtained for the Phase 1 turbines and with the results previously presented in Figure 3.29. Consequently, these experimentally derived frequencies are adopted as reference targets for the calibration of the wind turbine numerical model presented in Chapter 6.

Table 3.4 – Natural frequencies (1st mode FA) of the WTG 8 monitored with LVDT sensors in 2023 and 2024

Frequency, f (Hz)	
2023	0.33
2024	0.34

3.4 CONCLUSIONS

The experimental campaigns described in this chapter establish a robust framework for the structural health monitoring of onshore wind turbines, covering both the AC and TFS foundation typologies. The implemented monitoring systems, comprising strain gauges, accelerometers, inclinometers, load cells, and LVDTs, provided comprehensive datasets for assessing operational behaviour and validating numerical models.

The long-term monitoring at Serra del Rei demonstrated the reliability of the instrumentation layout designed for the AC foundation solution. The data processing methodologies, particularly the calibration routines for strain gauges involving temperature compensation and 360° yawing cycles, proved effective in deriving accurate stress resultants. The experimental bending moments at the tower base showed a strong correlation with theoretical predictions based on thrust coefficients, validating the load estimation approach. Furthermore, the load cells monitoring the anchor bolts confirmed that the prestressing forces remained stable and consistent with the expected theoretical behaviour, providing a verified baseline for the numerical modelling of undamaged AC connections.

The campaign at Serra da Lousã offered critical insights into the structural performance of foundations with the TFS solution, specifically regarding the detection of interface damage. The measurement of

relative vertical displacements between the steel tower and the concrete pedestal proved to be a decisive indicator of the presence of gaps. In Phase 1, the significant asymmetry and magnitude of displacements observed in WTG 9 (reaching 1.80 mm) compared to other turbines clearly identified the loss of contact at the interface.

In Phase 2, the comprehensive monitoring system confirmed that the relative vertical displacements on the traction side exceeded linear elastic predictions, providing direct evidence of gap opening. The analysis of the moment rotation response further revealed a non-linear hysteretic behaviour at the tower base during shutdown events, characterised by reduced rotational stiffness and energy dissipation. A particularly relevant feature is the offset observed in the cloud of points for the tower base in Figure 3.33 (b). At higher bending moments, the response tends to align along a well-defined branch, which may be interpreted as a quasi-stable contact configuration. When the emergency shutdown is triggered, the rapid change in aerodynamic loading induces free oscillations of the structure, during which frictional resistance at the tower-foundation interface is likely to be exceeded locally, enabling slip and possible rocking over an uneven contact surface. Consequently, the tower base may settle into a different contact pattern, associated with the existence of gaps. In the moment rotation plane, this behaviour manifests as a second cluster of points with a similar slope but shifted in rotation, that is, an apparent offset relative to the pre-shutdown branch.

Collectively, the experimental evidence gathered in this chapter constitutes a valuable empirical basis for the work developed in the remainder of the thesis. The results presented here are revisited and explored further in Chapters 4, 5, and 6, where they support the calibration and validation of the non-linear numerical models and enable a more accurate simulation of the complex interactions between the tower and the foundation.

4 NONLINEAR NUMERICAL MODELLING OF ONSHORE WIND TURBINE FOUNDATIONS

4.1 INTRODUCTION

This chapter presents a detailed and comprehensive modelling strategy developed to reproduce the behaviour of onshore wind turbine foundations, incorporating the specific details of the connection between the tower and the foundation. The study focuses on comparing the performance of the two most common solutions for this connection: prestressed anchor cage (AC) and embedded tower foundation section (TFS).

The footings are heavily reinforced elements, with the structural design of the pedestal, namely the connection of the tower to the footing, requiring special attention. In the AC system, the flow of forces is clear, as schematically depicted in Figure 4.1 (a): the anchor bolts bring the tensile forces down to the base of the footing, from where an equilibrium system can be developed involving the flexural and shear reinforcement of the tapered slab. On the other hand, as shown in Figure 4.1 (b), in the solution with TFS, the force in the tensile side of the tower is balanced by compression stresses developing in the upper surface of the embedded flange pushing the concrete upward. This force can only be brought down to the base of the footing by vertical suspension reinforcement. Equilibrium requires radial reinforcement crossing the TFS. Being a passive system, the activation of the reinforcement forces implies internal concrete cracking. The usual design procedures rely on equilibrium models, where compatibility is not explicitly checked and the implications on the rotational stiffness of the system are not addressed.

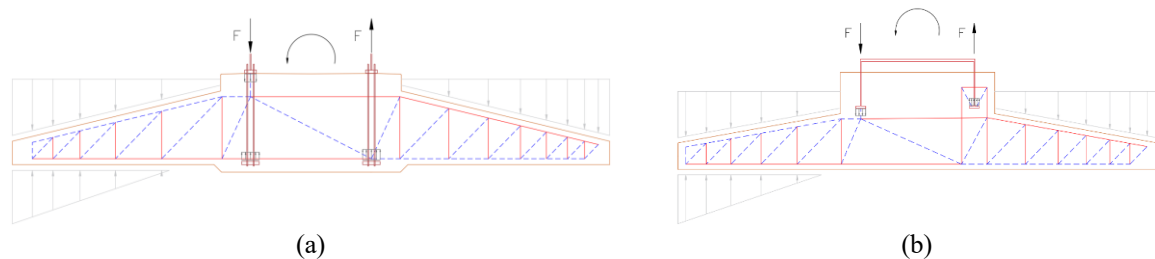


Figure 4.1 – Simple 2D strut-and-tie model for wind turbine foundation design subjected to bending moment. Dashed lines represent concrete struts and continuous lines the steel ties: (a) AC foundation; (b) TFS foundation (strut ---; tie —)

In current engineering practice, the majority of onshore wind turbine foundations are designed using the Finite Element Method (FEM) with the foundation typically discretized by shell elements. Linear elastic behaviour is assumed for the concrete parts and no-tension elastic bedding is adopted to represent the soil/rock mass. Stress resultants, such as bending moments and shear forces, are readily obtained allowing for the design of the concrete tapered slab, including both flexural and shear reinforcement. This method is well established and easy to implement [31]. The analysis is relatively fast and can accommodate multiple load cases without significant time consumption. However, the interface between the steel tower and the concrete foundation is not reproduced and there is no information on inelastic phenomena such as crack formation. The nonlinearities, except the one observed in the contact between the foundation and soil beneath, are not reproduced in this type of analysis, which can have a significant impact on the behaviour of the wind turbine, especially in the case of the foundations with embedded TFS.

For this reason, it is important to develop numerical models that accurately reproduce these nonlinearities and design methodologies that can take advantage of this type of structural analysis. Several studies have explored different aspects of this challenge. For instance, simplified methods and numerical analyses for predicting the performance of circular wind tower foundations have been compared [32], and numerical models simulating material nonlinearities have been developed and validated against experimental 1:15 scaled models [33]. The study presented in [20] investigates the mechanical behaviour of a prefabricated dense ribs foundation (PDRF) for onshore wind turbines under complex loading conditions using a 1:6 scaled model experiment. Further research was also developed on the fatigue behaviour of concrete around the embedded ring through numerical analysis [34], on monitoring the structural behaviour of an onshore foundation using continuous strain measurements [35], and on the failure of foundations under extreme typhoon conditions [36]. However, the force transfer between the steel tower and the concrete footing requires further research due to its impact on the rotational stiffness, and consequently on the operational behaviour of the wind turbines, and on the safety of the system.

The analysis in this chapter examines the behaviour of these systems under service conditions and at the ultimate limit state with different levels of modelling complexity, providing insights into their operational performance and load-bearing capacities. Furthermore, the developed models are validated against experimental results under operational conditions presented in Chapter 3 to ensure that the simulations accurately reflect the observed behaviour and to enhance the reliability of the findings. The combination of advanced modelling with rarely available monitoring results permits the proposal of a duly validated and original modelling strategy of great relevance for the evaluation of the structural behaviour of onshore wind turbine foundations.

A final objective of this chapter is the analysis of stress redistribution. As previously established, the design of these foundations commonly relies on linear stress analysis. The stresses used for design are derived from loads typically determined under the assumption of unidirectional wind flow throughout the entire lifetime of the wind turbine. The foundation design is based on these peak stresses, and the resulting reinforcement is often applied uniformly without taking into account stress redistribution. Therefore, to investigate the consequences of this procedure, a comparison is presented between the AC foundation simulated using linear stress analysis and the same foundation modelled using nonlinear stress analysis.

4.2 NUMERICAL MODELS

4.2.1 MODELLING STRATEGY

Two different types of reinforced concrete foundations were considered in the analysis, which are representative of the two real case-studies already described in detail in Chapter 3: (1) circular foundation (Figure 4.2 (a)) with 18 m diameter and 2.6 m height at the pedestal, resulting in 345 m³ of concrete and 49 tons of steel, connected with the tower through the post-tensioned AC embedded in the concrete; (2) octagonal foundation (Figure 4.2 (b)) with 16.5 m longest diagonal and 3.89 m height at the pedestal, resulting in 372 m³ of concrete and 36 tons of steel, connected with the tower through the TFS embedded in the concrete.

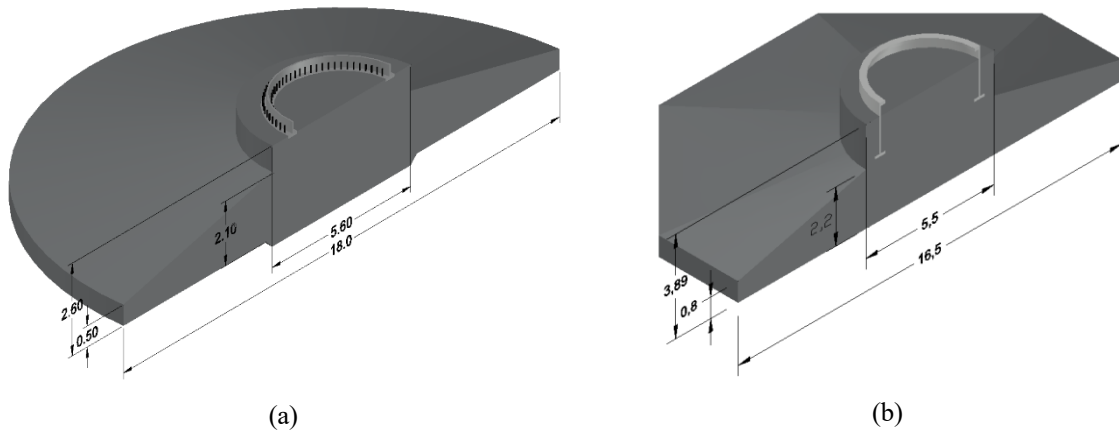


Figure 4.2 – Wind turbine foundation dimensions (units in meters): (a) AC system; (b) TFS system (Units: m)

For each foundation, two different finite element modelling approaches are used: an initial analysis with shell elements representing the slab, as per current engineering practice; and a detailed analysis employing 3D solid elements.

As previously mentioned, thick shell finite element models are usually employed to assist the design of onshore wind turbine foundations. Figure 4.3 illustrates the finite element mesh used to simulate the foundation with AC system employing shell elements of varying thickness.

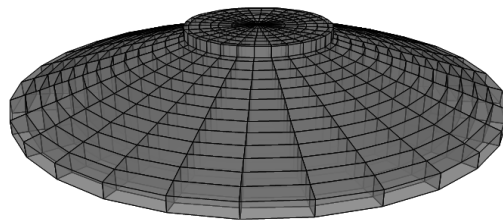


Figure 4.3 – Finite element mesh using shell elements with variable thickness

To overcome the limitations of the shell element approach and enable a more detailed analysis of the steel-concrete interface, a second simulation was conducted using 3D solid elements. In this simulation, the same wind turbine foundations were modelled using DIANA FEA 10.4 [37], as represented in Figure 4.4 and Figure 4.5. The concrete footing and the base of the steel tower were discretized using an eight-node isoparametric solid brick element with a 2x2x2 Gaussian integration scheme. In a first stage,

concrete and reinforcement were treated as linear elastic materials. In the footing with the AC connection, the anchor bolts are modelled as regular truss elements unbonded from the surrounding concrete and connecting the embedded flange with the bottom tower section. The soil beneath the slab concrete is represented by no-tension interface elements distributed along the bottom slab area, thereby allowing for detachment at the interface.

All reinforcement bars were explicitly modelled using an embedded formulation, in which the nodal displacements of the two-node bar elements were determined from the nodal displacements of the embedding concrete elements via interpolation functions. This assumption implies perfect bond, and therefore bond-slip was not modelled for the reinforcement in general. An exception was made for the bars crossing the TFS holes, where the radial reinforcement was explicitly simulated with bond-slip in order to capture the relative displacement between concrete on either side of the steel plate and to reproduce the associated pull-out mechanisms. This was achieved through a numerical strategy involving the use of auxiliary elements with negligible stiffness. This approach facilitated the embedding of the reinforcement without kinematic constraints from the displacements of the steel tower, thereby ensuring that the displacements of the bars remained independent of the steel components.

Interface elements with null tensile strength and Mohr-Coulomb frictional behaviour are employed to simulate the interaction between the steel and concrete. These elements allow for separation and sliding at the interface between the anchor cage flanges and concrete (in the AC foundation) and between the tower foundation section and surrounding concrete (in the TFS foundation). This approach with 3D solid elements and interface elements enables a more realistic representation of the complex interaction between the steel tower and the concrete foundation.

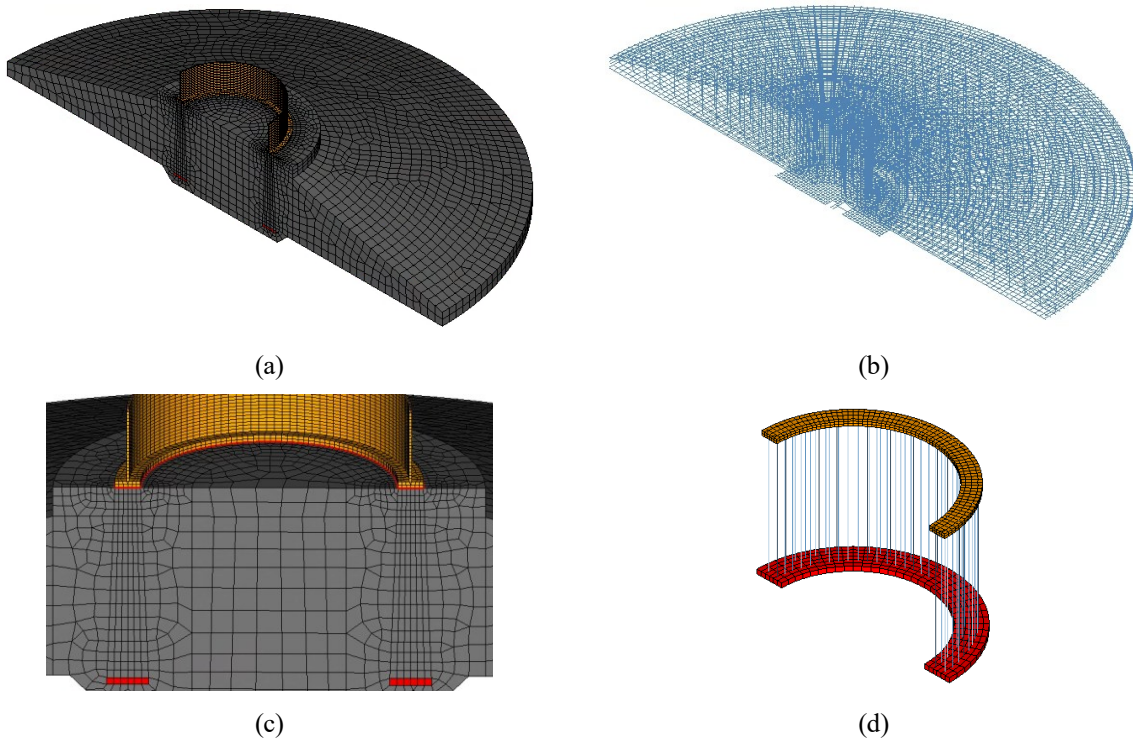


Figure 4.4 – AC foundation: (a) finite element mesh; (b) embedded reinforcement finite elements; (c) interfaces between concrete and steel flanges highlighted in red; (d) detail of anchor bolts and flanges

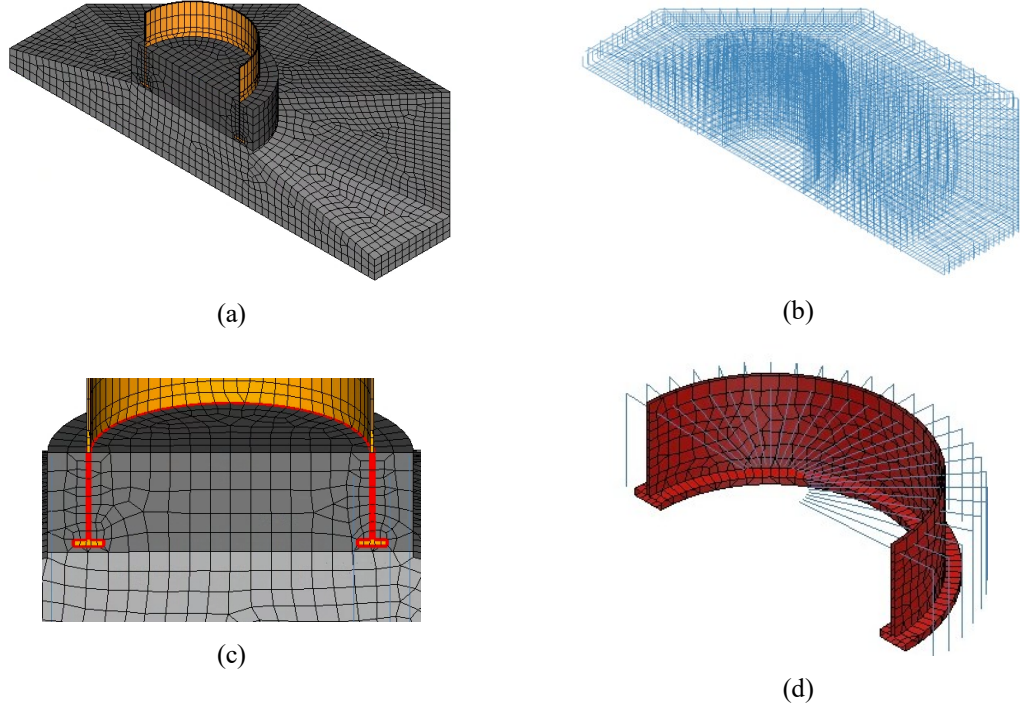


Figure 4.5 – TFS foundation: (a) finite element mesh; (b) embedded reinforcement finite elements; (c) interfaces between concrete and steel flanges highlighted in red; (d) interface finite elements with the detail of the horizontal rebars crossing the TFS

4.2.2 CONSTITUTIVE MODELS AND MATERIAL PROPERTIES

The concrete behaviour considered in this work follows the total strain smeared crack model with rotating cracks [37], [38] and [39]. This type of model describes the stress as a function of the total strain and a set of state variables that keep track of the loading history and previous damage. In the rotating crack model, the stress-strain relations are evaluated in the principal directions of the total strain tensor. A crack is assumed to form when the maximum principal tensile stress exceeds the tensile strength. In this model, the cracks are actually fictitious cracks, whose directions are continuously rotating so that the normal to the crack is aligned with principal tensile strain. The stresses are evaluated in the local crack coordinate system and the crack shear stresses are null. Therefore, collinearity between stresses and strains is enforced by the model. One of the advantages of the rotating crack model lies in its simplicity in the sense that only the compressive and the tensile behaviour of concrete need to be described.

The total strain vector, ε_{xyz} , in the element coordinate system, xyz , is updated with the strain increment, $\Delta\varepsilon_{xyz}$, according to the expression (4.1).

$$\varepsilon_{xyz,(t+\Delta t)} = \varepsilon_{xyz,(t)} + \Delta\varepsilon_{xyz,(t+\Delta t)} \quad (4.1)$$

The transformation to the crack direction is done using the transformation matrix T as follows:

$$\varepsilon_{nst,(t+\Delta t)} = T \cdot \varepsilon_{xyz,(t+\Delta t)} \quad (4.2)$$

The state variables are grouped in a vector $\mathbf{k}$ and are updated after every converged load step. The stress tensor is then evaluated in the crack coordinate system as a nonlinear function of the total strains and state variables:

$$\sigma_{nst,(t+\Delta t)} = f(\varepsilon_{nst,(t+\Delta t)}, \mathbf{k}_{t+\Delta t}) \quad (4.3)$$

The final transformation to the element coordinate system is given by the following expression:

$$\sigma_{xyz,(t+\Delta t)} = T^T \cdot \sigma_{nst,(t+\Delta t)} \quad (4.4)$$

The tensile behaviour of the concrete is described using the exponential softening curve based on the fracture energy, G_f , and crack bandwidth, h (Figure 4.6 (a)). The crack bandwidth can be determined, among others, by the Govindjee's projection method [40]. In this projection method, the crack bandwidth is considered as the projected length of the element containing the crack in the cracked plane. Therefore, the crack bandwidth depends not only on the element dimension but also on the crack orientation and element aspect ratio. In the smeared crack model, the cracks are represented in the continuum element by changing only the constitutive properties of the element, keeping the element as a continuum. After crack formation, the displacement field is a C^0 class function.

The compressive behaviour is represented by the parabolic curve, schematically illustrated in Figure 4.6 (b), with a post peak branch dependent on the compressive fracture energy, G_c . This allows reducing the mesh dependency of the results when localization of compressive strains occurs in the post/peak domain.

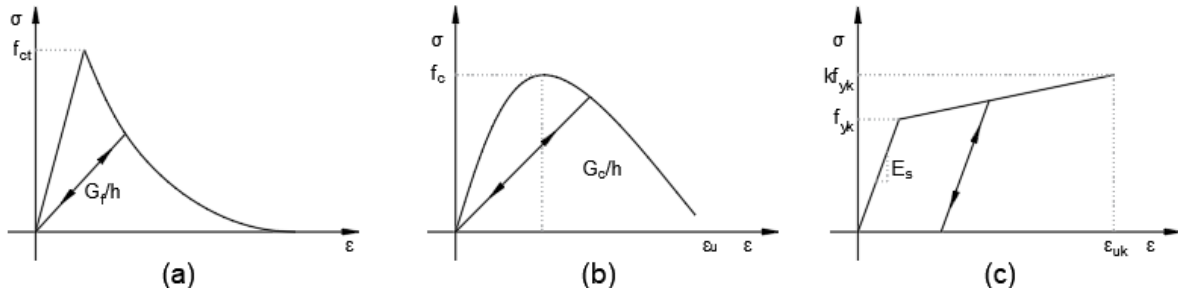


Figure 4.6 – Stress-strain relationship curves: (a) concrete tensile behaviour (adapted from [38]); (b) concrete compressive behaviour (adapted from [38]); (c) reinforcement behaviour (adapted from [24])

It is well known that the concrete compressive stress-strain relation is influenced by the confinement. When confined laterally, concrete show high compressive strength with larger associated strains. For this work, the increase of strength in a multiaxial stress state is evaluated using the four-parameter Hsieh-Ting-Chen failure surface, which is defined by the expression (4.5):

$$f = 2.0108 \cdot \frac{J_2}{f_{cc}^2} + 0.9714 \cdot \frac{\sqrt{J_2}}{f_{cc}} + 9.1412 \cdot \frac{f_{c1}}{f_{cc}} + 0.2312 \cdot \frac{I_1}{f_{cc}} - 1 \quad (4.5)$$

with I_1 the first invariant of the concrete stress tensor, J_2 the second invariant of the deviatoric stress tensor, f_{c1} the maximum concrete stress and f_{cc} the concrete compressive strength.

When concrete cracks, confinement is lost and the tensile strains develop perpendicular to the compressive direction are responsible for the reduction of the concrete compressive strength. Vecchio

and Collins [41] proposed a reduction factor, β , for the concrete compressive strength as a function of the lateral cracking, as show in Figure 4.7 and adopted in the present work.

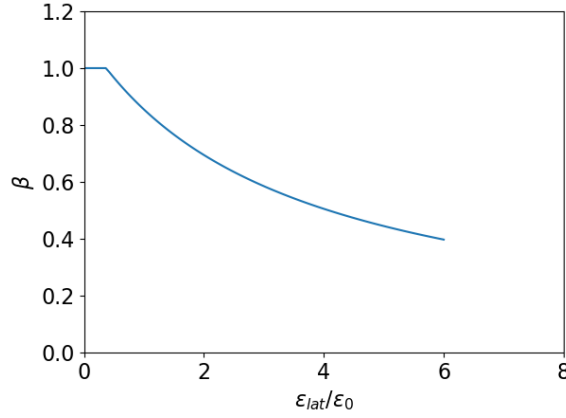


Figure 4.7 – Reduction factor due to lateral cracking proposed by [41]

Material parameters:

Characteristic values of the material strength parameters were adopted according to the semi-probabilistic approach applicable to the safety evaluation of concrete structures using nonlinear finite element analysis, following the recommendations in the prEN 1992-1-1:2022, Annex F [42]. Specifically, the characteristic values are determined assuming lognormal variables with known mean values and coefficients of variation. As for the concrete and steel elasticity moduli, the mean values were adopted.

The reference mean value of the concrete compressive strength, $f_{c,m}^*$, was determined as follows:

$$f_{c,m}^* = \mu_{f_{c,R}} \cdot f_{c,k} \quad (4.6)$$

with $f_{c,k}$ being the characteristic value of the compressive strength corresponding to the specified strength class and $\mu_{f_{c,R}} = 1.12$ being the recommended bias factor. The characteristic value of the concrete compressive strength used in the calculations, $f_{c,k}^*$, is determined by:

$$f_{c,k}^* = f_{c,m}^* \cdot e^{(-1.645 \cdot v_{f_{c,R}})} \quad (4.7)$$

with $v_{f_{c,R}} = 0.16$ being the coefficient to variation of the concrete compressive strength.

The reference mean value of the concrete tensile strength, $f_{ct,m}^*$, was determined as follows:

$$f_{ct,m}^* = \mu_{f_{ct,R}} \cdot 0.7 \cdot 0.3 \cdot f_{c,k}^{2/3} \quad (4.8)$$

with $\mu_{f_{ct,R}} = 1.28$ being the recommended bias factor. The characteristic value of the concrete tensile strength used in the calculations, $f_{ct,k}^*$, is determined by:

$$f_{ct,k}^* = f_{ct,m}^* \cdot e^{(-1.645 \cdot v_{f_{ct,R}})} \quad (4.9)$$

with $v_{f_{ct,R}} = 0.22$ being the coefficient to variation of the concrete tensile strength.

The variability of the fracture energy was assumed to be identical as for the concrete tensile strength, $f_{ct,k}$ [42]. In that regard, the fracture energy used for the calculations was determined according to equation (4.10):

$$G_{f,k}^* = \mu_{f_{ct},R} \cdot 0.7 \cdot G_{f,m} \cdot e^{(-1.645 \cdot v_{f_{ct},R})} \quad (4.10)$$

with $G_{f,m}$ determined based on the expression defined in the MC 2010 [23] as follows:

$$G_{f,m} = 73 \cdot f_{c,m}^{0.18} \quad (4.11)$$

Steel reinforcement was modelled considering a plasticity model with strain-hardening, as shown in Figure 4.6 (c) with a ratio between the tensile strength and yielding strength, $k = 1.08$. The characteristic value of the yield strength of the reinforcement used in the calculations, $f_{y,k}^*$, is determined by:

$$f_{y,k}^* = f_{y,k} \cdot \mu_{f_{y,R}} \cdot e^{(-1.645 \cdot v_{f_{y,R}})} \quad (4.12)$$

with $f_{y,k}$ being the characteristic value of the yielding strength corresponding to the specified strength class, $\mu_{f_{y,R}} = 1.06$ being the recommended bias factor, and $v_{f_{y,R}} = 0.048$ being the coefficient to variation.

The mechanical properties of the concrete and steel characteristic values used in the nonlinear analysis are summarized in Table 4.1 and Table 4.2, respectively.

Table 4.1 – Characteristic properties of the concrete

Concrete properties	C45/55	C35/45	C30/37
Tensile strength, $f_{ct,k}^*$ (MPa)	2.37	2.00	1.81
Fracture energy in tension, $G_{f,k}^*$ (kN/m)	0.093	0.090	0.088
Compressive strength, $f_{c,k}^*$ (MPa)	38.7	30.1	25.8
Fracture energy in compression, G_{ck} (kN/m)	41.88	40.34	39.45
Young's modulus, E_{cm} (GPa)	36	34	33
Poisson's ratio, ν		0.2	

Table 4.2 – Characteristic properties of the steel

E_s (GPa)	$\varepsilon_{y,k}$ (%)	$\varepsilon_{u,k}$ (%)	$f_{y,k}^*$ (MPa)	$f_{t,k}$ (MPa)
200	0.244	5.0	489	528

The bond-slip relation defined in the MC 2010 [23], schematically represented by the Figure 4.8 was adopted.

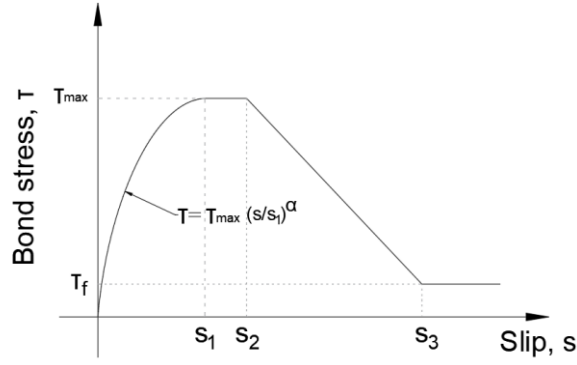


Figure 4.8 – Bond stress-slip relationship for monotonic loading (extracted from [23])

The parameters describing the bond-slip relation between reinforcement and concrete, as shown in Table 4.3, are derived from [23], assuming pull-out failure mode and non-ideal bond conditions.

Table 4.3 – Properties for the bond-slip relation between concrete and reinforcement

s1 (mm)	s2 (mm)	s3 (mm)	α	f_{cm} (MPa)	τ_{max} (MPa)	τ_f (MPa)
1.8	3.6	15	0.4	53	9.1	3.6

where $\tau_{max} = 1.25 \cdot \sqrt{f_{cm}}$, $\tau_f = 0.4 \cdot \tau_{max}$ and s_3 the clear distance between ribs.

4.2.3 INTERFACE PROPERTIES

In both the shell and 3D solid element models, the soil beneath the concrete slab is represented by interface elements distributed along the bottom slab area following the Mohr-Coulomb failure criteria with null tension-cut-off, thereby allowing for the detachment at the interface. Compressive stiffness of the interface elements representing the soil was computed considering the properties of a semi-indefinite homogeneous medium with linear elastic behaviour and following the recommendations of DNV/RISO [22]. The properties adopted to evaluate the soil behaviour beneath the foundation are given in Table 4.4. They are representative of a rocky foundation, typical of many wind farms in Portugal located in mountain regions.

Table 4.4 – Soil interface properties for AC foundation

Solution	Young modulus, E_{soil} (MPa)	Normal stiffness, k_v (kN/m ³)	Tangential stiffness, k_h (kN/m ³)	Friction angle (°)	Cohesion (kPa)	Poisson's ratio
AC	1000	87 880	182 525	42	1	0.3
TFS	1000	102 606	217 082	42	1	0.3

The contact between steel and concrete was modelled using interface elements with 8 nodes (4 on each face) with the 2x2 Newton-Cotes integration scheme. The stiffness of the interface must be sufficiently high to minimize the unrealistic elastic interface displacements, but not too high to cause numerical instabilities. For these simulations, a frictionless and cohesionless interface is initially considered with linear elastic compressive stiffness 2×10^{12} kN/m³ in the normal plane and 2×10^8 kN/m³ in the tangential

plane. Null tensile strength is assumed, with the shear stiffness being also null when detachment occurs. The effects of friction and cohesion between concrete and steel are further analysed in Chapter 5.

4.2.4 LOADS

The design loads of the wind turbine foundations are provided by the wind turbine manufacturers as stress resultants, located at the tower base. Due to confidentiality, the numerical values cannot be disclosed, and the results are normalized by the quasi-permanent loads. The different sets of loads are introduced in the numerical model as linear distributed forces along the circumference defined by the mean diameter of the tower bottom section, acting at the elements that represent the tower bottom section. A constraint was defined ensuring that the cross-section at which the linearly distributed forces are applied remains plane after the deformation. The self-weight of the reinforced concrete and soil are automatically evaluated by the software, considering the geometry and density of the materials. For the model using shell elements, the external forces from the tower and soil are applied as surface stresses.

4.2.5 SOLUTION TECHNIQUES

An incremental-iterative solution procedure following the regular Newton-Raphson method was adopted. The convergence criterion employed is based on the energy norm ratio:

$$\left| \frac{\delta u_i^T (f_{int,i+1} + f_{int,i})}{\Delta u_0^T (f_{int,1} + f_{int,0})} \right| \leq tol \quad (4.13)$$

where δu_i and $f_{int,i}$ are the incremental displacement and the internal force vectors at iteration i , respectively, and Δu_0 is the incremental displacement vector at the first iteration of the increment. A reasonable balance between accuracy and computational efficiency is usually achieved if the tolerance is set equal to $tol=10^{-4}$ [43]. The arc-length method was adopted for constraining the loading vectors within each incremental step. This improves the convergence rate and allows determination of local and global maxima in the moment-rotation curve. In the present work, a linearized constraint solution, known as the updated normal plane constraint method, was adopted.

4.3 ANALYSIS FOR SERVICEABILITY CONDITIONS

4.3.1 GENERAL

The objective of this section is to investigate the structural response of the two previously described foundation systems under normal operating conditions, that is, in a serviceability scenario. The analyses presented in this section were conducted assuming that the concrete and reinforcement behave as linear elastic materials, since no significant concrete cracking is expected under serviceability loads, and the materials (concrete and steel) are expected to remain within the linear regime. This is confirmed by the full material nonlinear analyses up to failure presented and discussed in Section 4.4. This approach enhanced computational efficiency while enabling: (1) to focus the investigation on the nonlinear behaviour of the interfaces between the steel elements and concrete and its effect on the rotational

behaviour of the structure; and (2) to assess on the impact of the modelling strategy (2D shell vs 3D solid elements) on the final results.

In the following sections, the rotational behaviour is evaluated through moment-rotation curves. This evaluation involves calculating the rotations under monotonically increasing loads from the vertical displacements obtained in the numerical simulations at three key locations: (1) steel tower bottom section, at the level of the top concrete surface; (2) top surface of the concrete pedestal; (3) bottom surface of the concrete slab (see Figure 4.9). The rotation evaluated as per (1) corresponds to that occurring at the base of the tower and provides the boundary condition for determining the tower displacements. The rotation evaluated as per (2) should be representative of that obtained when a clinometer is placed over the concrete pedestal.

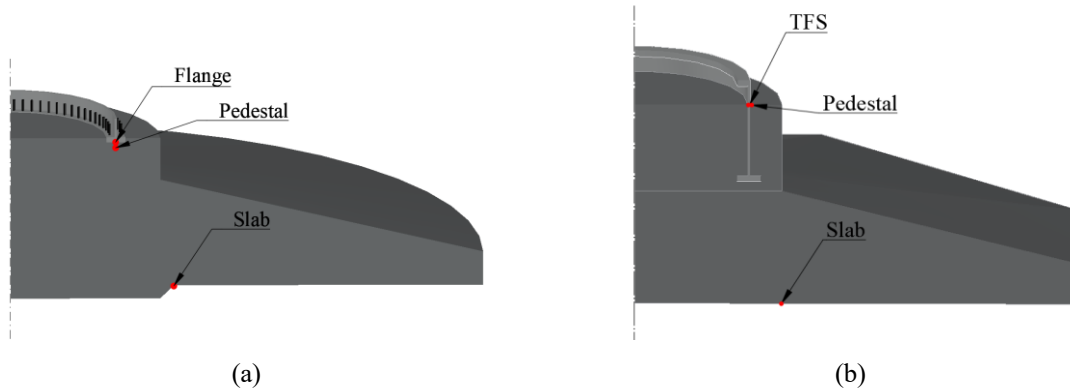


Figure 4.9– Location points for rotational stiffness evaluation: (a) AC foundation; (b) TFS foundation

Figure 4.10 shows the deformed shape of the AC and TFS foundations modelled with 3-D solid elements, highlighting the detachment between concrete and steel components. Specifically, Figure 4.10 (a) illustrates the detachment of the anchor cage flange in the AC foundation, while Figure 4.10 (b) shows the separation occurring at the embedded T-flange interface in the TFS foundation.

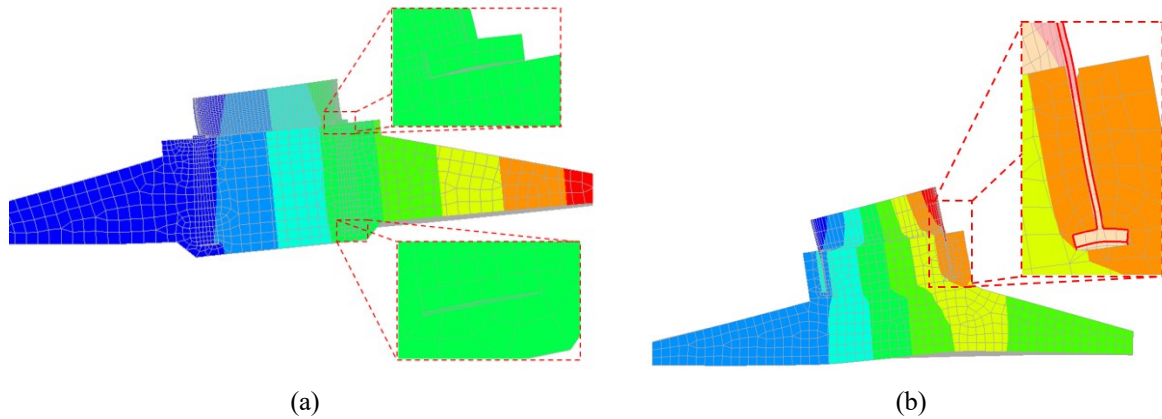


Figure 4.10 – Deformed shapes highlighting the detachment between concrete and steel parts: (a) AC foundation; (b) TFS foundation

4.3.2 RESULTS

Here the serviceability behaviour of the foundations are analysed. Figure 4.11 and Figure 4.12 show the moment-rotation curves for the AC and TFS foundation, respectively. The rotation curves have been evaluated using the vertical displacements at the three key locations described above. The bending moment is normalized by the quasi-permanent bending moment, M_{SLs} . The comparison is made for: (1) analysis with shell elements ("Shell"); (2) analysis with 3-D solid elements ("3D FEM"); (3) analysis with 3-D solid elements with frictionless and cohesionless interface elements between the steel tower and the concrete ("3D FEM+"), as described in Section 4.2.3. Major events, such as detachment from soil, are identified.

The first observation, as expected for the load levels corresponding to serviceability conditions, is that the relationship between the rotation and bending moment is linear regardless the type of analysis and the location points used for determining the rotation.

For the AC foundation, the rotation evaluated at the base of the tower and at the concrete pedestal (Figure 4.11 (a) and (b), respectively) are approximately the same in the "3D FEM" and "3D FEM+" simulations. This indicates that no relative rotation occurs between these two elements. The reason for this is that, even when the interfaces are activated in "3D FEM+", the prestress forces applied to the anchor bolts ensure that decompression is not reached, and the steel flange remains in full contact with the concrete pedestal. The results obtained with the "Shell" model in these same locations provide rotations that are approximately 20% lower. This extra stiffness of the shell element model is due to its inability to reproduce the deformation state in the concrete pedestal, most notably the local vertical deformation component due to the introduction of the tower forces in the footing. When evaluating the rotation at the base of the slab (Figure 4.11 (c)), the results in the "Shell" simulation become closer to those provided by the 3D solid models. At this location, the behaviour is essentially governed by flexure of the tapered slab, which is properly captured by the kinematics of the shell elements. In any case, given its simplicity, it may be assumed the shell element model provides sufficiently accurate estimates for engineering purposes of the rotation stiffness at the base of the tower in normal operating conditions

In contrast, the TFS foundation exhibits considerable variation in the rotation values depending on the location of its evaluation. Significant differences are observed when the rotation is assessed at the steel TFS (Figure 4.12 (a)) and at the concrete pedestal (Figure 4.12 (b)), whereas no variation is noted at the base of the slab (Figure 4.12 (c)). Taking as reference the more detailed "3D FEM+" model, where the interfaces are fully activated, the rotations provided by the "Shell" model at the TFS level (Figure 4.12 (a)) are about 55% lower. This means that, even in the case of an undamaged foundation, the rotational stiffness at the base of the tower in serviceability conditions is overestimated by a factor of about 2.2. This difference is mostly explained by the relative movements between the steel tower elements and the surrounding concrete, which in this case does not rely on passive force transfer mechanisms, and which can only be taken into account by the model "3D-FEM+". When the evaluation is conducted at the concrete pedestal (Figure 4.12 (b)), the simulation with activated interfaces ("3D FEM+") exhibits lower rotation compared to the simulation without interfaces ("3D FEM"). In the latter, the concrete in the pedestal is artificially connected to the steel tower elements and unrealistic vertical deformations are obtained in the concrete finite elements adjacent to the steel tower. This does not occur in the "3D FEM+" model because the interface elements allow relative slip between the concrete pedestal and the steel tower. It is interesting to notice that for the rotations evaluated at the slab, in all models, provide

similar results. Again, at this location, the behaviour is governed by flexure of the tapered slab, which is identically captured by all the models.

The findings indicate that the results of the simulations using a finite element model with shell elements, which is typically used in design practice, can provide a reasonable estimate of the rotational behaviour of the foundations with AC. However, the rotation of the TFS is largely underestimated, which may have consequences on the behaviour of the tower under normal operating conditions, even for the case of foundations without pre-existing damages.

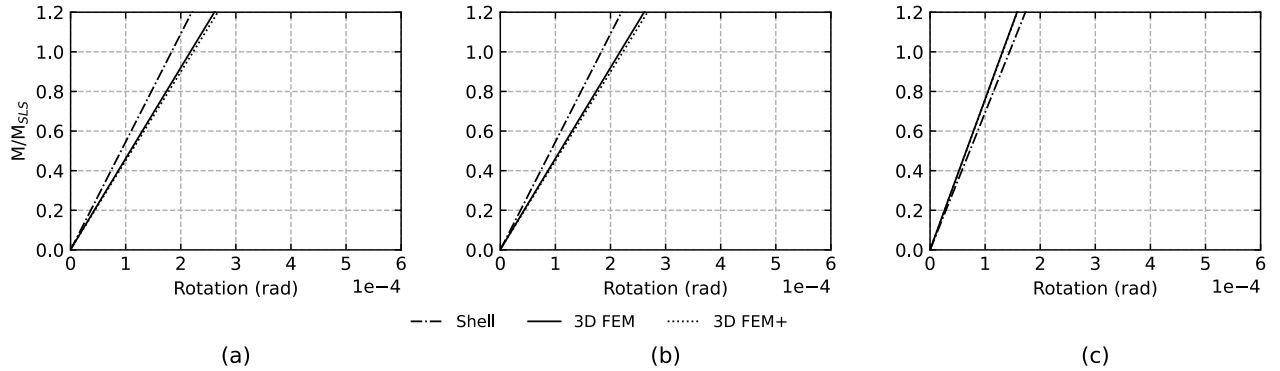


Figure 4.11 – Comparative analysis of the moment-rotation curves in the AC foundation. Evaluation at: (a) tower base flange; (b) concrete pedestal; (c) slab bottom surface

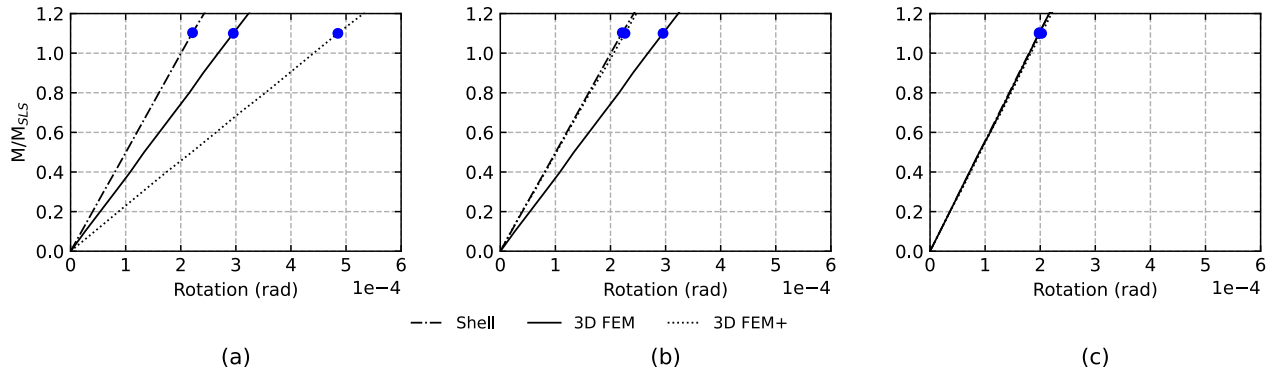


Figure 4.12 – Comparative analysis of the moment-rotation curves in the TFS foundation. Evaluation at: (a) tower base flange; (b) concrete pedestal; (c) slab bottom surface (● detachment from the soil)

4.4 ANALYSIS UP TO FAILURE

In this section the behaviour of the two foundations systems is modelled up to failure, considering nonlinear material behaviour with the properties defined in Section 4.2.2.

4.4.1 CRACK PATTERNS

Figure 4.13 to Figure 4.16 shows the computed crack patterns and vertical stresses below the slab foundation, for the AC and TFS solutions, respectively. Comparison is made for the load step corresponding to a rotation of $\theta = 0.002 \text{ rad}$ and to the last converged step. In the solution with the AC, one important crack propagating vertically develops beneath the most compressed part of tower base.

This is associated to concrete bursting stresses due to the combination of prestress and compressive forces resulting from the bending moment at the tower base. With increasing load, extensive flexural cracking at the tapered slab becomes more evident. The crack pattern in the footing with embedded TFS shows one important horizontal crack at the transition from the pedestal to the slab, promoting the separation of these two elements and further decreasing the rotational stiffness. The region where the TFS is under tension is extensively cracked, with the model being capable of reproducing diagonal cracks, indicative of impending pull-out failure.

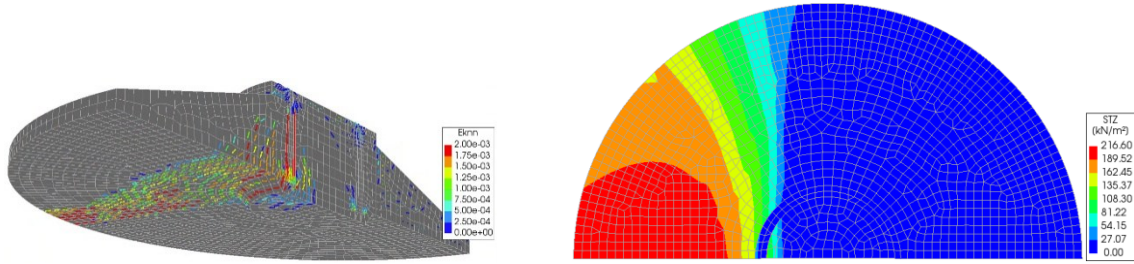


Figure 4.13 – AC foundation results for a tower bottom section rotation of $\theta = 2 \times 10^{-3}$ rad ($M/M_{SLS}=2.5$): crack strain (Eknn) pattern; vertical stress (STZ) below slab foundation

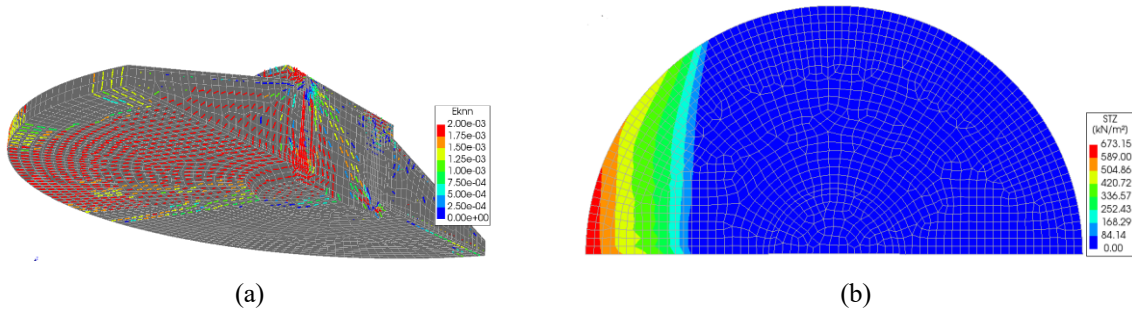


Figure 4.14 – AC foundation results for the last converged solution [$M/M_{SLS}=3.4$]: (a) crack strain (Eknn) pattern; vertical stress (STZ) below slab foundation

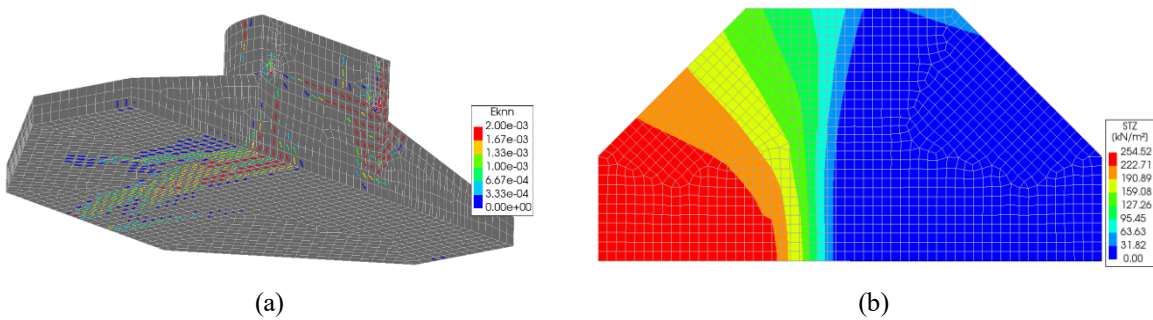


Figure 4.15 – TFS foundation results for a tower bottom section rotation of $\theta = 2 \times 10^{-3}$ rad ($M/M_{SLS}=2.0$): (a) crack strain (Eknn) pattern; vertical stress (STZ) below slab foundation

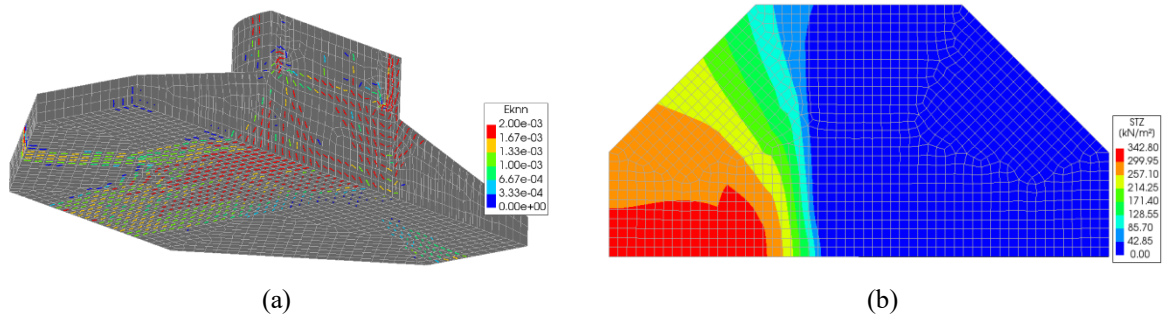


Figure 4.16 – TFS foundation results for the last converged solution [$M/M_{SLS}=2.17$]: (a) crack strain (Eknn) pattern; vertical stress (STZ) below slab foundation

4.4.2 MOMENT-ROTATION CURVES

The influence of the nonlinearities introduced by the material properties on the structural response of the foundation systems (AC foundation and TFS foundation) is evaluated through the analysis of the moment-rotation curves. This section presents a comparative analysis of different moment-rotation curves obtained from different types of numerical analysis, which are:

- Linear: analysis considering linear material behaviour with nonlinear interface soil-foundation;
- Interfaces: extension of the previous analysis with the inclusion of nonlinear interfaces between tower and foundation;
- NL: fully nonlinear analysis incorporating the nonlinear material behaviour of concrete and reinforcement.

Figure 4.17 compares of the moment-rotation curves evaluated at the base of the steel tower (T-flange) for the AC foundation under the three different analysis conditions (Linear, Interfaces and NL). As already noticed in Section 4.3 for the serviceability load levels, the results indicate that the moment-rotation curves for the “Linear” and “Interfaces” analyses are nearly coincident also for load levels well above M_{SLS} . Furthermore, the introduction of the material nonlinearities only leads to a noticeable rotational stiffness reduction for load levels above $1.9M_{SLS}$, which coincides with the formation of significant cracks in the pedestal and slab. However, the moment-rotation curve remains smooth and converged results are obtained for load levels reaching $3.4M_{SLS}$, at which approximately 17% of the radial reinforcement has yielded. These observations confirm the adequacy of the prestressed solution for keeping the behaviour of the structure less affected by crack formation and propagation.

Figure 4.18 shows the moment-rotation curves evaluated at the steel shell of the TFS foundation under the same analysis conditions. Unlike the AC foundation, and as discussed already in Section 4.3, the inclusion of the interfaces between the TFS and the concrete results in an increase of the rotation of the tower base. The inclusion of the material nonlinearities, most notably concrete cracking, is responsible for a notable stiffness decrease when the loads reach approximately $1.1M_{SLS}$. This behaviour is attributed to the formation of a significant horizontal crack at the base of the pedestal. This crack develops due to the uplift forces that are generated by the bending moment acting at the base of the tower and activates the vertical rebars in the pedestal near the TFS. When the moment reaches $1.4M_{SLS}$, further reduction in the rotational stiffness occurs due to the propagation of new cracks in the tapered slab. At $1.9M_{SLS}$, additional crack propagation in the tapered slab leads to further reduction in the rotational stiffness.

Beyond this load level, the rotational stiffness decreases continuously, and the simulation ends at approximately $2.15M_{SLs}$ when the vertical rebars (suspension reinforcement) in the pedestal reach their yielding stress, causing numerical instabilities.

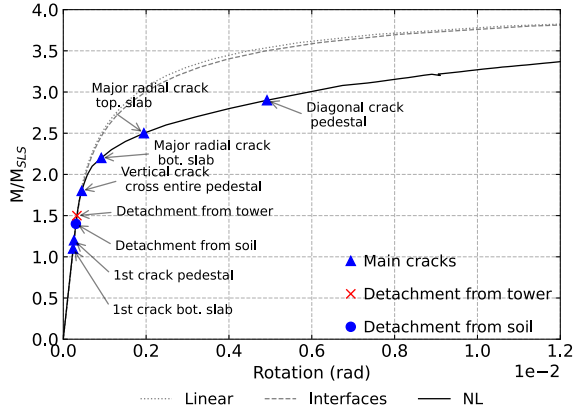


Figure 4.17 – AC foundation: Moment-rotation curves evaluated at the T-flange. Comparison between Linear, Interface and NL analyses

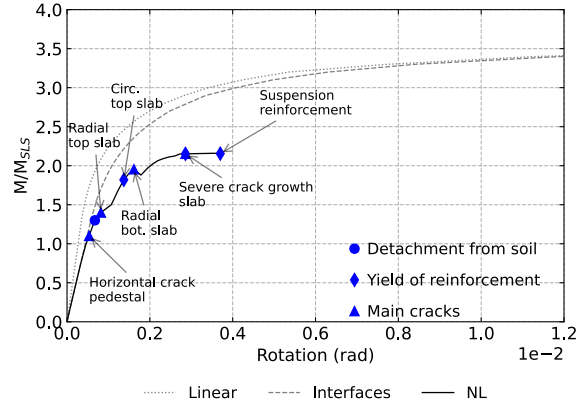


Figure 4.18 – TFS foundation: Moment-rotation curves evaluated at the steel shell. Comparison between Linear, Interface and NL analyses

4.5 COMPARISON WITH EXPERIMENTAL EVIDENCE

This section presents a comparative analysis between the results obtained from the numerical simulations and the experimental data outlined in Chapter 3. The purpose of this comparison is to assess the accuracy and reliability of the numerical models in replicating the real behaviour of the foundations under analysis.

It is important to highlight that this comparison focuses exclusively on the AC foundation, as the TFS foundation is currently considered to be in a damaged condition. Comparative analyses involving the TFS foundation will be presented in Chapter 5.

The experimental observations correspond to load levels occurring under normal operating conditions. As discussed in Chapter 3, during the two-year monitoring period of the WTG in the Serra del Rei windfarm the acting moments only marginally exceeded the quasi-permanent value.

4.5.1 MOMENT-ROTATION CURVE

The real structural behaviour of the wind turbine-foundation system involves complexities that are not fully captured by the thrust-based theoretical model, particularly the influence of tower deflection and foundation stiffness. A key design parameter is therefore the rotational stiffness at the tower base, which is commonly defined through analytical expressions that neglect nonlinear mechanisms at foundation level.

The monitoring system provides an opportunity to evaluate this rotational stiffness directly and to compare it with the predictions of the numerical model. Based on the inclinometer and strain gauge data (10-minute average values recorded over approximately 1300 days), the rotation of the foundation, measured at the base of the tower, can be plotted as a function of the bending moment at the tower

bottom section. This relationship is presented in Figure 4.19, where the experimental data are compared with the numerical results, considering the normalised bending moment M/M_{SLS} .

Figure 4.19 (a) shows the moment-rotation curve obtained from the numerical model for a ground Young modulus of $E = 1000$ MPa, which corresponds to the value adopted by the foundation designer. In this case, the response is plotted up to $M/M_{SLS} = 2.5$ in order to represent mostly the entire range of bending moments covered by the numerical simulation. The experimental points are superimposed and follow closely the numerical curve within the range of recorded bending moments. Since the wind turbine control system is designed to limit the bending moments at the tower base so that they rarely exceed the SLS load combination under normal operating conditions, the monitoring data remain largely within the initial linear branch of the moment-rotation curve. In this range, very good agreement is observed between measured rotations and the numerical results for a ground with a E-modulus of $E = 1000$ MPa.

To investigate the influence of the stiffness of the soil on the footing rotation, two additional numerical scenarios were analysed. The reference case adopted the E-modulus of the soil equal to 1000 MPa, corresponding to the value used by the foundation designer (Figure 4.19 (a)). The two further simulations were performed with E-modulus values of 200 MPa and 5000 MPa. The comparison, presented in Figure 4.19 (b), provides a zoomed view of the moment-rotation response, restricted to $M/M_{SLS} = 1.5$, in order to enable a more detailed comparison between the curves. This analysis highlights the sensitivity of the rotation to the assumed ground stiffness and confirms that the numerical response obtained with $E = 1000$ MPa provides the best agreement with the experimental moment-rotation curve in the operational range covered by the monitoring data.

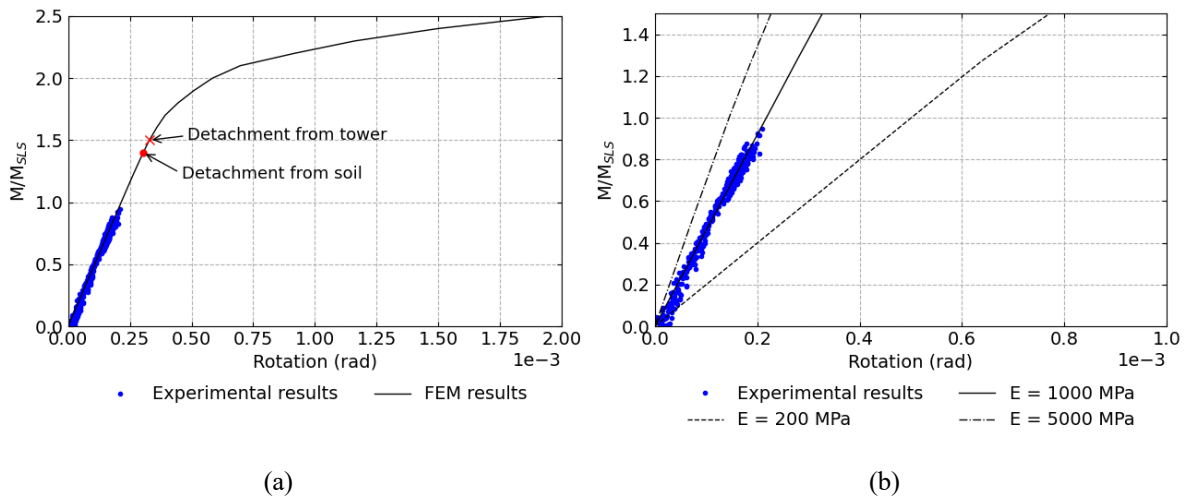


Figure 4.19 – AC foundation rotation with increasing bending moment. Comparison between experimental and numerical results. Numerical model results are presented for: (a) ground E-modulus $E = 1000$ MPa; (b) ground E-modulus values $E = 200$ MPa, $E = 1000$ MPa and $E = 5000$ MPa;

4.5.2 ANCHOR BOLT FORCES

In the wind industry, the process of designing ring flange connections typically relies on the methodology introduced by Petersen [44]. This well-established calculation model centres its analysis on the performance of each discrete segment within the flange, emphasizing the segment containing the bolt subjected to the highest tensile force (Figure 4.20).

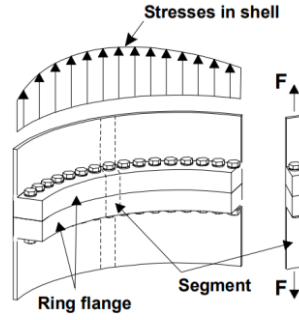


Figure 4.20 – Simplified model of the ring flange with the preloaded bolts (adapted from [17])

Despite the model was originally developed for tower segment connections, its application was extended to the design of anchor bolts. In this case, the behaviour of the connection is assessed by considering the relative stiffness of the steel and the concrete, which governs how the externally applied stress resultants are shared between the anchor bolts and the compressed concrete. As the applied bending moment increases, the bolt force evolves in two distinct stages, separated by the onset of joint separation (gapping). The design is performed considering one pair of anchors (one inner and one outer bolt). The tower reaction acting on that segment, P_{td} , is treated as the external action applied to the bolt pair and is obtained by combining the design normal force, N_d , and the design bending moment, M_d .

As the bending moment increases (for example under wind loading), the force on the tensile side, P_{td} , increases. The maximum force per pair of anchors is computed as:

$$P_{td} = -\frac{N_d}{n} + \frac{4 \cdot M_d}{n \cdot D} \quad (4.14)$$

where n is total number of pair bolts and D is the mean diameter at the tower base.

To determine how much of P_{td} is transferred into bolt tension and how much is accommodated by relief of concrete compression, the axial spring stiffness of both components, steel stiffness (C_s) and concrete stiffness (C_c), is estimated, in order to determine the stiffness ratio that defines the fraction p (expression (4.15)) of the external force that increases the bolt tension while the joint remains fully in contact [45].

$$p = \frac{1}{1 + C_c/C_s} \quad (4.15)$$

The relationship between bolt force, P_a , and the external action, P_{td} , is piecewise, with the transition governed by the separation threshold Z_I .

For $P_{td} < Z_I$, the tower flange remains in full contact with the grout or concrete, and the system responds around the applied preload P_p . In this regime, the bolt force increases slowly because only the fraction p of the external action is transferred into additional bolt tension:

$$P_a = P_p + p \cdot \frac{P_{td}}{2} \quad (4.16)$$

The factor $P_{td}/2$ is used because P_{td} is defined for a pair of anchors, whereas P_a refers to the force in an individual bolt, assuming symmetric force sharing within the pair.

The threshold Z_I corresponds to the point at which the initial compressive stress in the concrete induced by preload is fully relieved by the tensile external action. At this point the joint begins to open and gapping initiates. The threshold depends on the preload level:

$$Z_I = 2 \cdot \frac{P_p}{1 - p} \quad (4.17)$$

Once P_{td} exceeds Z_I , contact is lost over part of the interface and there is no longer a compressed concrete spring contributing to load sharing. Consequently, any additional external action is carried directly by the bolt, leading to a much steeper increase in bolt force:

$$P_a = \frac{P_{td}}{2} \quad (4.18)$$

Numerical models based on the FEM have shown that the analytical methods currently used to design flanges and bolts are conservative. One such study is presented in [46].

In the context of this work, the forces in the anchor bolts were determined using both analytical and numerical methods. These results were then compared with experimental data obtained through load cells. As illustrated in Figure 4.21, the experimental data is represented by a cloud of points corresponding to the forces in bolts P1 and P4 recorded during the experimental campaign period. The analysis indicates that the curve obtained through the numerical model follows closely the experimental data. The analytical formulation, which is used for designing such connections, tends to overestimate force variations in the bolts, resulting in the need of higher grout and concrete grades, as well as oversized anchor bolts and flanges.

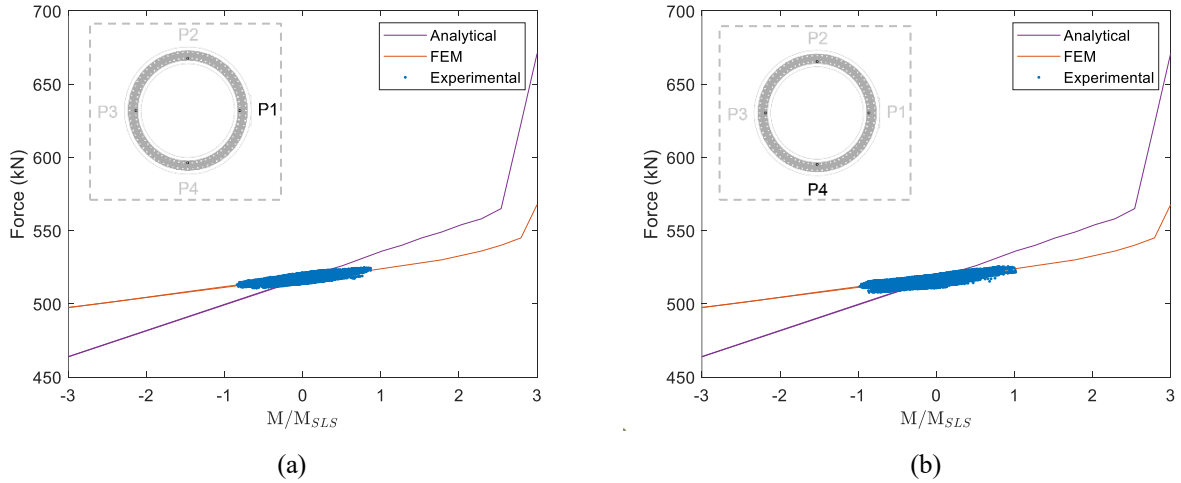


Figure 4.21 – Forces in the anchor bolts (AC foundation). Comparison between experimental, numerical and analytical results: (a) force at P1; (b) force at P4

4.6 STRESS REDISTRIBUTION

This section presents an evaluation of stress redistribution to assess its potential impact on foundation design. As established in the introduction, conventional design relies on linear elastic analysis, which

does not account for inelastic phenomena such as concrete cracking, loss of stiffness, and the redistribution of internal forces. This assessment is therefore conducted by comparing the stress resultants derived from the linear elastic analysis with those from the nonlinear analysis. The comparison focuses on the primary stress resultants: bending moments and axial forces. These are presented for both the circumferential and radial directions under SLS and ULS load combinations. This analysis is presented in two parts. First, a qualitative comparison of the stress resultant contour maps is provided to visualize the mechanism of redistribution. This is done by comparing the linear with the nonlinear stress resultant colourmaps. Second, the stresses in the reinforcement evaluated along selected circumferential and radial paths are compared between linear and nonlinear analyses at both SLS and ULS.

4.6.1 COMPARISON OF STRESS RESULTANTS

Figure 4.22 to Figure 4.29 present the distributions of bending moments and axial forces in the slab for the SLS and ULS combinations, obtained from both the linear and nonlinear analyses. In each case, subfigure (a) corresponds to the linear elastic model, whereas subfigure (b) shows the nonlinear model. The aim is to highlight how material nonlinearities modify the internal force patterns relative to the linear elastic models. The local axes x - and y - represent the circumferential and radial directions, respectively.

The linear models produce smooth and regular contour lines in all SLS and ULS plots, as exemplified in Figure 4.22 (a). This behaviour is representative of a continuous, homogeneous, uncracked elastic plate with constant (stress independent) stiffness. The internal forces follow the geometric symmetry of the foundation and the applied loading, and the contour maps exhibit gradual spatial variation, without abrupt gradients or localisation.

In contrast, the colourmaps from the nonlinear analysis show distinct, “banded” contours, more pronounced in the radial direction, and already visible at the SLS. This phenomenon is not primarily due to physical material nonlinearity (such as micro-cracking) at this low load stage. Instead, it is a direct consequence of the post-processing methodology used. These stress resultants are determined using “composed elements”, a post-processing tool in the FEA software (DIANA FEA [37]) that calculates stress resultants by integrating the stresses over the thickness. This integration explicitly includes the stresses from the concrete continuum elements and the stresses from the discrete embedded reinforcement elements. Therefore, the “banded” contours observed in the nonlinear SLS plots (e.g., Figure 4.23 (b)) are the numerical visualization of this composite action. The “bands” in the contour plot effectively map the reinforcement layout, indicating the localized, higher stiffness and concentrated force contribution from the steel reinforcement, which is absent in the homogeneous linear model. Disregarding this numerical visualization of the reinforcement, the overall stress distribution and the magnitudes of the stress resultants at SLS remain comparable to the linear elastic model. This high degree of correlation indicates that, at the serviceability limit state, the physical behaviour is still predominantly elastic, and no stress redistribution has yet occurred.

At ULS (Figure 4.26), the circumferential bending moments in the NL model are significantly lower than those obtained from the linear analysis. This reduction is consistent with a loss of stiffness in the circumferential direction after the development of radial cracking (that is, cracks oriented radially, driven by circumferential tensile stresses). Once this cracking pattern forms, the effective

circumferential continuity of the foundation slab degrades, and the continuous ring mechanism is partially replaced by a response closer to that of radially oriented sectors with reduced mutual coupling. From a physical standpoint, the slab can no longer mobilise M_{xx} with the same efficiency, and part of the equilibrium is transferred to directions and modes that remain comparatively stiffer. This redistribution leads to a generalised reduction of M_{xx} in the post-cracking regime. In contrast, the radial bending moments (Figure 4.27) in the NL model remain broadly similar to those in the linear model. This outcome indicates that, even after the degradation of circumferential stiffness, the moments in the radial direction do not change significantly.

The membrane forces (Figure 4.28 and Figure 4.29) exhibit pronounced differences between the linear and NL models. In slab and shell representations, N_{xx} and N_{yy} correspond to membrane resultants (forces per unit length), which may become increasingly significant when cracking and stiffness degradation modify the balance between bending and membrane action. In the NL model, cracking and the associated reduction of stiffness favour the activation of arching and membrane mechanisms, including compressive membrane trajectories and the redistribution of tensile forces towards regions that remain less damaged. These effects are reflected in both the magnitude and the spatial distribution of N_{xx} and N_{yy} . Furthermore, because composed elements integrate the contribution of discrete reinforcement explicitly, the NL maps of the membrane forces may display banding associated with zones where reinforcement is actively participating in equilibrium, whereas the linear model tends to distribute the membrane resultants more uniformly across the domain.

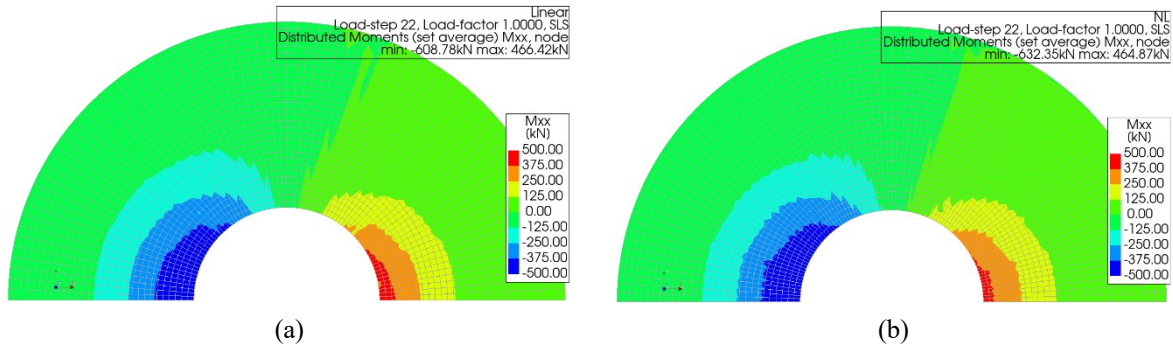


Figure 4.22 – Bending moment in the circumferential direction (M_{xx}) for SLS load combination: (a) Linear analysis; (b) Nonlinear analysis

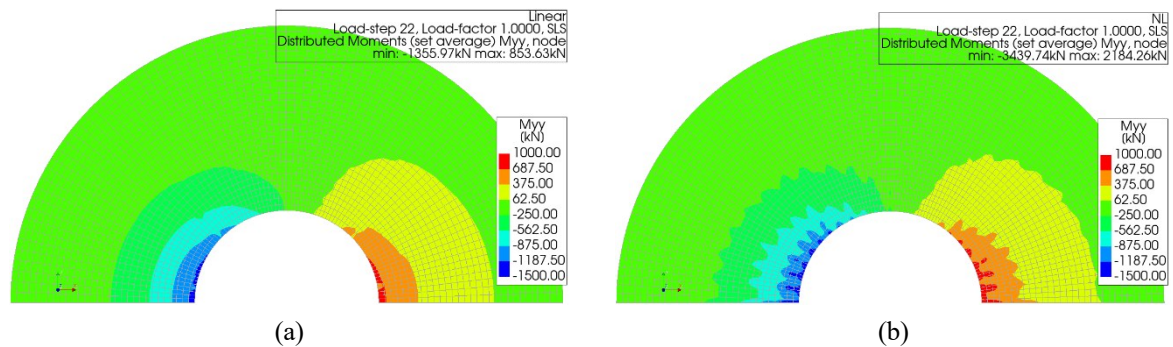


Figure 4.23 – Bending moment in the radial direction (M_{yy}) for SLS load combination: (a) Linear analysis; (b) Nonlinear analysis

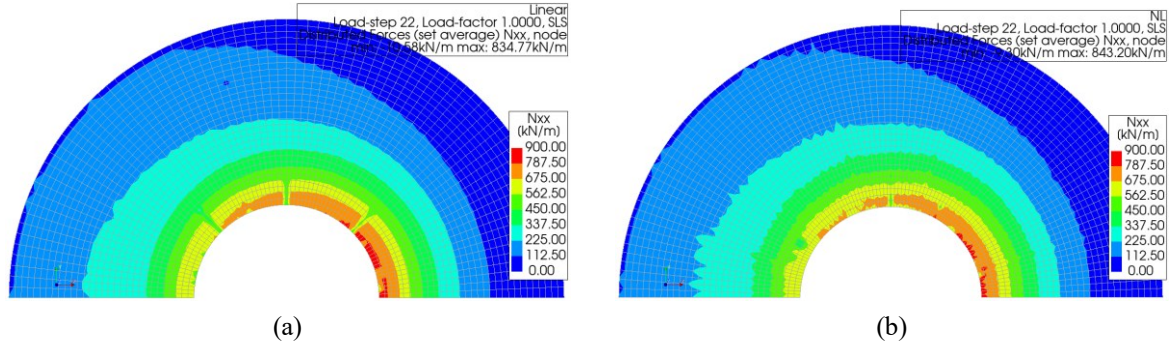


Figure 4.24 – Circumferential force (N_{xx}) for SLS load combination: (a) Linear analysis; (b) Nonlinear analysis

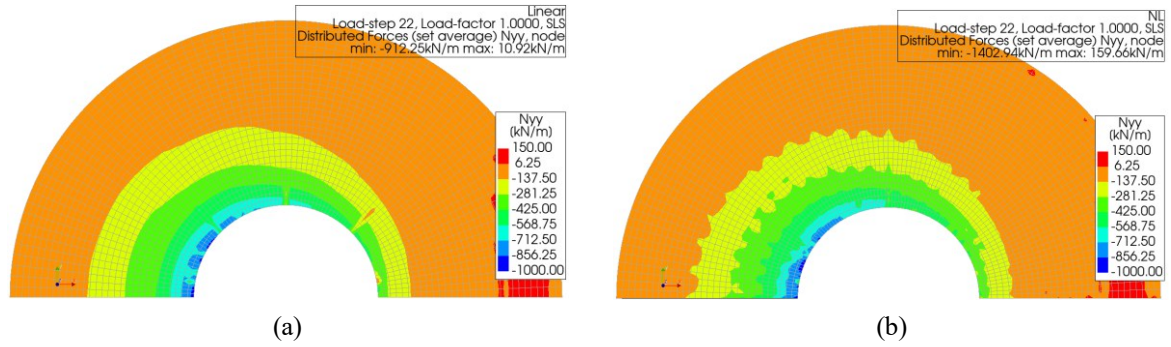


Figure 4.25 – Radial force (N_{yy}) for SLS load combination: (a) Linear analysis; (b) Nonlinear analysis

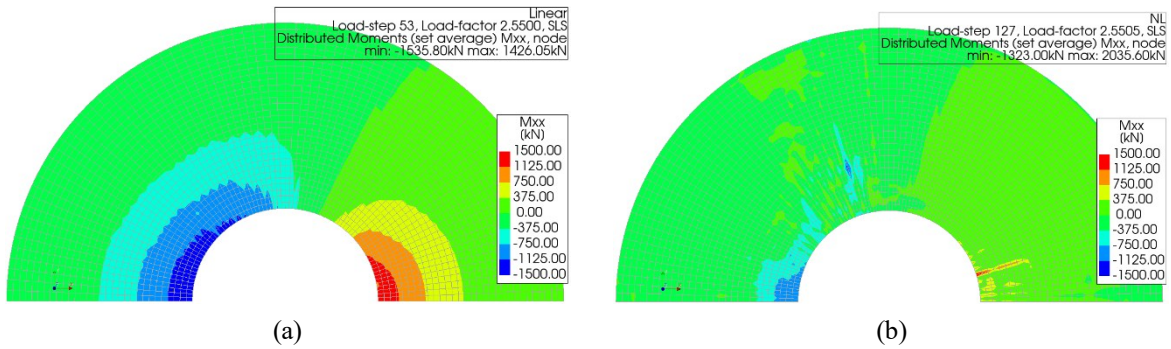


Figure 4.26 – Bending moment in the circumferential direction (M_{xx}) for ULS load combination: (a) Linear analysis; (b) Nonlinear analysis

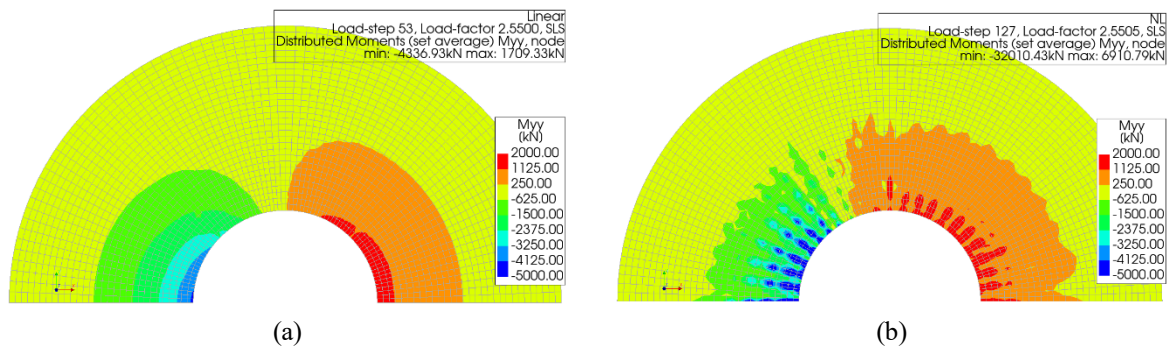
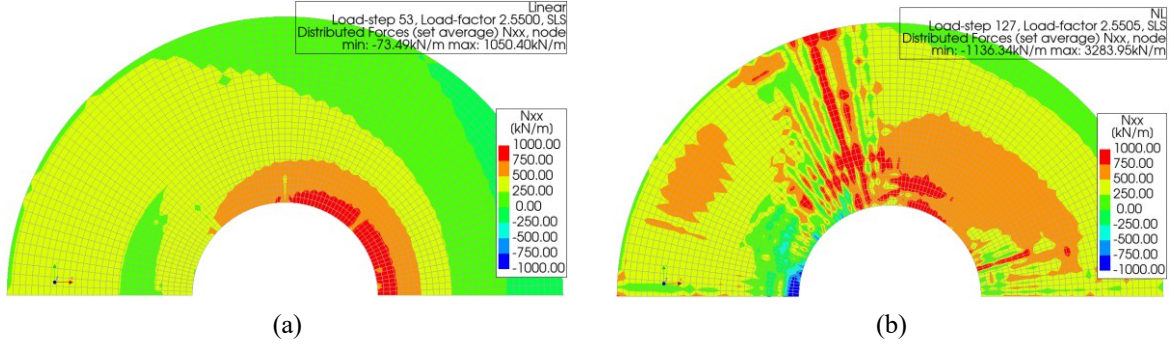
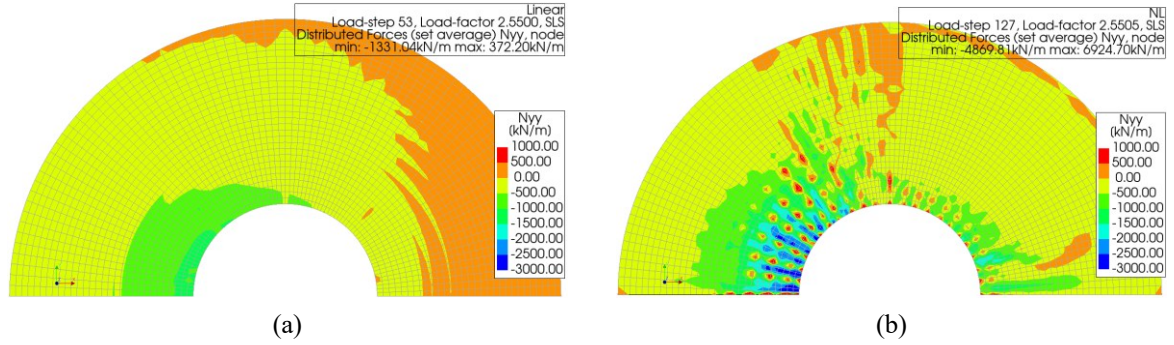


Figure 4.27 – Bending moment in the radial direction (M_{yy}) for ULS load combination: (a) Linear analysis; (b) Nonlinear analysis

Figure 4.28 – Circumferential force (N_{xx}) for ULS load combination: (a) Linear analysis; (b) Nonlinear analysisFigure 4.29 – Radial force (N_{yy}) for ULS load combination: (a) Linear analysis; (b) Nonlinear analysis

4.6.2 REINFORCEMENT STRESSES

The previous subsection established that material nonlinearities significantly modify the distribution of the stress resultants, particularly at ULS. The present subsection investigates how these changes translate into differences in the required reinforcement and hence into potential design implications.

For this purpose, the approach utilized for the ULS design of planar members, such as shell and slab elements, adopted in prEN 1992-1-1:2023 Annex G [42], is now applied in the present study to access the stress in the reinforcement.

Current practice in the design of concrete foundations involves determining the eight stress resultants (membrane forces n_x, n_y, n_{xy} ; bending moments and torsional moments (m_x, m_y, m_{xy}); and transverse shear forces (v_x, v_y) from the FE analysis. As detailed in Annex G.4 [42], the reinforcement required to resist the combination of in-plane forces, bending moments, and torsional moments in shell elements is determined by adopting the sandwich model. This model transforms the sectional forces into a set of statically equivalent in-plane stresses acting in the top and bottom layers. The design reinforcement in each of the two layers may then be calculated by the method outlined for in-plane stress conditions.

The required reinforcement force in the bottom and top layers, $F_{rebar}^{bottom/top}$ are determined using the following equations:

$$F_{rebar,x}^{bottom/top} \geq \pm \frac{m_x}{d} + \frac{n_x}{2} + \left| \frac{m_{xy}}{d} + \frac{n_{xy}}{2} \right| \quad (4.19)$$

$$F_{rebar,y}^{bottom/top} \geq \pm \frac{m_y}{d} + \frac{n_y}{2} + \left| \frac{m_{xy}}{d} + \frac{n_{xy}}{2} \right| \quad (4.20)$$

where d is the lever arm between top and bottom layers.

In the case under investigation, the required applicability conditions, $m_x + |m_{xy}| \geq 0$ and $m_y + |m_{xy}| \geq 0$, requested in [42], are satisfied, thereby allowing the use of the equations (4.19) and (4.20).

The stresses in the reinforcement are calculated in the linear analysis with the use of the equations (4.19) and (4.20). For the nonlinear analysis, the stresses are obtained directly from the embedded reinforcement elements in the numerical model. To enable a direct comparison, both datasets are evaluated along three representative paths defined in Figure 4.30: a circumferential path at $r = 3$ m (C-C') and two radial alignments at 100° (B-B') and 180° (A-A').

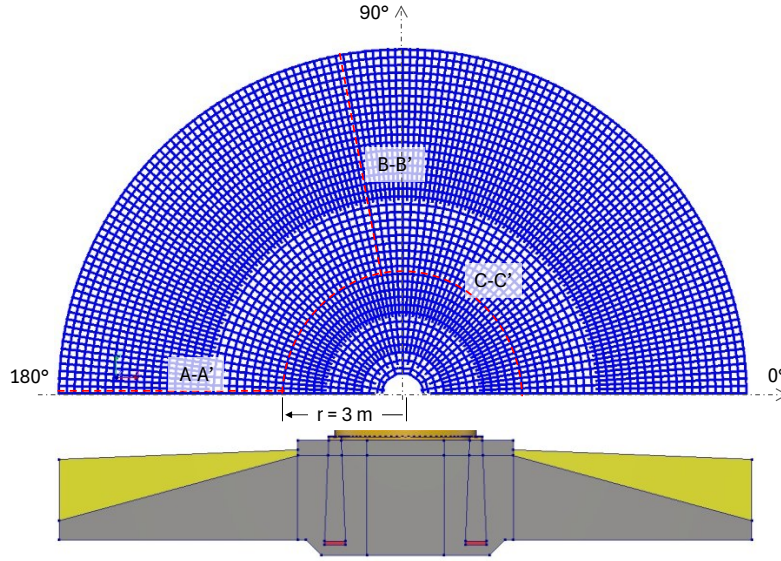


Figure 4.30 – Bottom reinforcement mesh of the foundation, showing the circumferential and radial reinforcement (blue lines). The dashed red lines indicate the paths along which the stresses in the reinforcement are evaluated for comparison between linear and nonlinear analyses: the circumferential path at $r = 3$ m (C-C'), and the radial paths at 100° (B-B') and 180° (A-A'). The bottom drawing schematically represents the foundation cross-section to support interpretation.

4.6.2.1 BOTTOM RADIAL DIRECTION

Figure 4.31 compares the stresses in the bottom radial reinforcement evaluated along the circumferential path at $r = 3$ m. The stresses corresponding to the linear analysis (solid lines) are derived from the design stress resultants using the sandwich model, whereas the stresses corresponding to the nonlinear analyses (dashed lines) are obtained directly from the embedded reinforcement elements. Two reinforcement configurations are reported for the nonlinear analyses: the reference reinforcement layout (100%) and a reduced reinforcement area (50%) to evaluate the redistribution effect in the structure. In complement, Figure 4.32, presents the corresponding stress colourmap for the reinforcement derived from the nonlinear analysis under the ULS load combination.

The numerical simulations indicate that the radial rebars are mobilised predominantly between 90° to 180° , which coincides with the sector where the bottom side of the slab is in tension under the applied external bending moment. At ULS, the linear analysis predicts a rapid increase in stress, with a peak of 410 MPa at 160° , yet remaining below the design yield strength (f_{yd}) of the rebars.

The quantitative comparison of the radial design reinforcement forces reveals a significant discrepancy between the total reinforcement force obtained from the linear and nonlinear analyses considering 100% reinforcement area, with a ratio of about 0.40.

When comparing between 100% and 50% reinforcement areas, indicates that even when the available reinforcement area is reduced by half, the structure possesses the capacity to redistribute internal forces without exceeding the design yield strength. Reducing the reinforcement area changes the stiffness of the system and therefore modifies the internal force path by which equilibrium is achieved.

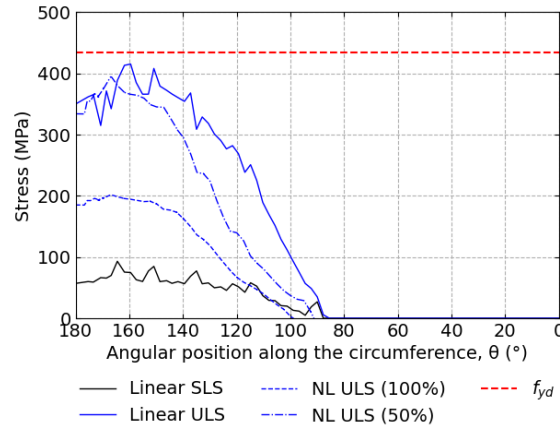


Figure 4.31 – Stresses in the bottom radial reinforcement evaluated along the circumferential path at $r = 3$ m. Comparison between linear analysis (solid lines) and nonlinear analysis (dashed lines) for the SLS and ULS load combinations. The nonlinear results are shown for the reference reinforcement area (100%) and a reduced reinforcement area (50%)

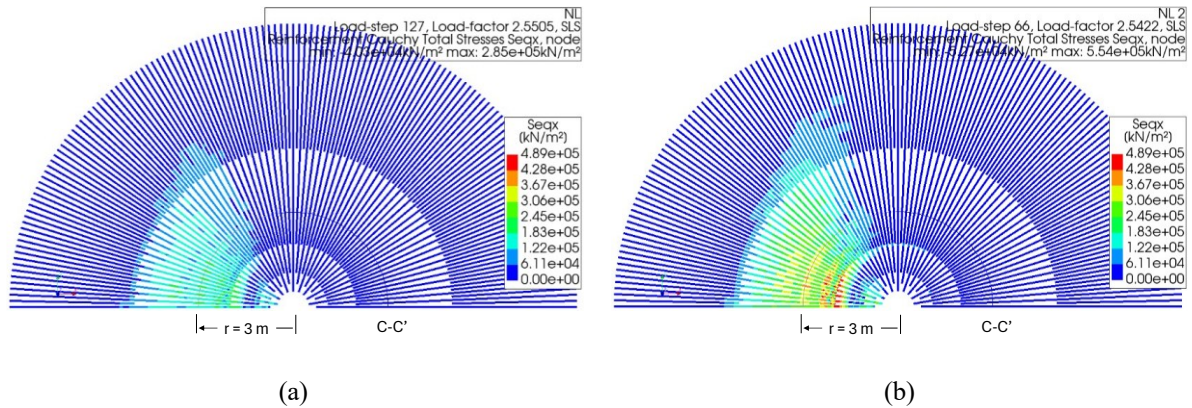


Figure 4.32 – Colourmaps of the stresses in the bottom radial reinforcement for the ULS load combination: (a) reference reinforcement area (100%); (b) reduced reinforcement area (50%)

4.6.2.2 BOTTOM CIRCUMFERENTIAL DIRECTION

Figure 4.33 compares the stresses in the bottom circumferential reinforcement along two radial alignments: $\theta = 100^\circ$ (B-B') in Figure 4.33 (a) and $\theta = 180^\circ$ (A-A') in Figure 4.33 (b), both for SLS and ULS, comparing the linear and nonlinear analyses results. The linear analysis shows a distinct peak at approximately $r = -5.2$ m, followed by a secondary peak towards the outer region. This stepwise variation is consistent with the transition in bar diameters from $\phi 20$ to $\phi 25$ and finally $\phi 32$ near the foundation core. Under ULS conditions, the nonlinear predictions for both the 100% and 50%

reinforcement layouts remain below the stresses obtained from the linear analysis along almost the entire path, indicating reduced peak demand as a consequence of cracking induced redistribution. The only exception occurs for the 100% nonlinear case at the outer edge of the slab ($r > 7.2 \text{ m}$), where the circumferential stress locally exceeds the linear estimate, suggesting redistribution of internal forces to the outer edge of the foundation.

Figure 4.34 presents the colourmap of the bottom circumferential reinforcement stresses for the nonlinear analysis under the ULS load combination, showing (a) the 100% reinforcement layout and (b) the 50% reinforcement layout. In both cases, the stresses remain below the design yield strength.

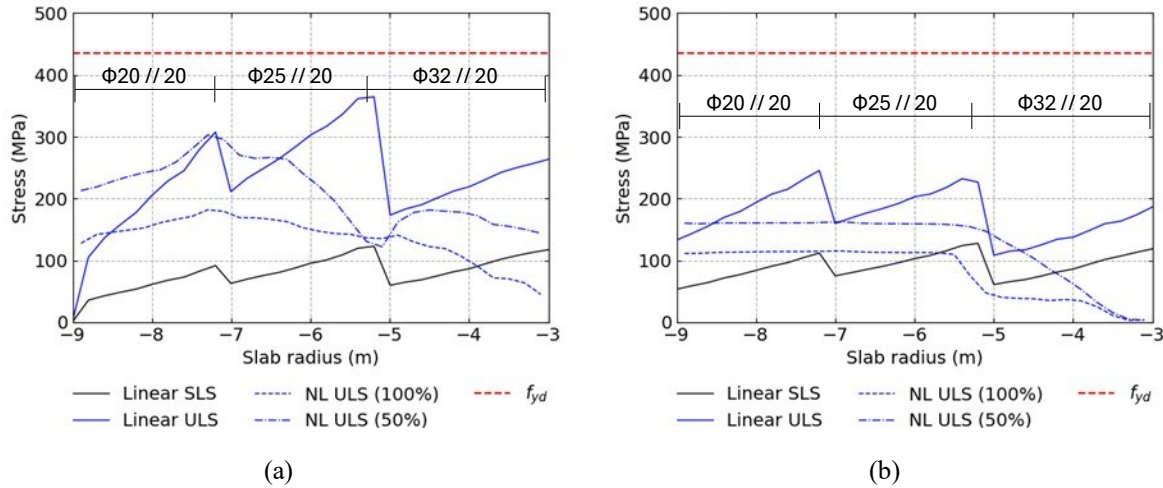


Figure 4.33 – Stresses in the bottom circumferential reinforcement. Comparison between linear analysis (solid lines) and nonlinear analysis (dashed lines) for the SLS and ULS load combinations. The nonlinear results are shown for: (a) the radial alignment at $\theta = 100^\circ$ (B-B') and; (b) the radial alignment at $\theta = 180^\circ$ (A-A'), considering the reference reinforcement area (100%) and a reduced reinforcement area (50%).

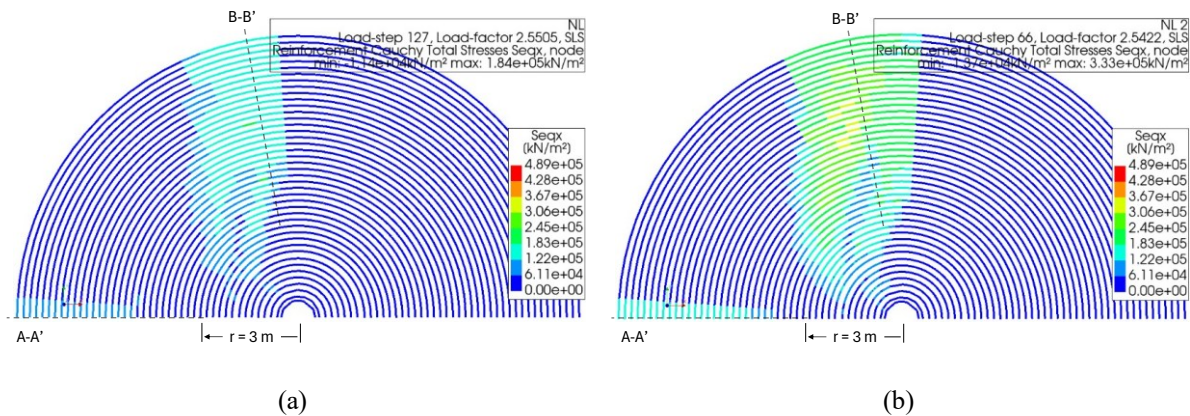


Figure 4.34 – Colourmaps of the stresses in the bottom circumferential reinforcement for the ULS load combination: (a) reference reinforcement area (100%); (b) reduced reinforcement area (50%)

Figure 4.35 clarifies the mechanism responsible for the angular bias. The vertical displacement field in Figure 4.35 (a) shows clearly that more than half of the foundation is detached from the soil beneath, with compression on one side of the foundation and uplift on the opposite side. This contact loss amplifies the tensile stresses in the circumferential reinforcement near the transition between contact

and gap as proved by the crack pattern in Figure 4.35 (b), and in the radial reinforcement bars close to the 180° orientation.

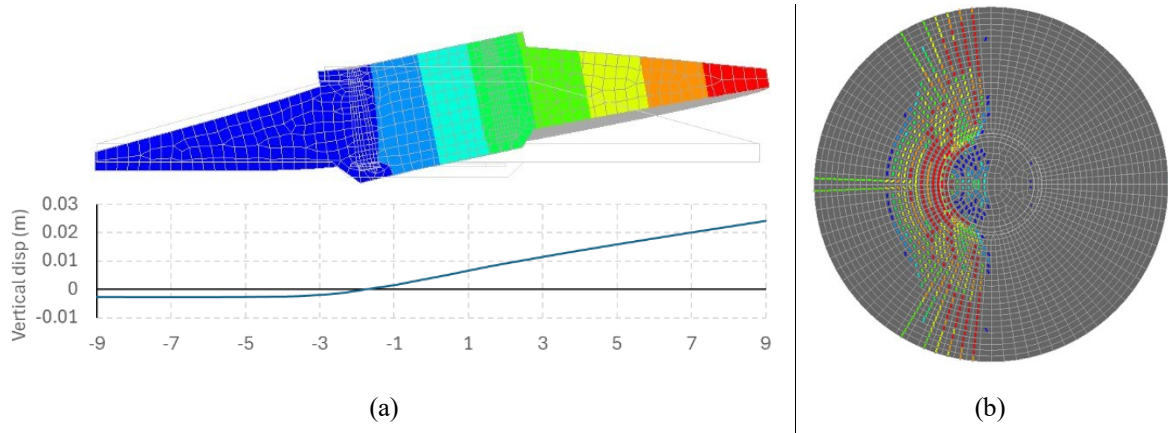


Figure 4.35 – (a) Vertical deformation of the foundation for ULS load combination; (b) crack pattern (bottom view) with red arrows illustrating the circumferential traction

The reduction in circumferential reinforcement demand is driven by mechanisms distinct from those acting in the radial direction. The degradation of the circumferential stiffness due to the formation of radial cracks, visible in Figure 4.35 (b), induces a severe loss of stiffness in the circumferential direction. As a consequence, the structural behaviour progressively shifts from that of a continuous ring slab towards a mechanism closer to a set of independent radial cantilevers thereby relieving the circumferential reinforcement from the requirement to ensure the elastic continuity (ring effect) imposed by the linear model.

4.7 CONCLUSIONS

This chapter investigated the structural behaviour of onshore wind turbine foundations through advanced nonlinear numerical modelling, addressing limitations of conventional design methodologies. Standard shell-element approaches were compared with 3D solid finite element simulations, and the numerical predictions were confronted with experimental evidence. The main conclusions concern modelling strategy selection, structural performance, and implications for safety assessment and design optimisation.

The analyses indicated that the AC foundation exhibits a robust structural response, with limited sensitivity to concrete cracking under serviceability conditions. Prestressing of the anchor bolts was shown to be essential for maintaining the integrity of the steel-concrete interface, thereby preventing relative rotations even at load levels well above service demands. For global response quantities, standard shell-element models were found to provide sufficient accuracy for engineering practice. However, for local actions, particularly the force evolution in the anchor bolts, the 3D nonlinear solid model provided a more realistic representation than simplified analytical approaches, highlighting clear potential for reducing over-conservatism associated with plane-section assumptions.

Within the same 3D nonlinear modelling structure, the stress-field assessment further demonstrated that material nonlinearities promote substantial redistribution of internal forces, a mechanism neglected in linear elastic design verifications. The nonlinear simulations showed that the AC foundation can

mobilise alternative equilibrium paths, enabling the structure to accommodate the applied loading even with a 50% reduction in reinforcement of the slab. These results suggest that design practices derived solely from linear elastic stress fields may be overly conservative, and that nonlinear analysis can support more efficient reinforcement layouts, with benefits in material consumption and associated carbon footprint.

In contrast, the TFS foundation exhibited a more complex nonlinear response that is not adequately captured by standard design methodologies. This typology was found to be particularly susceptible to relative movements between the embedded steel flange and the surrounding concrete. Force transfer from the TFS to the concrete relies on the activation of passive reinforcement, which requires crack initiation and propagation, leading to a marked reduction in rotational stiffness at load levels close to the serviceability limit. The shell-element approach was shown to overestimate the rotational stiffness by a factor of approximately 2.2, confirming that simplified linear modelling is inadequate for reliable assessment of this type of foundations.

5 STRUCTURAL BEHAVIOUR OF DAMAGED FOUNDATIONS

5.1 INTRODUCTION

Foundations with TFS, presented in Chapter 4 were widely employed during the early development stages of the wind industry. However, during the operation of wind turbines, under regular inspections, unexpected issues in the concrete pedestal have been identified over the past years [47] [48]. The most common ones are cracks developed in the pedestal, concrete spalling and unexpected relative vertical movement between the tower and the foundation with records up to 20 mm or more [48]. Figure 5.1 shows two examples of the spalling and cracks developed in the concrete pedestal.



Figure 5.1 – Concrete spalling at the interface between the steel tower and concrete pedestal (photo on the right is courtesy from Kay Bode)

The study conducted in [48] investigated multiple wind farms in order to identify the cause of the vertical movements observed at the base to the tower. Through endoscopic filming of boreholes drilled in the foundations, the investigation revealed a considerable number of voids filled with water beneath the embedded T-flange. Additionally, significant erosion of the concrete was observed on the upper side of the T-flange. These voids have contributed to excessive vertical displacement observed at the tower base, ultimately leading to concrete cracking. Another study developed by [49] on repair of wind turbine

foundations identified the same type of voids between the concrete and the embedded T-flange, as shown in pictures from Figure 5.2.

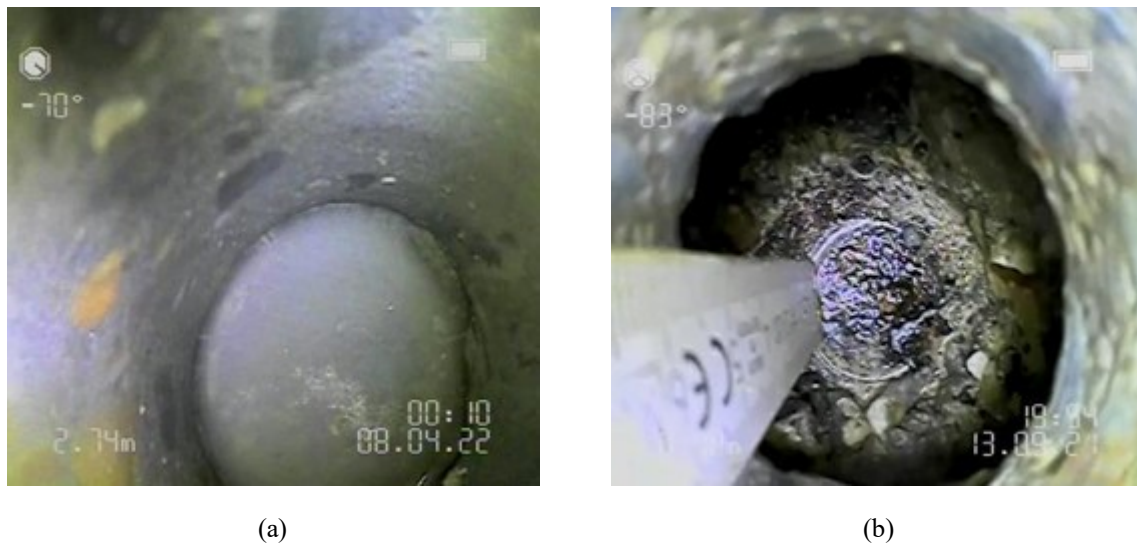


Figure 5.2 – Gap between the embedded T-flange and the concrete above: (a) 1 mm thickness; (b) > 50 mm thickness (extracted from [49])

As schematically represented in Figure 5.3, the gaps can be attributed to the air entrapped under the steel flange during the casting/concrete vibration procedures. These regions of porous concrete are also more prone to degradation under cyclic loading effects, thereby further increasing these gaps.

The larger the gaps, the larger the relative movements between the steel tower and the surrounding concrete. This may have a significant impact on the operation of the wind turbine due to:

- Modified dynamic behaviour of the wind turbine system impacting the control algorithms of the wind turbine;
- Increased displacements and acceleration amplitudes at the rotor impacting the operation of the generator and the tower stresses;
- In extreme situations, damage of the concrete in contact with the TFS is observed, mostly spalling caused by the upward frictional forces (see Figure 5.1).
- Whenever water infiltrates through the interface between the TFS and the surrounding concrete, the relative movements generate pumping pressures which further accelerate the degradation of the concrete.

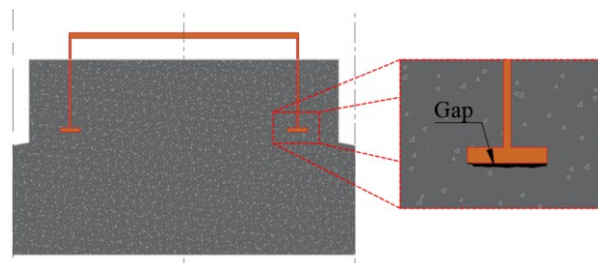


Figure 5.3 – Detail of the gap location beneath the TFS flange

The industry-standard methods for foundation repair involve the injection of grouts or resins to fill voids. This process often requires expensive drilling operations and results in downtime periods. The study presented in [50] examines the advantages and disadvantages of various repair methods, specifically focusing on the use of resins and cement-based materials to fill these voids. In cases of severe damage, various strengthening and retrofitting methods have been proposed. A strengthening method was proposed by [51], which includes the addition of welded studs, the application of circumferential prestressing, and the increase of the embedded depth of the steel ring. The study concludes that these measures reduce stress concentration and improve the fatigue life of the structure. Another study conducted by [52] proposed a combined-connection retrofitting method. This method involves the addition of anchor bolts to the existing wind turbine foundation with TFS.

However, there remains a lack of clarity and consensus on strategies for the safety assessment of existing foundations using the TFS system, particularly when gaps are present between the embedded steel flange and the concrete. In particular, and considering the costs involved in the repair works, objective criteria need to be established for triggering a repair intervention.

5.2 NUMERICAL MODEL

The influence of pre-existing damage in the TFS foundations, in the form of gaps between the embedded T-flange of the TFS and the surrounding concrete, is assessed in this section by using the numerical model developed in Chapter 4, modified to allow for the simulation of gaps and study its effect on the structural integrity of the TFS and concrete foundation. The modification was applied to the interface affected by the gap (Figure 5.5). Here, a bilinear law was employed relating the normal relative displacement and the normal stress. Null tensile strength was assumed, with the shear being also null when detachment occurs. Figure 5.4 compares the bilinear laws for both the interface without a gap and the interface with a gap of 1 mm.

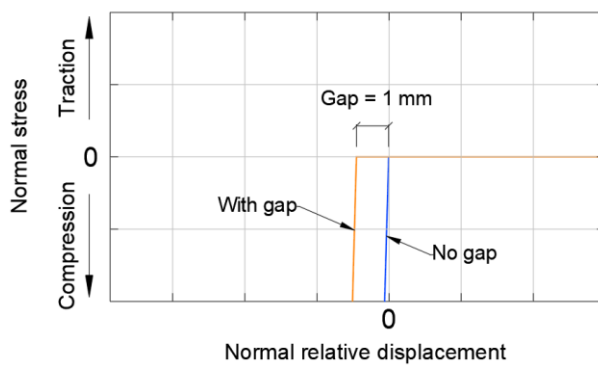


Figure 5.4 – Relationship between normal displacement and normal stress used to represent the gap

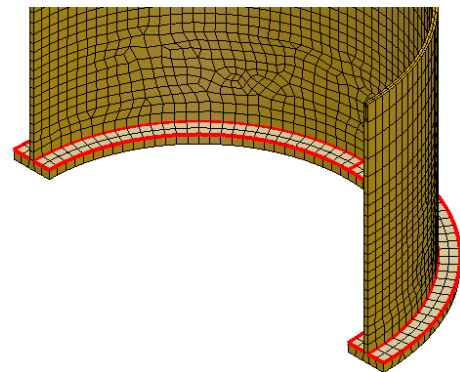


Figure 5.5 – Location of the interface where the bilinear gap law is applied (highlighted in red)

In addition, to more accurately capture the influence of the gap between the TFS and the concrete pedestal, the mesh was further refined in the elements representing these regions, as illustrated in Figure 5.6.

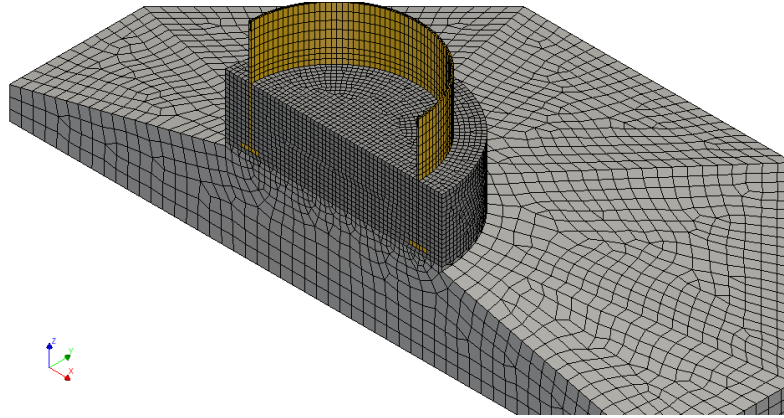


Figure 5.6 – TFS foundation mesh

5.3 INTERFACES - PARAMETRIC STUDY

The substantial surface contact between the TFS and the adjacent concrete may suggest that parameters such as friction and cohesion between these two materials influence the rotational behaviour of the system. To better understand the impact of these parameters, a comprehensive parametric study was conducted. The study systematically varied three key parameters: gap thickness, friction coefficient, and cohesion. The parameters adopted in the numerical simulations are presented in Table 5.1. The gap thickness value of 1 mm is commonly encountered in this type of foundation, while gaps in the 1 mm to 5 mm range represent the threshold at which many wind farm operators consider the need of repair works. The friction angle values encompass a broad spectrum, from 0° (which represents an extreme scenario of frictionless interface) to 45° , a reasonable upper bound given the nature of the steel to concrete surfaces. The cohesion values are small and were tentatively selected, with the 80 kPa value providing the best fit to the experimental data presented in Chapter 3.

The analysis was conducted assuming that both the concrete and steel behave as linear elastic materials. As demonstrated in [53] and in Chapter 4, no significant concrete cracking is expected under serviceability loads, and the materials (concrete and steel) are expected to remain within the linear regime. This approach enhanced computational efficiency while enabling to focus the investigation on the nonlinear behaviour of the interfaces between the steel elements and the surrounding concrete, and its effect on the rotational response of the structure.

The conclusions drawn from this study will be used to define the mechanical properties of the interfaces. These properties will serve as a basis for evaluating the impact of the gaps on the structural response of the foundation and on the dynamic behaviour of the wind turbine.

Table 5.1 – Properties of the horizontal and vertical interface between steel and concrete

	Normal stiffness, k_n (kN/m ³)	Tangential stiffness, k_t (kN/m ³)	Gap thickness (mm)	Friction angle ($^\circ$)	Cohesion (kPa)
Scenario 1	2×10^{12}	2×10^8	0, 1 and 3	0	0
Scenario 2			1	0, 30 and 45	0
Scenario 3			1	30	0, 40 and 80

5.3.1 MOMENT-ROTATION CURVES

The results from the parametric study are presented from Figure 5.7 to Figure 5.9. One load cycle was applied, varying the bending moment at the base of the tower between $\pm M_{SLs}$. Initially, the gap thickness was varied while maintaining the friction angle and cohesion null. In the next step of the parametric study, the gap was kept constant along with the cohesion, while the friction angle was varied. Finally, both the gap and friction angle were held constant while the cohesion was changed. Figure 5.10 shows the moment-rotation curves at the base of the tower and at the pedestal, considering a gap thickness of 1 mm, a cohesion of 80 kPa, and a friction angle of 30° .

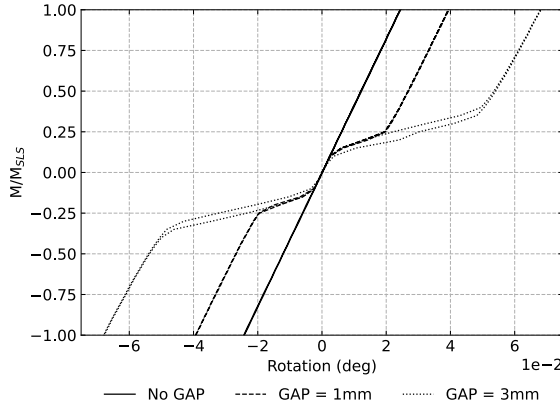


Figure 5.7 – Moment-rotation curve of the tower bottom section with varying gap thickness, assuming frictionless and cohesionless interface

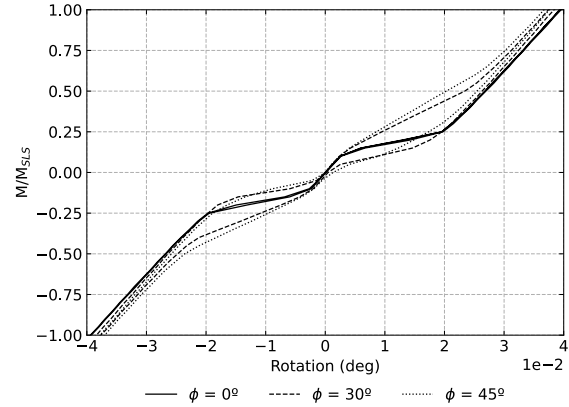


Figure 5.8 – Moment-rotation curve of the tower bottom section with varying friction angle, assuming cohesionless interface and gap thickness of 1 mm

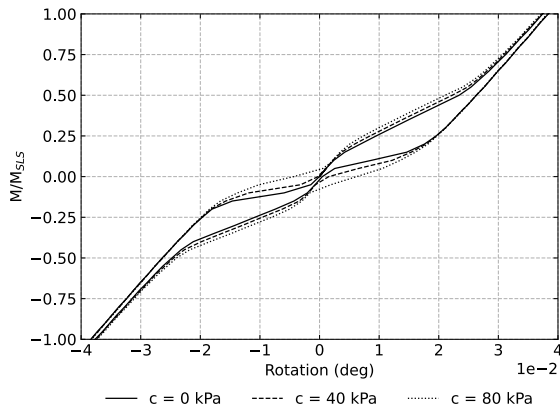


Figure 5.9 – Moment-rotation curve of the tower bottom section with varying cohesion, assuming friction angle of 30° and gap thickness of 1 mm

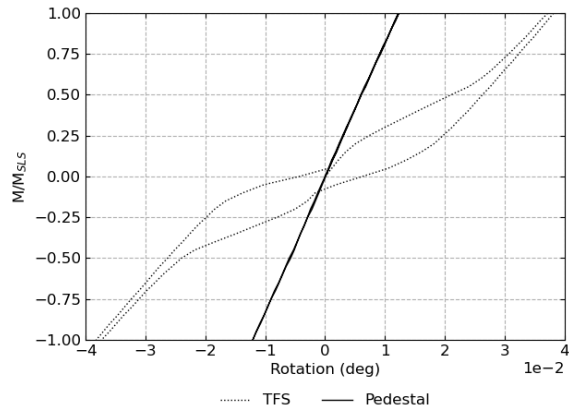


Figure 5.10 – Comparison of the moment-rotation curve at: (1) TFS; (2) pedestal. The analysis considers a cohesion of 80 kPa, a friction angle of 30° , and a gap thickness of 1 mm.

The influence of the interface between the TFS and the surrounding concrete is clearly visible in the moment-rotation curves, with a substantial increase in the tower base section rotation for scenarios involving gaps, as illustrated in Figure 5.7. The inclusion of gaps shows an increase in the tower base section rotation, reaching a factor of 5 for the scenario with a gap thickness of 3 mm. Up to a load factor

of approximately $0.1M/M_{SLS}$, the moment-rotation relation remains linear and unchanged across all the simulations. After $0.1M/M_{SLS}$, the self-weight of the wind turbine is overcome, causing the detachment of the T-flange from the concrete, and there is a sudden increase in the rotation of the tower bottom section in the simulations where gaps are considered. Once the gap thickness is overcome, the system recovers the initial stiffness. This effect is evident in Figure 5.7, where the curves become approximately parallel to the curve without gap once the gap is closed. When frictionless interfaces between the TFS and the surrounding concrete are considered, the relationship between rotation and bending moment remains unchanged upon load removal, regardless of whether or not gaps exist between the TFS and the surrounding concrete, and the gap thickness does not influence the load path behaviour under the loading-unloading cycle.

The effect of the friction between the TFS and the surrounding concrete in the rotation behaviour is illustrated in Figure 5.8, considering a cohesionless interface with a constant gap thickness of 1 mm. A comparative analysis is conducted among three scenarios: (a) frictionless interface; (b) friction angle equal to 30° ; (c) friction angle equal to 45° . Results indicate that friction mitigates the sudden loss of stiffness associated with gaps. Additionally, friction is responsible for delaying the gap closure upon load reversals. Finally, the observations also show that when friction is introduced into the model, a hysteretic behaviour is observed particularly when the gap is open.

The effect of the cohesion between the TFS and the surrounding concrete is shown in Figure 5.9, considering a friction angle of 30° and a constant gap thickness of 1 mm. A comparison is made between: (a) no cohesion; (b) cohesion equal to 40 kPa; (c) cohesion equal to 80 kPa. The hysteresis effect becomes more noticeable as cohesion increases, with residual rotations becoming evident for null bending moment.

The comparison of the bending moment versus rotation curves at the base of the tower and at the pedestal, as illustrated in Figure 5.10, indicates that under serviceability loads and in the presence of gaps as small as 1 mm, the tower undergoes rotations that are over three times larger than those observed in the concrete pedestal.

The next step in the parametric study was to investigate the effect of the gap location. In reality, once the damage has initiated, the location of the gap can vary randomly. It may appear on either the top or bottom surface of the T-flange, or on both surfaces simultaneously. This variability is influenced by the specific instance at which the measurements are taken, as the dynamic behaviour of the wind turbine during operation can cause the gap location to shift unpredictably. To enable a comparison with the experimental results, the foundation is simulated under three different scenarios: (1) gap located on the top surface of the T-flange, (2) gap located on the bottom surface of the T-flange, and (3) gap located on both surfaces. The corresponding results, depicting the relationship between bending moment and rotation, are presented in Figure 5.11.

The results demonstrate that the initial location of the gap plays a key role in determining the rotational stiffness of the system in the early stages of the loading, specifically up to the point when the contact is re-established between the T-flange and the surrounding concrete, meaning, the gap width is overcome. The simulation in which the gap is located on the top surface of the T-flange exhibits the highest initial rotational stiffness. In contrast, the configuration with the gap on the bottom surface shows the lowest initial stiffness. The recontact between the T-flange and the concrete occurs at lower rotation levels in the case where the gap is located on the bottom surface. In the case where the gap is symmetrically

distributed, the response falls between the two previous scenarios. Once the rotation is sufficient to overcome the gap width and restore full contact at the interface, the moment-rotation curves of all three configurations converge. Beyond this point, the rotational stiffness becomes identical, and the rotations corresponding to the SLS-QP load combination are equivalent across all the three scenarios. This convergence indicates that the initial location of the gap primarily influences the early rotational behaviour, while having negligible impact on the structural response at higher load levels where full contact has been re-established.

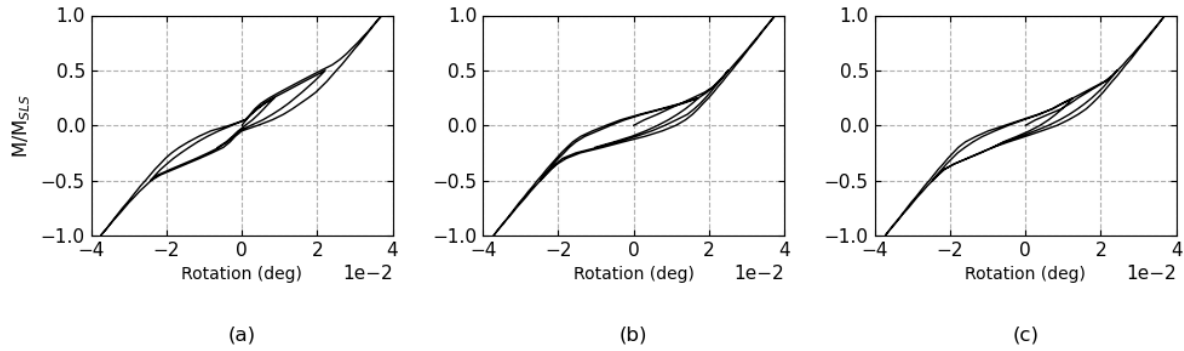


Figure 5.11 – Moment-rotation curves obtained from the numerical simulations with decreasing loading cycles, assuming a cohesion of 80 kPa and a friction angle of 30°: (a) gap with 1 mm located at the T-flange top surface; (b) gap with 1 mm located at the T-flange bottom surface; (c) gap with 1 mm distributed as 0.5 mm at the top surface and 0.5 mm at the bottom surface of the T-flange

5.3.2 COMPARISON WITH EXPERIMENTAL EVIDENCE

5.3.2.1 LVDT SENSORS

To place experimental observations from the LVDT sensors described in Section 3.3 into context and assess the expected relative vertical displacement in the absence of damage, a numerical comparison is required. If both the concrete and the TFS remain undamaged and in full contact, with no voids between the two elements, the relative displacement would solely result from the elastic deformation of these materials. Through numerical simulations, it is possible to accurately determine the displacements induced by elastic deformations. Any measurements recorded by the LVDT sensors that exceed the values obtained from the numerical analysis can therefore be attributed to the presence of gaps at the interface with the embedded T-flange. By employing the numerical model of the TFS foundation developed in Chapter 4 and further refined in this chapter, the relative vertical displacement can be determined as a function of the applied bending moment, assuming the TFS foundation without voids at the connection with the T-flange. This approach provides a reliable method of distinguishing between displacements arising purely from the elastic properties of the materials and those influenced by gaps developed at the interface.

Figure 5.12 presents the relationship between the bending moment and the relative vertical displacements between the TFS and the concrete pedestal. The numerical results, assuming a foundation without gaps, are compared with the experimental data obtained from the LVDT sensors D1 and D4, which are aligned with the wind direction. Two scenarios are considered in the numerical analyses: one without friction between the TFS and the concrete, and another with a friction angle of 45°. In the numerical model, the relative vertical displacement is defined as the difference between the vertical

displacement at the TFS node, located 24 cm above the pedestal surface, and the vertical displacement at the node positioned on the pedestal surface, as schematically illustrated in Figure 5.13.

As previously discussed in Chapter 3, the shutdown events that occurred in 2023 and 2024 recorded a maximum bending moment of $17 \text{ MN}\cdot\text{m}$ ($\approx 0.42M_{SLs}$) immediately prior to the initiation of the shutdown event, and a peak negative value of $-33 \text{ MN}\cdot\text{m}$ ($\approx 0.81M_{SLs}$) during the event. The plot in Figure 5.12 shows the calculated relative vertical displacements for a foundation without gaps when subjected to the same bending moment range observed during these shutdown events. This is shown for both frictionless and frictional contact conditions between the TFS and the concrete. The experimental data from the LVDT sensors D1 and D4 are included in the same plot to enable a direct comparison between the numerical response of an undamaged foundation and the measured behaviour of a potentially damaged foundation.

For the given load range, and in the absence of gaps, the relative vertical displacements are caused by the elastic deformation of the steel and concrete. In the present analysis, the calculated displacements for the observed moment range reach 0.58 mm when a friction angle of 45° is assumed, and approximately 0.68 mm when no friction is considered at the interface. When compared with the experimental data, it becomes evident that the LVDT sensor D4 records relative displacements that significantly exceed the calculated values, suggesting the presence of gaps between the T-flange and the concrete. On the other hand, the measurements from the LVDT sensor D1 closely align with the numerical results, indicating that the interface in the vicinity of this sensor likely remains in full contact, or has minimal gap formation. These observations provide evidence that the foundation of the wind turbine is affected by localised gaps, with more expression near the location of the sensor D4.

The influence of the gap may also explain why the centre of the cloud of points recorded by the sensors D1 and D4 does not coincide with the origin of the data from the numerical results. Gaps may introduce an offset in the effective axis of rotation of the system. This axis is determined during the zeroing of the experimental data. The adopted zeroing procedure consists of shifting the displacement results such that the point where the oscillation amplitude tends to zero is taken as the reference origin. However, due to the action of the friction at the interface between the TFS and the surrounding concrete, this procedure may not ensure that the origin of the experimental dataset coincides with the one assumed in the numerical model.

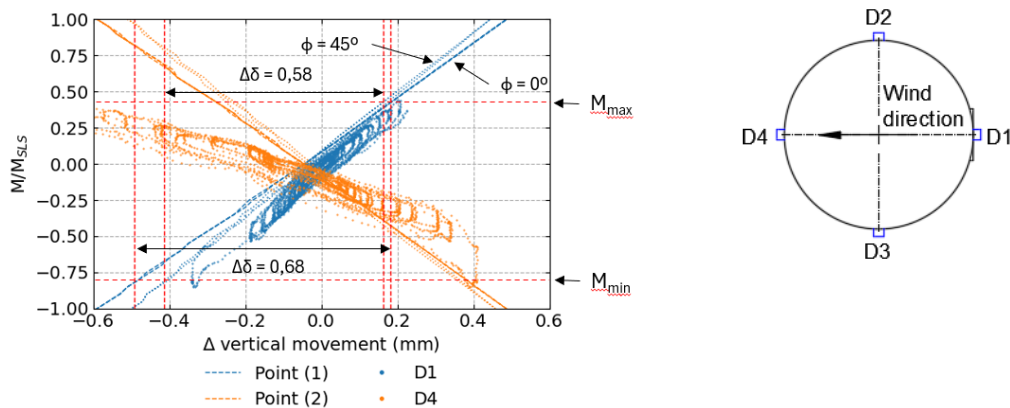


Figure 5.12 – Relative vertical displacements between the tower base section and the concrete pedestal surface obtained from the numerical simulations for two scenarios: one without friction between the TFS and concrete, and another with a friction angle of 45° . Results are compared with the experimental measurements from LVDT sensors D1 and D4

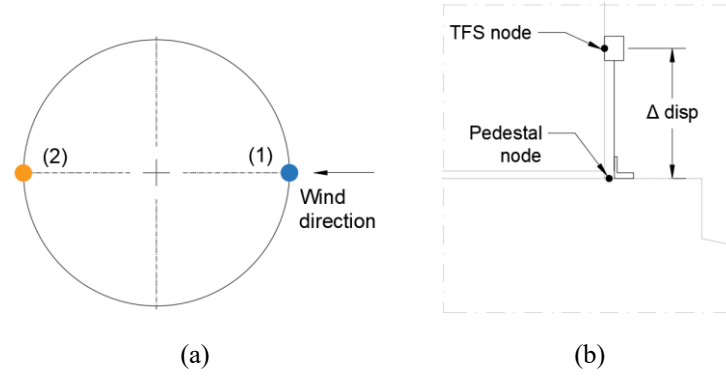


Figure 5.13 – Location of the measurement points in the numerical model: (a) top view; (b) lateral view

5.3.2.2 INCLINOMETERS

The rotations observed during the forced shutdown events occurred on 24/10/2023 and 11/10/2024 were recorded by the two inclinometers, as described in Chapter 3. The experimental results have been compared with the numerical simulations where the gap thickness was maintained 1 mm around the entire perimeter, while the friction angle and the cohesion were set to 45° and 80 kPa, respectively. The resulting moment-rotation curves are depicted in Figure 5.14.

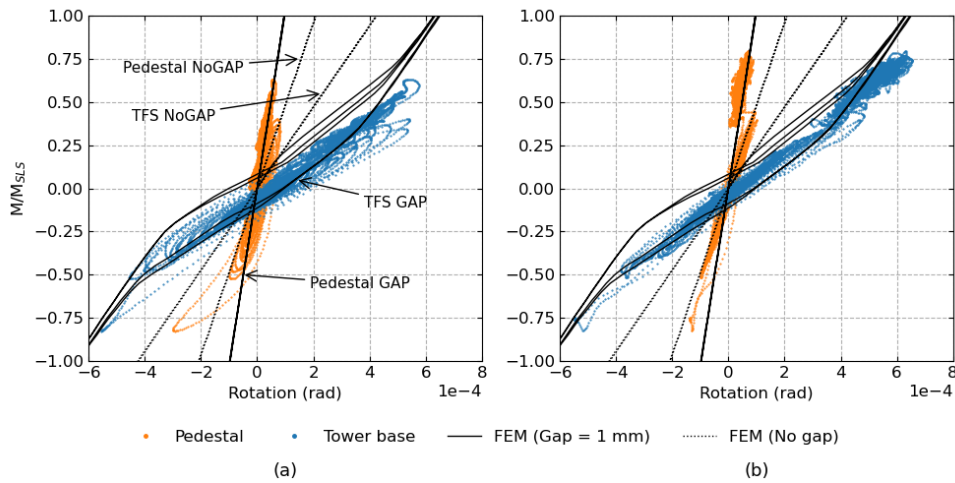


Figure 5.14 – Correlation between bending moment and rotation in the pedestal and TFS. Comparison between numerical simulations and experimental observations for the shutdown events on: (a) 24/10/2023; (b) 11/10/2024

The moment-rotation curves evaluated at the base of the tower by the inclinometers exhibit a response that closely follows the numerical predictions for the damaged foundation. The presence of hysteresis and the amplitude of the measured rotations strongly suggest that the foundation is in a damaged state, likely with a gap exceeding 1 mm. The inclusion of a gap with 1 mm in the numerical model not only introduces hysteresis but also more than doubles the rotation the TFS when compared to the simulation without the gap. This outcome aligns well with the experimental data, reinforcing the accuracy of the model in capturing the mechanical response of the damaged condition.

The rotations measured at the pedestal show very limited signs of hysteresis and follow a response similar to a linear-elastic behaviour, particularly evident during the event that occurred on 11/10/2024.

It is also worth noting that the pedestal appears stiffer when the foundation is in a damaged state. This is because, for the same bending moment, the pedestal rotates less due to the effect of the gap.

The overall consistency between the model and the field data indicates that the modelling strategy provides a reasonable qualitative and quantitative representation of the complex structural interaction at this critical connection. These results validate the modelling assumptions and demonstrate the potential of such approaches for detecting and characterising damage in foundations of wind turbines under operation.

5.4 ANALYSIS FOR SERVICEABILITY CONDITIONS

Following the parametric study in which the influence of the interface properties, namely gap thickness, friction angle, and cohesion, on the structural behaviour of the TFS/foundation system was systematically investigated, along with the validation with experimental data, the analysis now proceeds to assess the serviceability performance of the foundation. This subsequent stage aims to determine the threshold for the gap thickness beyond which begins to compromise the serviceability of the foundation and the overall performance of the wind turbine. The numerical model incorporates the nonlinear behaviour of both the concrete and the reinforcement to provide a realistic representation of the foundation response. The constitutive model and material properties employed in the analysis are detailed in Chapter 4 and summarised in Table 4.1 and Table 4.2.

Table 5.2 – Concrete properties

Concrete properties	C35/45	C30/37
Tensile strength, $f_{ct,k}^*$ (MPa)	2.00	1.81
Fracture energy in tension, $G_{f,k}^*$ (kN/m)	0.090	0.088
Compressive strength, $f_{c,k}^*$ (MPa)	30.1	25.8
Fracture energy in compression, G_{ck} (kN/m)	40.34	39.45
Young's modulus, E_{cm} (GPa)	34	33
Poisson's ratio, ν	0.2	

Table 5.3 – Steel properties

E_s (GPa)	$\epsilon_{y,k}$ (%)	$\epsilon_{u,k}$ (%)	$f_{y,k}^*$ (MPa)	$f_{t,k}$ (MPa)
200	0.244	5.0	489	528

5.4.1 CRACK PATTERNS

Figure 5.15 shows the computed crack patterns for the undamaged foundation and for foundations with circumferential gaps of 1 mm and 2 mm, respectively, under SLS-QP load combination. The crack patterns clearly demonstrate the progressive deterioration of the structural performance of the foundation with the increase of the gap thickness.

In the foundation with the gap of 2 mm (Figure 5.15 (c)), two significant cracks exceeding the critical width threshold of 0.3 mm, established by the standard EN 1992-1 [24] for serviceability conditions, are identified. The first prominent crack develops horizontally, extending approximately halfway around

the pedestal perimeter. This extensive horizontal cracking indicates substantial tensile stress concentrations near the pedestal surface, reflecting the asymmetric contact conditions caused by the gap. The second critical crack is a radial crack oriented at approximately 90° to the symmetry plane of the foundation. The occurrence of these substantial cracks at relatively low load levels clearly indicates the tensile stress amplification introduced by the presence of the gap of 2 mm.

In the case of the foundation with the 1 mm gap (Figure 5.15 (b)), the crack pattern at the SLS-QP load combination already exhibits the initial formation of a horizontal crack exceeding 0.3 mm width. However, this crack is significantly less developed when compared to the simulation with the 2 mm gap.

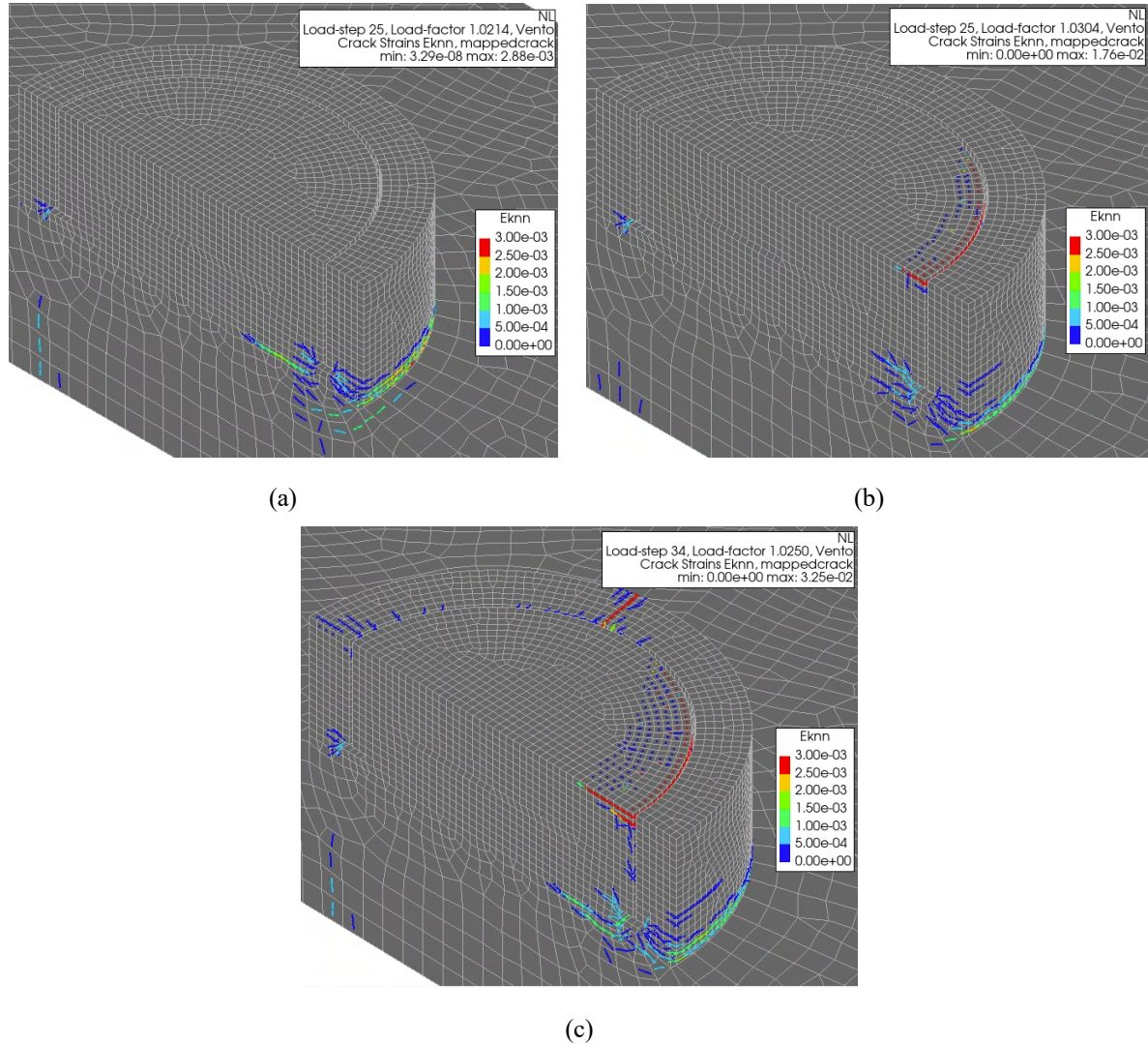


Figure 5.15 – Crack strain (Eknn) pattern: (a) no gap; (b) gap = 1 mm; (c) gap = 2 mm

5.4.2 CONCRETE COMPRESSIVE STRAINS

Figure 5.16 presents the distribution of the concrete compressive strain in the foundations with 1 mm and 2 mm circumferential gaps. The results reveal a clear distinction in the extent of the damage between the two cases.

In the case of the 2 mm gap (Figure 5.16 (b)), a significant region of the concrete pedestal exceeds the strain corresponding to the peak compressive strength of the material. This is evidenced by the blue-shaded area in the figure, which indicates that the material has surpassed its ultimate strength capacity and entered the softening domain which is characterized by localization of the deformation in severely damaged zones. Approximately 13% of the pedestal located inside of the tower is affected, with the damage being restricted to the unconfined top layer of concrete, leading to premature spalling. The presence of such compressive damage may compromise the long-term durability and safety margin of the foundation, particularly under repeated loading conditions.

In contrast, the foundation with 1 mm gap (Figure 5.16 (a)) does not exhibit any indication of concrete damage under compression. The strain distribution remains below the threshold associated with the maximum compressive capacity of the concrete, suggesting that the structural integrity of the pedestal remains preserved in this scenario. This finding reinforces the conclusion that small gaps, up to 1 mm, may not significantly compromise the structural performance of the foundation under serviceability loading conditions.

Another important observation is that only the windward side of the pedestal experiences damage under compression, with no damage observed in the leeward side. This phenomenon can be explained by the formation of the radial crack in the pedestal, described in the previous section. This radial crack is responsible for relieving of the compressive stresses on the leeward side of the pedestal.

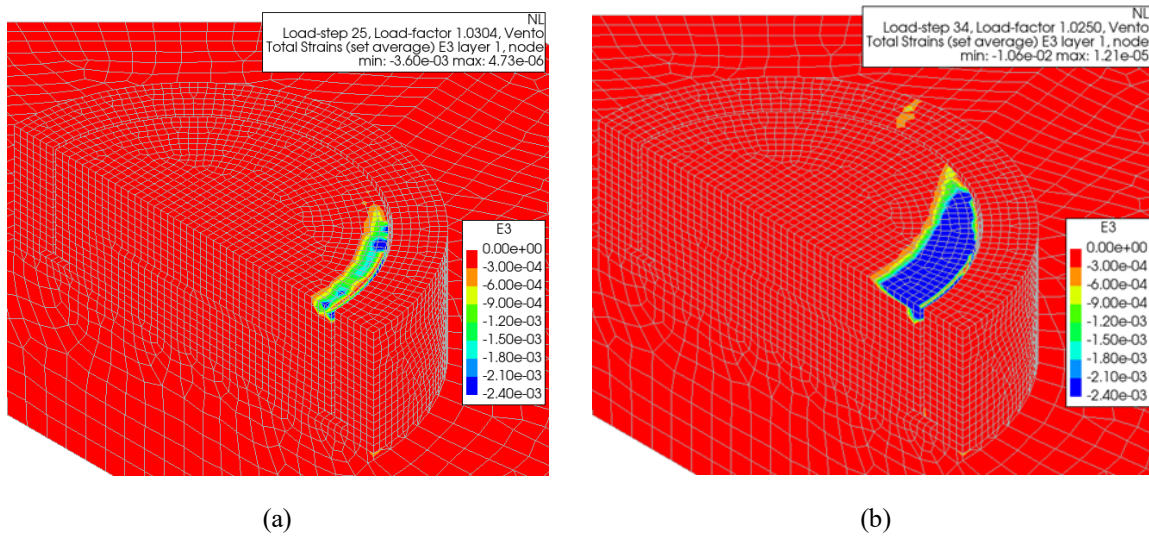


Figure 5.16 – Concrete compressive strain (E3): (a) gap = 1 mm; (b) gap = 2 mm

5.4.3 MOMENT-ROTATION CURVES

This section investigates the influence of circumferential gaps between the T-flange and the surrounding concrete on the rotational behaviour of the tower-foundation system. The evaluation is conducted by analysing the moment-rotation curves obtained for different gap scenarios.

Figure 5.17 shows the moment-rotation curves evaluated at the TFS, assuming an interface friction angle of 30° and a cohesion of 80 kPa between the embedded steel and the surrounding concrete. The

comparison includes three scenarios: an undamaged foundation, and foundations with circumferential gaps of 1 mm and 2 mm, respectively.

The undamaged foundation exhibits linear moment-rotation relationship up to around 90% of the SLS-QP load combination, at which point initial cracking occurs at the base of the pedestal, marking the transition from linear elastic behaviour to nonlinear response.

The introduction of gaps notably reduces the rotational stiffness at the tower base, resulting in increased rotations under similar load conditions. Specifically, the presence of a gap of 1 mm results in approximately double the rotation observed for the undamaged case. For the foundation with a gap of 2 mm, the increase in rotation is even more pronounced, with a rotation magnitude approximately 2.5 times greater compared to the undamaged foundation. These observations highlight the sensitivity of the rotational stiffness of the foundation to relatively small increments in gap width. Moreover, the analysis of the foundation with a gap of 2 mm reveals significant localised concrete damage at a load level of only about 32% of the SLS-QP load combination. Notably, concrete crushing occurs at this relatively low load level, even before full contact is re-established between the upper surface of the T-flange and the surrounding concrete. This early manifestation of concrete damage indicates that the presence of gaps with 2 mm can induce highly localised compressive stresses capable of compromising the structural integrity of the pedestal at relatively low load intensities.

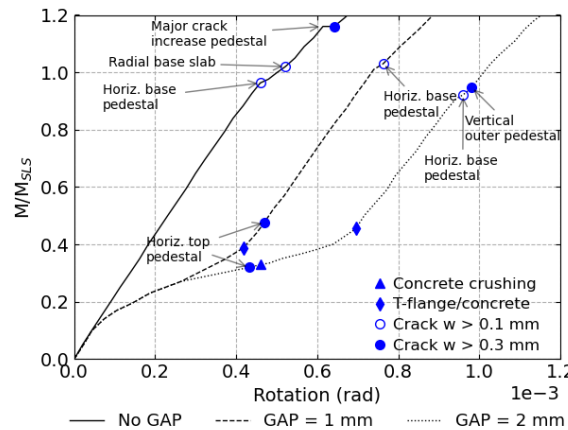


Figure 5.17 – Moment-rotation curves obtained from the numerical simulations, assuming a cohesion of 80 kPa and a friction angle of 30° between the TFS and the concrete. Comparison between scenarios with no gap, 1 mm gap, and 2 mm gap with the analysis conducted up to 20% above SLS-QP (♦ establishment of contact between the upper surface of the T-flange and the concrete)

5.4.4 MODIFICATION OF THE MOMENT TRANSFER MECHANISM

Figure 5.18 presents the distribution of concrete compressive stress in the pedestal, represented by the third principal stress component, S_3 , obtained from the nonlinear finite element analysis. Since S_3 corresponds to the most compressive principal stress, the plotted field provides direct evidence of the force transfer mechanism mobilised at the tower-foundation interface under the applied bending moment. Two configurations are compared: undamaged foundation (no gap) in Figure 5.18 (a) and damaged foundation with a 2 mm gap in Figure 5.18 (b). To support the interpretation of these numerical fields, Figure 5.19 provides simplified schematic representations of idealised stress transfer mechanisms. These schemes intentionally focus on the bending moment, since this stress resultant

governs the response at the interface, whereas the accompanying vertical and horizontal forces and the torsional moment are substantially smaller and therefore secondary in the present discussion.

For an undamaged foundation, Figure 5.19 (a) illustrates the idealised mechanism in which the applied bending moment is equilibrated primarily by a vertical force couple acting along the embedded T-flange. The contact is established between the embedded T-flange and the concrete, enabling opposing vertical reactions that generate the required couple. In practice, however, the force transfer is more realistically described by the intermediate mechanism in Figure 5.19 (b). In this case, in addition to the vertical reactions, shear stresses develop along the vertical joints (between concrete and steel shell), represented by τ_1 and τ_2 , contributing to moment equilibrium and redistributing the transfer of stresses within the pedestal. This combined mechanism is consistent with the numerical response observed in Figure 5.18 (a), where the compressive stress field is comparatively more distributed, indicating that the bending moment is transferred through a broader effective contact region.

When a gap is present, assuming that the joint opening occurs on the tension side (tower uplift force side), a substantial portion of the joint between the concrete and the T-flange loses contact, as evidenced by the stress redistribution in Figure 5.18 (b). Under these conditions, the vertical reaction required to form the vertical force couple becomes significantly reduced or absent. Consequently, the stress transfer mechanism transitions towards the extreme scenario illustrated in Figure 5.19 (c). The applied bending moment is then primarily equilibrated by a horizontal force couple. This horizontal component may be associated with tangential contact actions, such as friction, and becomes essential to satisfy global equilibrium when the vertical couple cannot be maintained. However, in reality, the transfer force mechanism, in case of damaged foundations, is somewhere between scenario (b) and (c) with the applied bending moment being equilibrated by the horizontal force couple, shear stress develop along the vertical interfaces and the vertical reaction concentrated on the compression side. The resulting concentration of compressive stresses observed in Figure 5.18 (b) and crack pattern shown in Figure 5.15 (c) are consistent with this modification in the equilibrium mechanism.

Overall, the results indicates that an undamaged foundation is most appropriately represented by the intermediate mechanism in Figure 5.19 (b), while a damaged foundation with gaps tends towards the extreme mechanism in Figure 5.19 (c). This observation is structurally significant, since it indicates that interface gaps do not merely amplify stresses, but also changes the fundamental equilibrium mechanism typically assumed in the design phase.

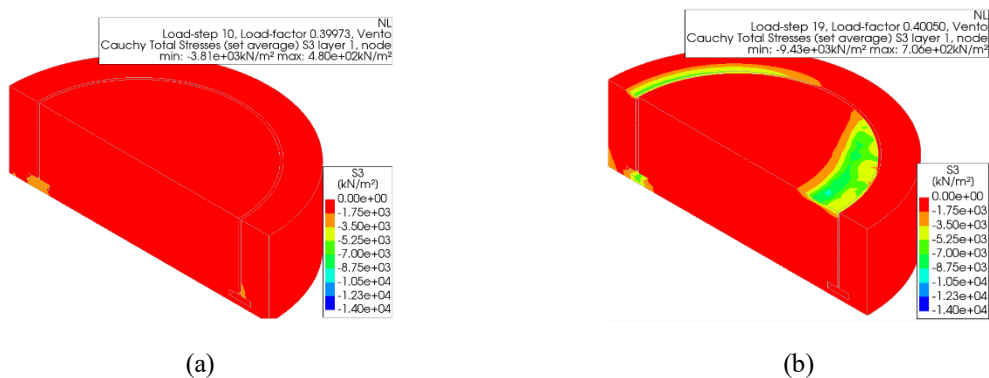


Figure 5.18 – Concrete compressive stress (S3): (a) no gap; (b) gap = 2 mm

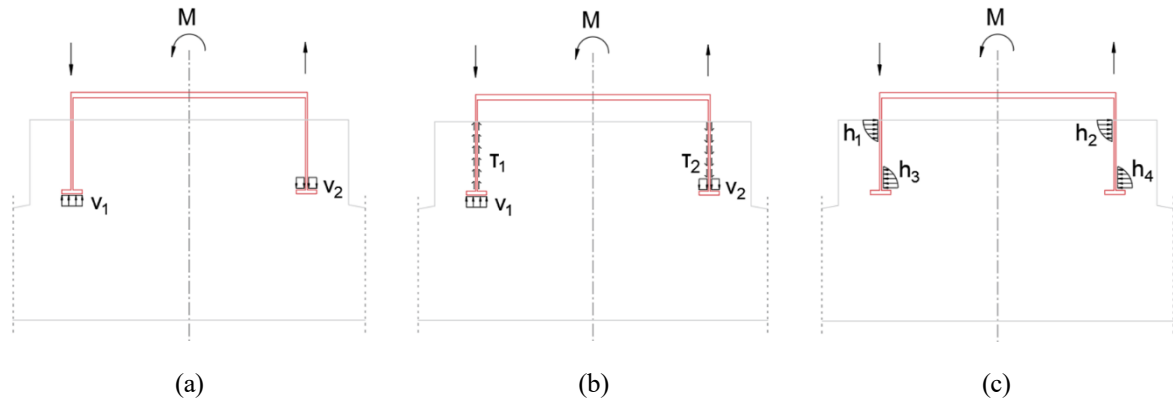


Figure 5.19 – Schematic bending moment transfer mechanisms in the TFS foundation: (a) undamaged foundation (idealised); (b) undamaged foundation (most representative, including shear transfer along the vertical joints T_1 and T_2); (c) damaged foundation (extreme scenario). Schemes are simplified and consider bending moment only

5.5 ANALYSIS UNDER EXTREME LOAD CONDITIONS

To understand the structural implications of interface gaps under extreme events, the numerical simulations were extended through proportional scaling of the SLS-QP load set up to approximately 200%. This scaling level was selected because it corresponds to the ULS load combination provided by the WTG supplier. The investigation was restricted to the undamaged foundation and the configuration with a 1 mm gap. The 2 mm gap case was excluded because, as shown previously, concrete damage initiates at a comparatively low load level (approximately $0.34 \cdot M_{SLS}$). This early onset indicates that a 2 mm discontinuity can generate highly localised compressive stress concentrations at the pedestal, potentially compromising structural integrity even under moderate load intensities.

5.5.1 CRACK PATTERNS

Figure 5.20 compares the computed crack patterns at a load level corresponding to approximately twice the SLS-QP load combination for the two foundation scenarios: (a) undamaged foundation, and (b) damaged foundation with gap of 1 mm. A close analysis of both images reveals that, at high load levels, the crack propagation patterns become similar between the two simulations. In each case, pronounced radial cracks develop predominantly around the pedestal perimeter and extend significantly into the slab region, forming similar patterns in terms of both spatial distribution and crack density. Additionally, the extent and location of crack concentrations at critical structural zones, such as the connection between the pedestal and the slab and along the vertical pedestal surface, closely match across both simulations.

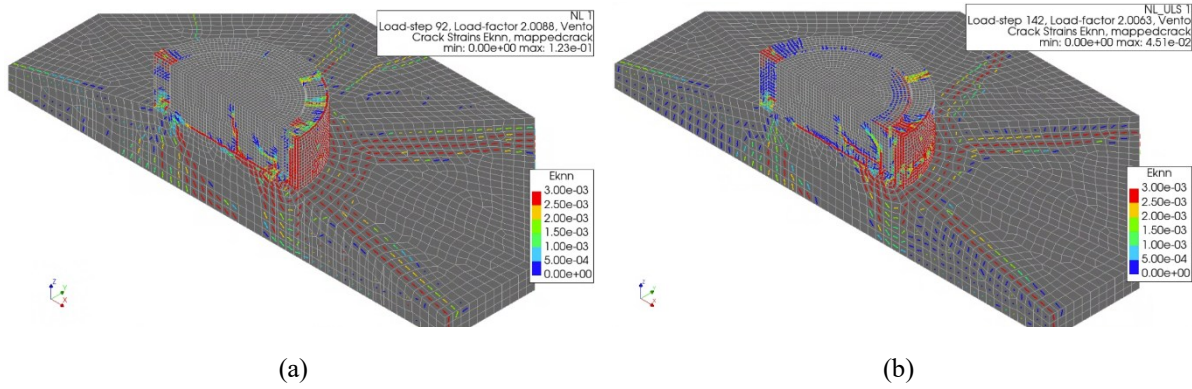


Figure 5.20 – Crack strain (E_{knn}) pattern: (a) no gap; (b) gap = 1 mm

5.5.2 STRESSES IN THE TFS

Figure 5.21 presents the von Mises equivalent stress distribution in the TFS for a load level corresponding to approximately twice the SLS-QP load combination, comparing (a) the undamaged foundation, and (b) the damaged foundation with a circumferential gap of 1 mm. The TFS is modelled considering a steel grade S355J0 with a shell thickness of 48 mm. According to EN 10025-2:2019 [54], the characteristic yield strength for this thickness range is $f_y = 335$ MPa.

The results in Figure 5.21 indicate a von Mises peak stress of approximately 181 MPa for the undamaged foundation and 177 MPa for the foundation with 1 mm gap, corresponding to elastic utilisations of about 0.54 and 0.53 relative to f_y . The spatial stress patterns are closely comparable between both cases, and no additional localised concentrations are observed that can be attributed to the presence of the gap. The difference in peak stress is limited to approximately 2% which is consistent with the expected numerical variability for models of this type, where results are sensitive to mesh density, contact penalty/tolerance settings, load-step sequencing, and minor differences in applied load (2.0088 for the undamaged foundation and 2.0063 for the damaged foundation). Overall, the introduction of a 1 mm circumferential gap has a negligible influence on the global TFS stress state at this load level, and the steel remains well below the yield limit.

The configuration with a 2 mm gap was not simulated up to the ULS load level considered in this section. As shown previously, this damage state leads to an early onset of concrete distress and highly localised compressive stress concentrations at the pedestal, such that, from an operational and safety perspective, the foundation would already require intervention before extreme loading conditions were assessed. Nevertheless, an additional analysis limited to a lower load level (up to approximately $1.2 \cdot M_{SLS}$) indicates that the von Mises equivalent peak stress in the TFS is 125 MPa, which remains well below the yield strength, f_y . This observation reinforces that, for the 2 mm gap configuration, the governing limitation is associated with the concrete and interface behaviour rather than yielding of the TFS.

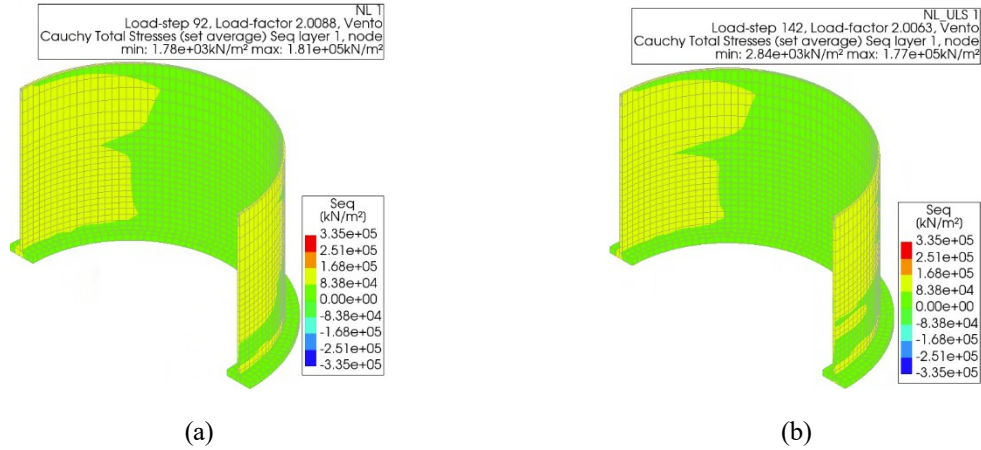


Figure 5.21 – Stress in the TFS for 2xSLS-QP load combination (Von Mises equivalent stress): (a) no gap; (b) gap = 1 mm

5.5.3 MOMENT-ROTATION CURVES

The comparative analysis presented herein focuses on the moment-rotation relationship of the undamaged foundation and the foundation incorporating a gap of 1 mm. The resulting moment-rotation curves for both scenarios are depicted in Figure 5.22. Critical events occurring throughout each simulation are explicitly marked on the curves. It is important to note that crack initiation was identified based on a threshold crack width exceeding 0.3 mm, in accordance with the criteria established in EN 1992-1-1 [24]. Only the formation of the first significant crack is indicated in the figure to maintain visual clarity. Further propagation or widening of existing cracks across the same surface was omitted from the figure to avoid unnecessary complexity.

Figure 5.22 clearly indicates that at lower load levels (below approximately 1.5 times M_{SLS}), the damaged foundation exhibits higher rotations compared to the undamaged foundation. This confirms that the presence of gaps significantly reduces the initial rotational stiffness of the foundation, thereby affecting the serviceability performance, as previously discussed in Section 5.4.3. However, at higher load levels (approximately 1.75 times M_{SLS} and above), the moment-rotation curves for the undamaged and damaged foundations tend to converge. This convergence indicates that under extreme loading conditions, the structural responses of both foundations become similar, suggesting that at these high load levels, the cracking propagation and concrete damage evolution occur in both cases with the same intensity, thus reducing the relative impact of the initial gap on the overall rotational response of the foundation.

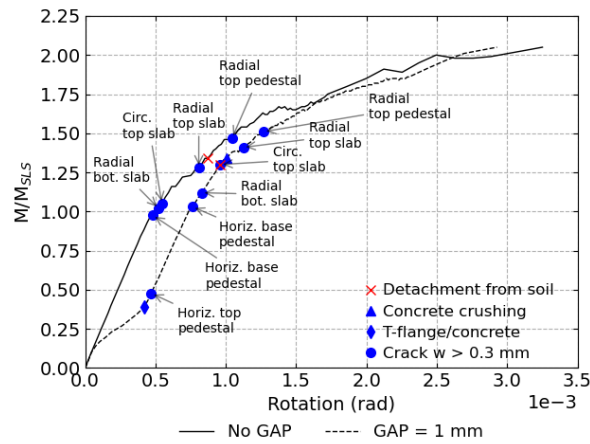


Figure 5.22 – Moment-rotation curves obtained from the numerical simulations, assuming a cohesion of 80 kPa and a friction angle of 30° between the TFS and the concrete. Comparison between scenarios with no gap and 1 mm gap, with the analysis conducted up to, approximately, 2xSLS-QP (♦ establishment of contact between the upper surface of the T-flange and the concrete)

In conclusion, the results highlight that while the occurrence of gaps below 1 mm significantly influence the foundation stiffness and damage progression at lower and moderate load levels, their impact becomes less significant during extreme loading scenarios, when extensive cracking and concrete damage dominate the structural response. Consequently, for very rare or extreme events, both undamaged and damaged foundation with 1 mm gap may exhibit comparable structural behaviours, indicating that gaps up to 1 mm may not critically compromise the performance of the foundation for extreme load conditions.

5.6 CONCLUSIONS

This chapter investigated the structural implications of defects at the connection between the steel TFS and the concrete foundation, specifically focusing on the formation of gaps at the interface. By employing advanced nonlinear numerical models calibrated with the experimental data from Chapter 3, it was possible to characterise the mechanical behaviour of damaged foundations under both serviceability and extreme loading conditions.

The parametric study on interface properties revealed that the rotational stiffness of the foundation is highly sensitive to the gap thickness, and to the friction and cohesion governing the shear stresses developing at the steel concrete interfaces. The analysis demonstrated that the presence of gaps significantly increases the rotation at the tower base compared to the concrete pedestal. Furthermore, the inclusion of friction and cohesion was found to be essential for reproducing the hysteretic cycles observed in field measurements, mitigating the sudden loss of stiffness upon gap opening and introducing residual rotations. The validation process confirmed that the numerical model, when incorporating a gap of approximately 1 mm with friction, accurately replicates the nonlinear moment-rotation loops and relative vertical displacements recorded during the forced shutdown events of the monitored wind turbine.

A critical finding of this research is the definition of a damage threshold regarding the serviceability limit state. The analysis showed that foundations with a circumferential gap of 1 mm exhibit reduced

stiffness but maintain structural integrity without significant concrete damage under service loads. However, increasing the gap thickness to 2 mm results in a severe degradation of performance. In this scenario, the rotation increases by a factor of 2.5 relative to the undamaged case, and the onset of concrete crushing occurs at the pedestal interface at load levels as low as 32% of the quasi-permanent load (SLS-QP). This indicates that gap thicknesses exceeding 1 mm likely trigger localised concrete crushing, spalling and extensive cracking under normal operating conditions, thereby establishing an objective criterion for the initiation of repair interventions.

Conversely, the analysis under extreme load conditions (approx. 2xSLS-QP) indicated that the relative influence of the gaps below 1 mm diminishes as the load increases. At these high load levels, the structural response is dominated by global flexural cracking of the reinforced concrete footing. The moment-rotation curves for damaged and undamaged foundations tend to converge, and the stress state in the steel TFS remains largely unaffected by the presence of the gap, remaining within the elastic range. This suggests that gaps below 1 mm do not impair the safety of the tower foundation under extreme loading events.

6 INFLUENCE OF GAPS IN TFS FOUNDATIONS ON WIND TURBINE OPERATIONAL RESPONSE

6.1 INTRODUCTION

The operational implications of voids or gaps at the steel-concrete interface of wind turbines using the TFS connection system are not yet clearly defined, because admissible levels of relative vertical movement remain uncertain and the available recommendations are inconsistent. While the study presented in [48] suggested that vertical movements of approximately 5 mm constitute a threshold beyond which repair actions are required, the investigation developed in [55] indicated that displacements exceeding 1 mm should already prompt remedial intervention. These discrepancies along with the absence of regulatory guidelines or standardised recommendations on this matter, highlight the need for further research to establish definitive threshold for safe operational limits.

Against this background, Chapter 5 undertook a detailed local assessment of gap formation at the embedded steel T-flange, quantifying its influence on the moment-rotation response and on the mechanisms governing force transfer between tower and foundation. The analyses demonstrated that the progressive opening and closing of the interface introduces a pronounced loss of rotational stiffness and a non-linear, hysteretic response. Importantly, the results indicated that gaps exceeding 1 mm can lead to unacceptable cracking and damage in the concrete pedestal, reflecting a fundamental modification of the load transfer mechanism from the tower into the footing and an associated deterioration of structural integrity at the connection.

Such local changes are expected to have system-level consequences, because the foundation and the wind turbine form a dynamically coupled assembly. A reduction in support stiffness and the emergence of hysteretic energy dissipation may modify the natural frequencies and effective damping of the wind turbine, thereby influencing resonance susceptibility, fatigue accumulation, serviceability response, and potentially the power production under operational loading. However, conventional design and assessment procedures typically idealise the foundation as rigid or represent it through linear rotational springs, which cannot capture gap-dependent stiffness degradation generated by the damaged interface.

The present chapter therefore evaluates how the foundation pathology identified in Chapter 5 propagates to the global response of the wind turbine. The validated numerical representations of damaged foundations developed in the preceding chapters are integrated into a comprehensive aero-servo-elastic

simulation framework, enabling time-domain analyses under representative operational conditions. This approach provides a physically consistent assessment of wind turbine performance in the presence of interface gaps, explicitly accounting for both stiffness loss and hysteretic behaviour, and supporting the discussion of operational thresholds and repair requirements on the basis of global structural response.

6.2 WIND TURBINE MODELLING

The previously discussed challenges associated with concrete defects in onshore wind turbine foundations with TFS requires a comprehensive investigation of their impact on the structural behaviour of wind turbines. This chapter aims to establish a robust framework for modelling a representative WTG that will serve as the basis for subsequent analyses.

In the research conducted, the wind turbine was modelled using the commercially available software *Ashes: aero-servo-hydro-elastic simulation* [56]. *Ashes* employs the finite element method (FEM) combined with a co-rotational formulation of beam elements. This combination is crucial for achieving accurate determinations of dynamic responses in structures, especially under conditions of large displacements and rotations. The co-rotational methodology guarantees precision in such complex scenarios. To address nonlinear phenomena, *Ashes* adopts the Newton-Raphson iterative approach. Depending on the nature of the model, the type of analysis required, and the specific characteristics of the load cases at hand, the software is designed to automatically select the most appropriate solver. For handling aerodynamic loads, it employs the blade element momentum (BEM) theory, which facilitates a fully integrated interaction between aerodynamic forces and the structural response. Moreover, *Ashes* is equipped to simulate a variety of wind conditions including steady wind, yawed inflow, wind shear, and different turbulence models effectively. The software also features a customizable proportional-Integral-Derivative (PID) controller for managing the generator torque application and the pitching of the blades, offering a versatile tool in the analysis of wind turbine dynamics. Further insights into the capabilities and technical specifications of *Ashes* are elaborated in the detailed documentation provided by the software [57].

6.2.1 WTG DESCRIPTION AND CALIBRATION

The wind turbine under analysis (Figure 6.1) corresponds to the monitored unit described in Section 3.3. It is a three bladed, variable speed, pitch regulated WTG with a rotor diameter of 88 m, a rated power of 2.5 MW, and a tubular steel tower with a hub height of 85 m. The rotor speed ranges between approximately 5 rpm and 16.5 rpm, and the rated generator speed is 1670 rpm [58].

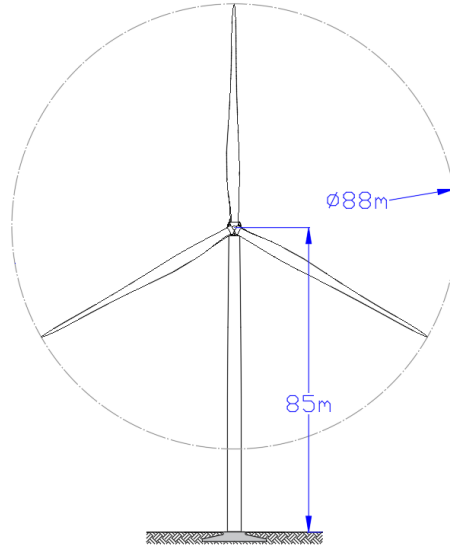


Figure 6.1 – Wind turbine main view

The information provided by the wind farm owner comprises the rotor diameter, the self-weight of the blades, hub, and nacelle assembly (including bottom frame, yaw system, gearbox, and generator), and the mean diameter and self-weight of the tower segments. These data define the total mass distribution with reasonable confidence, but do not uniquely define the mass moments of inertia, the exact centre of mass locations of the rotor nacelle assembly, the detailed tower thickness distribution, or the blade geometry and aerodynamic characteristics. Consequently, a set of physically consistent assumptions was required to complete the aeroelastic input description.

The blade geometry was estimated by scaling a reference blade (NREL 5 MW [59]) to the target rotor diameter, followed by adjustment of the blade mass distribution to match the total blade self-weight. The tower shell thickness distribution and the longitudinal placement of the main concentrated masses (hub and nacelle) were estimated to reproduce the supplied segment masses and nacelle layout constraints. These assumptions were subsequently refined through calibration against field measurements presented in Chapter 3.3 and SCADA data supplied by the owner of the wind turbine.

Since the proprietary turbine controllers were not available, simplified generator torque and collective pitch control laws were adopted and calibrated using SCADA data. For this study, a simple variable-speed control system was adopted, based on the work of Jonkman and Buhl [60]. The generator torque is determined based on the filtered generator speed, divided into five control regions: 1, 1½, 2, 2½, and 3. Region 1 is the pre-cut-in wind speed control region, where the generator torque is zero, and no power is extracted from the wind; instead, the wind accelerates the rotor for start-up. Region 2 focuses on optimizing power capture, with the generator torque proportional to the square of the filtered generator speed to maintain an optimal tip-speed ratio. In Region 3, the generator power remains constant. Region 1½ serves as a start-up region, providing a linear transition between Regions 1 and 2, and setting a lower limit on the generator speed to control the operational speed range of the wind turbine. Region 2½ offers a linear transition between Regions 2 and 3, with a torque slope matching that of an induction machine. Figure 6.2 (a) presents the theoretical curve employed to define the input parameters for the numerical model. Figure 6.2 (b) displays the curve derived from the SCADA data, which serves as the basis for calibrating the numerical model.

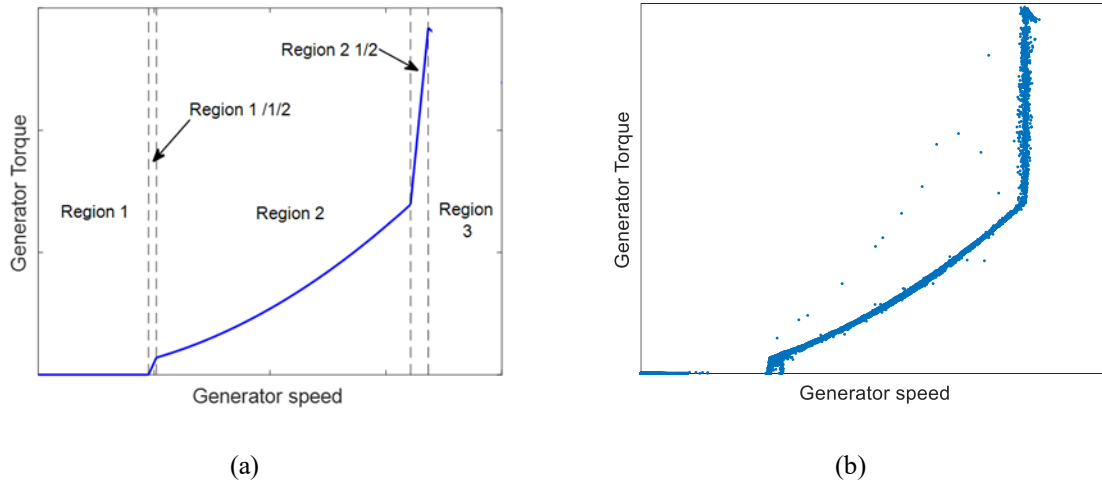


Figure 6.2 – Torque/Speed curve for simple variable-speed control: (a) theoretical curve adapted from [60]; (b) data from SCADA

Above the rated speed (Region 3), rotor speed regulation is achieved through collective blade pitching. This is implemented using a gain-scheduled proportional-integral-derivative (PID) controller acting on the speed error between the filtered generator speed and the rated generator speed [59]. The controller algorithm adjusts dynamically the pitch angle of the blades as the wind speed changes. For this work, a simple control routine written in Python was adopted from [56]. The control is made through the manipulation of the pitch angle of the blades as a function of the wind speed. The calibration of this routine was similarly derived from the SCADA data, from which the relationship between wind speed and blade pitch angle was derived. Figure 6.3 shows this relationship, where the function implemented in the numerical model was derived from the spatial distribution of the cloud of points obtained from the SCADA data.

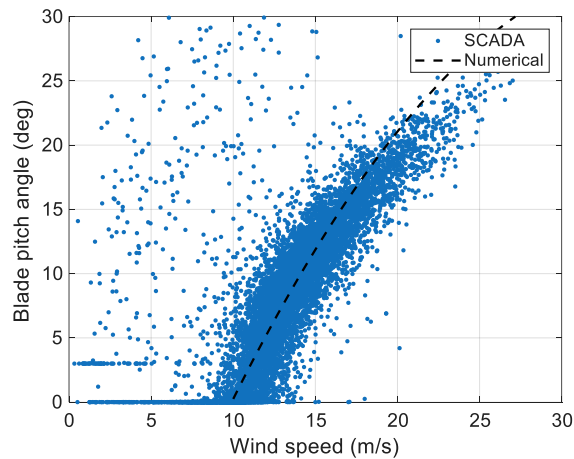


Figure 6.3 – Blade pitch angle function of the average wind speed

To verify that the assumed mass and stiffness distributions provide a realistic representation of the monitored WTG, the calibration used another complementary source of information: the modal behaviour of the tower identified from monitoring data presented in Chapter 3.3. The tower natural frequency provides a direct and sensitive indicator of the global bending stiffness and mass ratio and

therefore constrains the combined uncertainty associated with tower thickness estimation and rotor nacelle mass placement. In practical terms, the calibration was performed by iteratively adjusting (a) the tower sectional stiffness distribution (through thickness), and (b) the location and inertial properties of the concentrated rotor nacelle masses, until the numerical model reproduced the measured tower natural frequency within an acceptable tolerance. Structural damping was not included for simplicity, since the analysis focuses on the comparative influence of tower foundation interface conditions on the observed free vibration decay.

A further verification step assessed whether the numerical model reproduces the expected mean aerodynamic loading level by comparing the simulated bending moment at the tower base with an analytical estimate based on the thrust coefficients (c_T) provided by the wind turbine owner. Figure 6.4 presents the bending moment at the base of the tower as a function of the wind speed, provided by the SCADA system, comparing numerical results with a simplified analytical formulation. The thrust force is computed using the thrust coefficient and, assuming a standard air density ($\rho=1.225 \text{ kg/m}^3$). In the numerical model, the bending moments were obtained from a set of steady-state simulations, with one simulation performed for each wind speed, in which the inflow was prescribed as constant and laminar.

The bending moment at a generic tower section i is then evaluated as:

$$M_i = \frac{1}{2} \cdot \rho \cdot A_{rotor} \cdot v_1^2 \cdot c_T \cdot H_i - M_{ecc} \quad (6.1)$$

where H_i denotes the vertical distance between the rotor centre and the tower section at which M_i is evaluated, and M_{ecc} represents the additional moment arising from the eccentricity of the rotor-nacelle assembly relative to the tower centre of gravity.

In this analytical comparison, wind drag acting on the tower and second order contributions due to tower deflection were neglected, such that the analytical curve provides a reference load level rather than a complete aeroelastic solution.

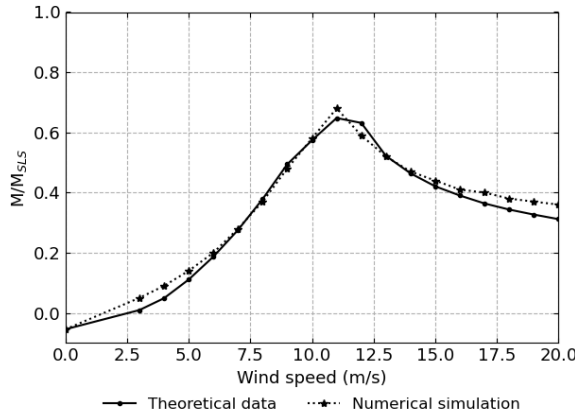


Figure 6.4 – Bending moment evolution with the wind speed variation. Comparison between the theoretical data and the numerical results

The agreement between the numerical and analytical trends indicates that the adopted aerodynamic and controller calibration yields realistic mean thrust levels across the operational wind speed range. This

verification supports the subsequent use of the calibrated WTG model as a baseline for assessing the influence of tower foundation interface conditions on the global response.

6.2.2 WIND TURBINE FOUNDATION INTERFACE

The foundation interface behaviour is represented using a macro-element approach, which aggregates the interface conditions between the foundation and the tower into a single conceptual element located at the connection between the foundation and the base of the tower. This unified macro-element effectively captures the nonlinear hysteretic response through a defined bending moment-rotation (M - θ) relationship at the base of the tower. This methodology is similar to the approach implemented in the study [19], which addressed offshore wind turbine foundations.

The findings reported in Chapter 5 highlight that the inclusion of gaps between the T-flange and the surrounding concrete significantly affects the rotational behaviour of the base of the tower, with substantial increase in rotation and inducing hysteretic behaviour, particularly when the gap is open. The experimental observations presented in Chapter 3.3, which are corroborated by the detailed numerical simulations in Chapter 5, confirm that under cyclic loading conditions, the tower base exhibits behaviour consistent with a nonlinear hysteretic response, which can be described in a simplified manner using the kinematic hardening model schematically illustrated in Figure 6.5 (b). This guided the development of the macro-element model for this research. The implementation was performed using a Python-based script integrated into the software Ashes enabling the simulation of the wind turbine behaviour under different operating conditions considering the effect of the gaps between the embedded T-flange and the concrete foundation. Figure 6.5 illustrates the foundation interface model, identifying the generic position of the macro-element (Figure 6.5 (a)) and providing a simplified M - θ diagram (Figure 6.5 (b)). This diagram visually represents the cyclic loading response, reflecting the nonlinear hysteretic behaviour observed experimentally and verified numerically.

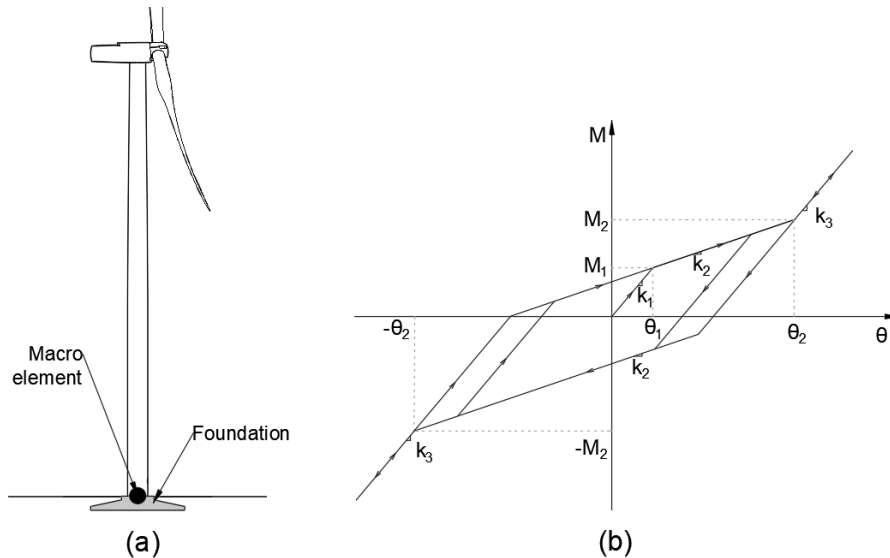


Figure 6.5 – Overview of the foundation model incorporated into the software Ashes

The kinematic-hardening model, implemented according to algorithm described in reference [61], retrieves the bending moment at the tower base when the rotation and its increment are given by the software Ashes. The Newton-Raphson iterative method is employed in the incremental formulation,

assuming that the following variables are given at load increment t^n : rotation increment provided by Ashes ($\Delta\theta$), bending moment (M^n) and back bending moment (α^n). The steps to compute the bending moment are as follows:

- **Elastic bending moment prediction:** Calculate the trial bending moment as follows: During the bending moment prediction, plasticity remains unchanged:

$$\Delta M^{tr} = k_1 \cdot \Delta\theta \quad (6.2)$$

$$M^{tr} = M^n + \Delta M^{tr} \quad (6.3)$$

The plastic predictor of the shifted bending moment is:

$$\eta^{tr} = M^{tr} - \alpha^{tr} \quad (6.4)$$

- **Check plasticity status:** Ensure the trial bending moment does not exceed the gap mobilization, meaning it must be lower than M_l . For that purpose, the trial plasticity function is defined as:

$$f^{tr} = |\eta^{tr}| - M_1^0 \quad (6.5)$$

If $f^{tr} < 0$, the gap has not been mobilized. Thus, $M^{n+1} = M^{tr}$ and the procedure moves to the next load increment.

- **Plastic corrector:** If $f^{tr} > 0$, then the gap is mobilized during this load increment, meaning that the friction between the TFS and the concrete is activated. In this situation, both the trial and back bending moments are updated as follows:

$$M^{n+1} = M^{tr} - \text{sgn}(\eta^{tr}) \cdot k_1 \cdot \Delta\theta_p \quad (6.6)$$

$$\alpha^{n+1} = \alpha^n + \text{sgn}(\eta^{tr}) \cdot k_a \cdot \Delta\theta_p \quad (6.7)$$

with $\Delta\theta_p$ being the rotation increment after gap occurs and k_a determined as follows:

$$k_a = \frac{k_1 \cdot k_2}{k_1 - k_2} \quad (6.8)$$

$\text{sgn}()$ is a sign function that takes the value ‘+1’ when its argument is positive and ‘-1’ when negative. This function is necessary because the procedure applies to both positive and negative bending moments, as schematically represented in Figure 6.5 (b).

- **Plastic consistency condition:** The unknown rotation increment in the gapping correction formulas (6.6) and (6.7) is determined from the gapping consistency condition:

$$f^{n+1} = |\eta^{n+1}| - M_1^0 = 0 \quad (6.9)$$

Expanding this in terms of rotation increment after gap is mobilized:

$$\begin{aligned} & |M^{tr} - \text{sgn}(\eta^{tr}) \cdot k_1 \cdot \Delta\theta_p - \alpha^{tr} - \text{sgn}(\eta^{tr}) \cdot k_a \cdot \Delta\theta_p| - M_1^0 = 0 \\ & |M^{tr} - \alpha^{tr}| - M_1^0 - (k_1 + k_a) \cdot \Delta\theta_p = 0 \end{aligned}$$

Thus, the rotation increment after gap mobilization is:

$$\Delta\theta_p = \frac{f^{tr}}{k_1 + k_a} \quad (6.10)$$

For kinematic hardening, the rotation increment after gap mobilization need not be stored as the load history is captured in the back bending moment.

Once the gap is closed, meaning for rotations higher than θ_2 and lower than $-\theta_2$ the entire foundation block is mobilized, and the stiffness of the system becomes k_3 .

Before the implementation of the Python scrip into the wind turbine model, a preliminary validation was carried out. This involved comparing the results from the script against the results from the numerical simulation of the foundation described in Chapter 5. Figure 6.6 presents the outcome of this comparison, considering a damaged foundation with 1 mm gap located at the top surface of the T-flange, a friction coefficient of 30° , and a cohesion of 80 kPa between the embedded steel tower and the surrounding concrete. The comparison with the numerical simulation of the foundation enabled the determination of the pre-required inputs in the Python script, namely the rotation θ_2 , bending moments M_1 and M_2 , and the stiffnesses k_1 , k_2 and k_3 .

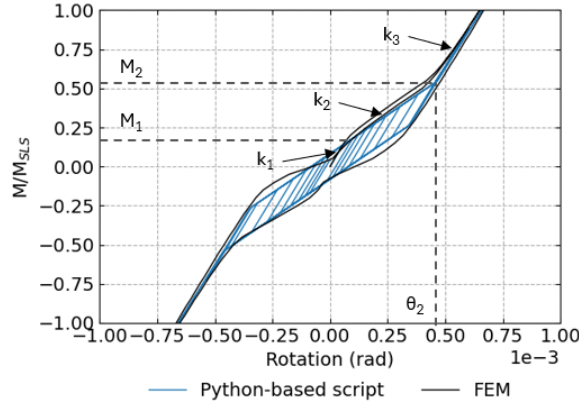


Figure 6.6 – Comparison of the simulation results from the model in Chapter 5 and the Python-based script, considering a damaged foundation with 1 mm gap located at the top surface of the T-flange, a friction coefficient of 30° , and a cohesion of 80 kPa between the embedded steel tower and the surrounding concrete.

6.2.3 COMPARISON WITH EXPERIMENTAL RESULTS

In this section, the results obtained from the experimental campaign conducted on WTG 8, as detailed in Chapter 3.3, are compared with the numerical model developed in the present study. The numerical simulations were performed using the aero-servo-elastic software Ashes. The main structural components, including the tower and blades, were modelled using FEM beam elements, while the aerodynamic loading was computed using Blade Element Momentum (BEM) theory with dynamic stall and tower shadow corrections. The simulations were carried out in the time domain, allowing transient effects, geometric nonlinearity, and large deflections to be captured.

The tower foundation connection was represented through an equivalent boundary condition at the tower base, replacing an idealised fixed support with a nonlinear moment rotation relationship governed by the kinematic hardening law described in Section 6.2.2. This formulation enables gap opening and closing and friction at the interface to be represented through a compact set of parameters. Consequently,

local interface degradation is translated into an amplitude-dependent rotational stiffness and an energy dissipation mechanism that directly influences the global dynamic response of the turbine. The calibrated set of parameters adopted for the comparison with the shutdown events is summarised in Table 6.1, where the friction angle and cohesion correspond to the parameters adopted in the analyses presented in Chapter 5 to represent the tower-foundation interface behaviour in the presence of gaps. The remaining parameters correspond to the kinematic hardening law implemented at the tower base in Ashes. These hardening parameters were obtained by curve fitting to the moment-rotation relationships derived from the simulations in Chapter 5.

Table 6.1 – Tower-foundation interface parameters and corresponding hardening parameters adjusted to the experimental results derived from the inclinometers for the shutdown events occurred in 24/10/2023 and 11/10/2024

Gap thickness (mm)	Friction angle (°)	Cohesion (kPa)	M_1/M_{SLS}	θ_2 (rad)	k_1 (N.m/rad)	k_2 (N.m/rad)	k_3 (N.m/rad)
1	45	80	0.08	3.90×10^{-04}	$7.50 \times 10^{+10}$	$4.20 \times 10^{+10}$	$1.43 \times 10^{+11}$

Calibration was performed by comparing simulated and measured response quantities associated with the tower base behaviour. The numerical model was adjusted to reproduce both the magnitude and the evolution in time of the response during the shutdown transient and the subsequent free oscillations, with emphasis on peak amplitudes, dominant oscillation frequency, and decay rate, which provide indicators of effective stiffness and damping. Agreement between numerical and experimental time histories was evaluated in terms of peak values immediately after shutdown, the dominant oscillation frequency, and the amplitude decay over successive cycles, while ensuring that the calibrated interface parameters remained physically consistent with the gap and friction mechanisms discussed in Chapter 5.

Prior to initiating the emergency shutdown, the wind turbine model was brought to a quasi-steady operating condition representative of rated operation. The time integration was performed using a fixed increment of 0.001 seconds, and output records were stored at 1000 Hz. This time step was selected to resolve the rapid load variations during the shutdown transient while maintaining numerical stability in the presence of nonlinear boundary conditions at the tower base. The simulation was initialised with the turbine operating at rated rotor and generator speed, with the controller active, such that aerodynamic and generator torques were in equilibrium and the mean rotor speed remained approximately constant. A run-in period of approximately 100 seconds was applied to allow initial transients to decay and to ensure that the response was governed by the turbulent inflow rather than numerical initialisation artefacts, followed by a further 150 seconds of normal operation prior to shutdown.

The inflow conditions adopted in the simulations were defined to replicate, as closely as possible, the atmospheric conditions recorded during the experimental shutdowns. SCADA data corresponding to the monitoring time windows were used to characterise the average wind speed at the hub height and the turbulence level, from which the mean wind speed and turbulence intensity for the numerical inflow were established. On this basis, the wind field was generated within Ashes using the Normal Turbulence Model (NTM) prescribed in IEC 61400-1, assuming an average wind speed at the hub height of 15 m/s, an IEC standard class 1 turbine, and high turbulence characteristics (class A). The corresponding turbulent inflow parameters adopted in the simulations are summarised in Table 6.2.

Table 6.2 – Wind conditions for the simulations

Wind regime	Wind turbine class	I_{ref}	$V_{\text{ref, mean}}$ (m/s)
Normal Turbulence Model (NTM)	I	0.16	15

After 150 seconds of simulation time under normal operation, an emergency shutdown was triggered and modelled as a forced pitch-to-feather manoeuvre, in which the blades were pitched collectively from 0° to 90° over 12 seconds. The pitch command was implemented as a prescribed time-varying collective trajectory through the Ashes control interface, representing a rapid reduction in aerodynamic loading and aerodynamic torque. After completion of the pitch-to-feather command, the simulation was continued to capture the free-decay oscillations of the tower. These post-shutdown oscillations were used to evaluate the effective stiffness and to support the calibration of the nonlinear moment rotation behaviour prescribed at the tower base.

1. Moment-rotation comparison

The comparison with the shutdown events is presented in Figure 6.7 to Figure 6.9. For the moment-rotation results (Figure 6.7 and Figure 6.8), the comparison includes the detailed nonlinear foundation model (FEM FND), the aero-servo-elastic wind turbine model incorporating the macro-element (FEM WTG), and the experimental measurements.

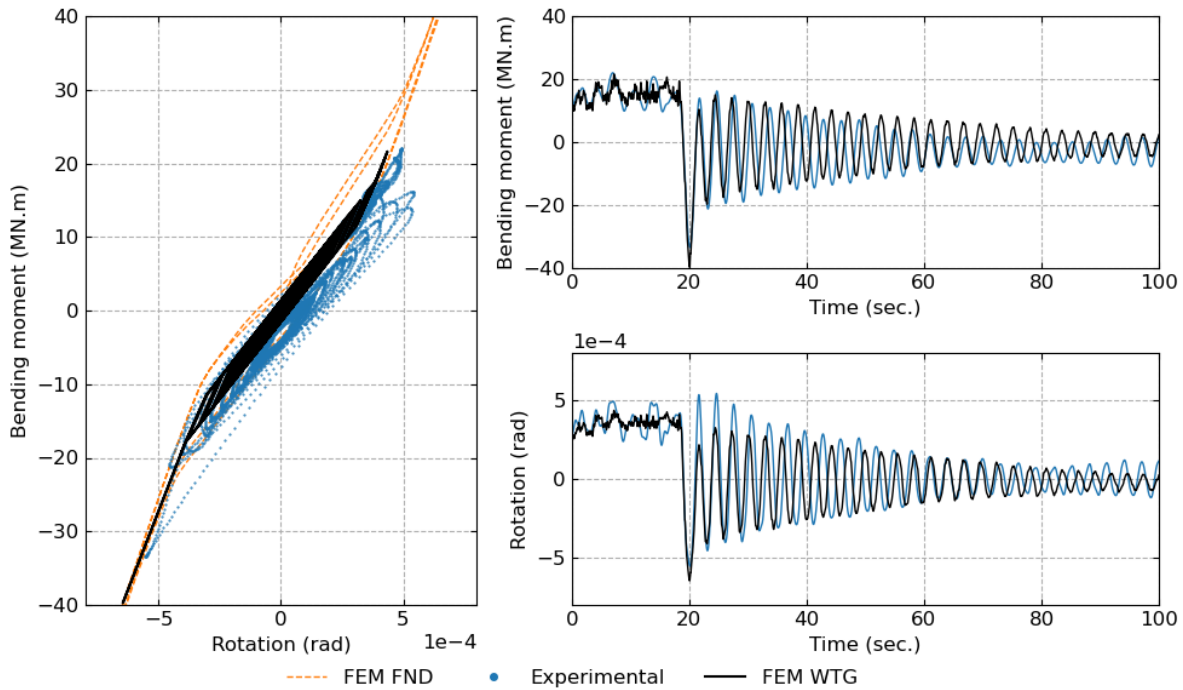


Figure 6.7 – Relationship between bending moment and rotation of the tower bottom section. Comparison between numerical simulations and experimental results (inclinometers from 24/10/2023)

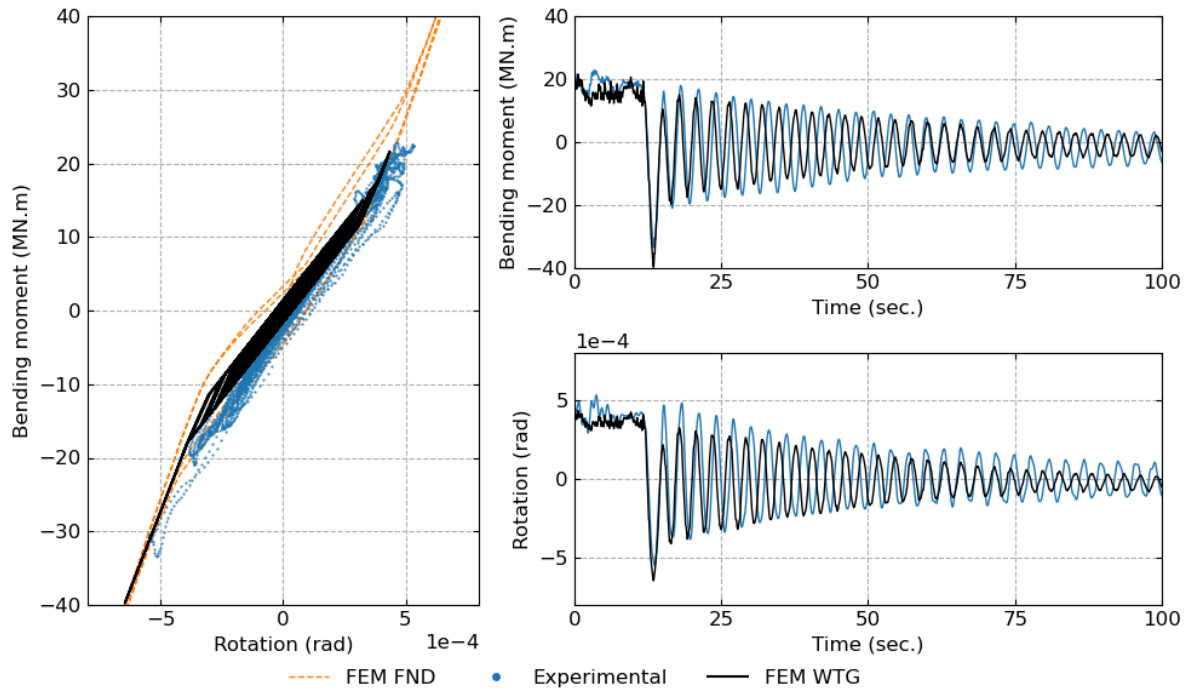


Figure 6.8 – Relationship between bending moment and rotation of the tower bottom section. Comparison between numerical simulations and experimental results (inclinometers from 11/10/2024)

The results from the shutdown event occurred on 24/10/2023 are shown in Figure 6.7. The left panel presents the relationship between the bending moment and rotation at the bottom section of the tower. The experimental data reveals a hysteretic behaviour, indicative of energy dissipation at the tower-foundation interface caused by friction between the steel and the concrete under cyclic loading induced by the shutdown event. The numerical simulations, including both the foundation model (FEM FND) and the complete wind turbine model (FEM WTG), closely replicate this hysteretic response. In particular, the initial stiffness, the shape, and general size of the hysteresis loops observed numerically align well with the experimental data. However, slight deviations are noticeable, primarily at larger positive rotations, where the numerical response is marginally stiffer, evidenced by a slightly narrower loop shape on the positive side.

The right panels illustrate the time histories of the bending moment and rotation throughout the shutdown event. Both numerical and experimental curves show good initial alignment in terms of amplitude, phase, and frequency. After the rotor stops, both experimental and numerical signals display damping characteristics. However, a discrepancy emerges in the decay pattern: the numerical simulation exhibits a less pronounced amplitude reduction, particularly for positive rotation values. This difference suggests that the numerical model tends towards a slightly more rigid response during the unloading phase.

Figure 6.8 presents the results corresponding to the shutdown event of 11/10/202. The moment-rotation diagram presented on the left panel reveals a similar hysteretic response to that observed in the 2023 event, confirming the repeatability and robustness of the nonlinear behaviour. Again, both numerical simulations and experimental data reveal hysteretic loops indicative of energy dissipation due to friction between the steel and the concrete. The numerical models (FEM FND and FEM WTG) replicate the experimental loops accurately, capturing the general shape and overall stiffness evolution across loading

and unloading cycles. Slight deviations persist, particularly at rotational extremes, where the numerical models continue to display a marginally stiffer response relative to the experimental data.

The time-history plots on the right-hand side further reinforce the consistency between simulation and measurement. Prior to the rotor shutdown, the numerical predictions for the bending moment and rotation align closely with the experimental results. After shutdown, both datasets exhibit a clear decay of oscillations, although, as already observed in the data from 24/10/2023, the numerical simulation exhibits a less pronounced amplitude reduction, particularly for positive rotation values. Despite these small deviations, the overall agreement remains strong.

2. Natural frequency comparison

The frequency domain comparison presented in Figure 6.9 provides an additional assessment of the calibrated stiffness through the identified natural frequencies. These were extracted from the measured accelerations by estimating the PSD using the FFT. The continuous time series were divided into segments of 80 seconds with 75% overlap between consecutive segments, resulting in a frequency resolution of approximately $\Delta f = 1/80 \approx 0.0125 \text{ Hz}$. This allows for a minimum detectable variation of about 3.75% regarding the natural frequencies.

Before the shutdown, the experimental spectra indicate a first tower mode at $f_1 = 0.34 \text{ Hz}$ and a second mode at $f_2 = 2.84 \text{ Hz}$. The numerical model estimates $f_1 = 0.35 \text{ Hz}$ and $f_2 = 2.86 \text{ Hz}$. A good agreement is observed, with the first mode falling within the frequency resolution limits and the second mode showing only a marginal deviation.

During shutdown, the second mode is not clearly identifiable in either the experimental or numerical spectra, indicating, possibly that it is not sufficiently excited. The comparison during shutdown is therefore restricted to the first mode. The measured frequency remains at $f_1 = 0.34 \text{ Hz}$, while the numerical model maintains $f_1 = 0.35 \text{ Hz}$.

Overall, the combined agreement in moment-rotation behaviour, time-domain decay, and modal frequency content validates the adopted macro-element representation of the tower-foundation connection and the calibration of the kinematic hardening parameters for the shutdown conditions considered.

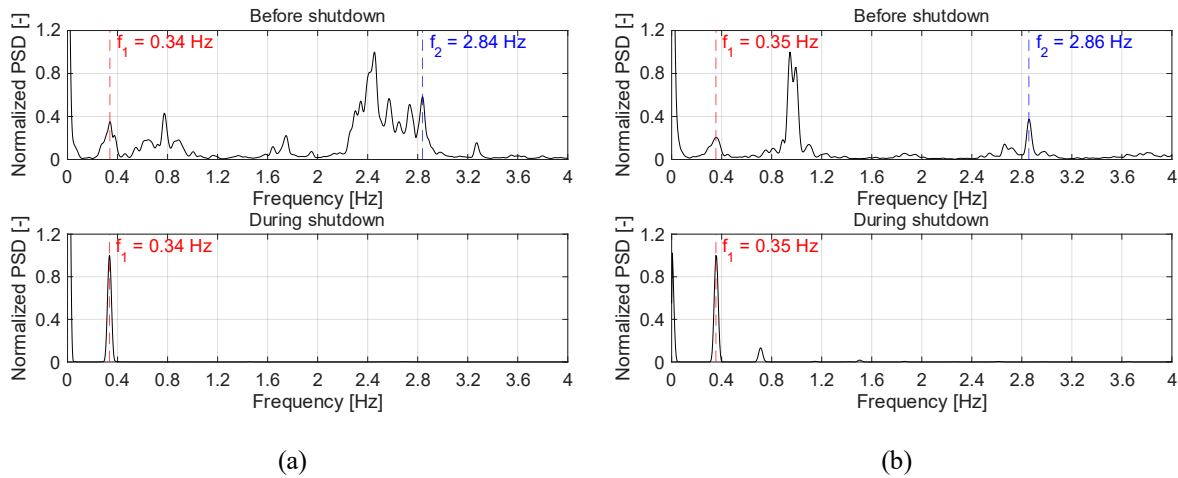


Figure 6.9 – Normalised power spectral density (PSD) estimates of tower acceleration for time windows immediately before shutdown (top row) and during shutdown (bottom row): (a) spectra derived from accelerometer measurements for the 2024 shutdown event; (b) spectra obtained from the corresponding

numerical simulation. In each case, the spectral ordinates are normalised by the maximum value of the respective spectrum. Vertical dashed lines indicate the identified first and second natural frequencies, f_1 and f_2

6.2.4 PARAMETRIC STUDY

With the numerical model of the WTG established, a preliminary parametric study was undertaken to understand the impact of gaps and friction between the TFS and the surrounding concrete on the dynamics of the wind turbine. In particular, the analysis focused on evaluating the first and second fore-aft eigenfrequencies, as well as the horizontal displacements at the top of the tower.

For each simulation, the foundation was modelled with a constant stiffness that varied depending on the assumed friction angle, as summarised in Table 6.3. To isolate the effect of friction, the analyses were performed under the assumption of an open gap, with the foundation stiffness represented exclusively by the parameter k_2 . This approach enables the investigation of the wind turbine dynamics across a range of boundary conditions, from lower to upper bounds. The lower bound is defined by the presence of a gap without friction at the interface, representing a system with minimal rotational restraint. In contrast, the upper bound corresponds to an undamaged foundation. This methodology facilitates the identification of the dynamic response envelope of the structure under extreme foundation interface conditions.

Table 6.3 – Foundation stiffness according to the friction angle of the interface

Friction angle (°)	k (N.m/rad)
No GAP*	9.24×10^{10}
0**	1.70×10^{10}
30**	3.50×10^{10}
45**	4.50×10^{10}

* constant stiffness (k) equal to k_1 represented in Figure 6.5 (b)

** constant stiffness (k) equal to k_2 represented in Figure 6.5 (b)

Figure 6.10 illustrates the normalized frequencies of the first and second vibration modes for both an undamaged foundation and a damaged foundation subjected to varying friction conditions between the TFS and the concrete interface. The considered friction conditions include a frictionless interface, as well as friction angles of 30° and 45° . These frequencies are normalized with respect to the simulation results of the undamaged foundation, providing a clear comparison of how friction influences the dynamic behaviour of the system.

Figure 6.11 presents a similar comparison but focuses on the normalized horizontal displacements at the top section of the tower under the same friction conditions. This comparison highlights the impact of friction on the lateral deflections of the tower.

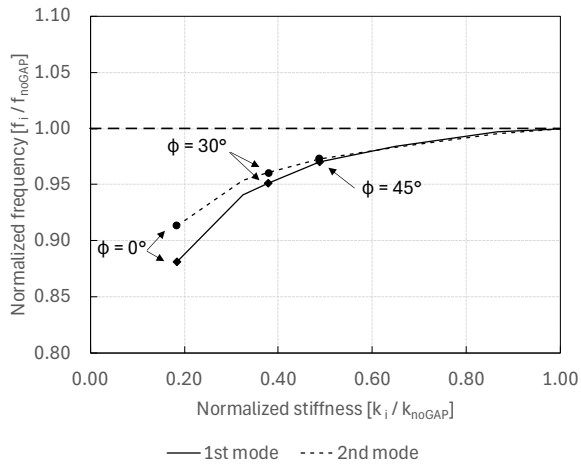


Figure 6.10 – Evaluation of the natural frequencies (1st and 2nd modes) of the WTG as a function of the foundation interface stiffness

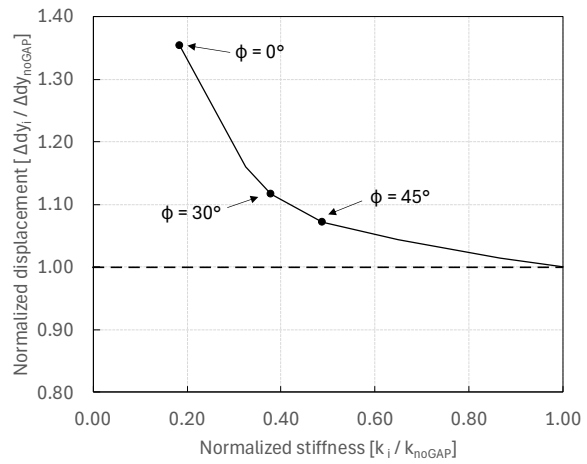


Figure 6.11 – Evaluation of the horizontal displacement at the tower top as a function of the foundation interface stiffness

The results shown in Figure 6.10 indicate that, for a damaged foundation, the 1st natural frequency exhibits a maximum deviation of approximately 12% relative to the undamaged foundation. For the 2nd natural frequency, the maximum deviation is approximately 9%. Regarding the horizontal displacement at the top of the tower, Figure 6.11 reveals that the horizontal displacement of the damaged foundation reaches a maximum of 35% when compared to the undamaged foundation.

A parametric study was subsequently conducted with the primary objective of assessing the influence of three critical parameters: gaps width, friction coefficient, and cohesion, on the structural behaviour of the WTG. These parameters were recognised in the previous chapter as significant contributors to the structural response. Each parameter was systematically varied within realistic ranges to evaluate their impact on the performance of the WTG. The simulations were carried out using the software Ashes, incorporating the previously described kinematic hardening law, considering the scenarios summarised in Table 6.4.

Table 6.4 – Tower-foundation interface parameters and corresponding hardening parameters

	Gap thickness (mm)	Friction angle (°)	Cohesion (kPa)	M_1/M_{SLS}	θ_2 (rad)	k_1 (N.m/rad)	k_2 (N.m/rad)	k_3 (N.m/rad)
Scenario 1	0	0	0	N/A	N/A	9.24×10^{10}	N/A	N/A
Scenario 2	1	0	0	0.08	3.46×10^{-4}	8.00×10^{10}	1.70×10^{10}	8.00×10^{10}
Scenario 3	1	30	80	0.08	3.70×10^{-4}	8.00×10^{10}	3.50×10^{10}	1.50×10^{11}
Scenario 4	1	30	120	0.12	3.70×10^{-4}	8.00×10^{10}	3.50×10^{10}	1.50×10^{11}

The wind conditions used for the simulation are summarized in Table 6.5.

Table 6.5 – Wind conditions adopted in the simulation

Wind regime	Wind turbine class	I_{ref}	$V_{ref, mean}$ (m/s)
Normal Turbulent Model (NTM)	I	0.16	10

To evaluate the response of the WTG under the four scenarios presented in Table 6.4, the turbine was initially operated at its rated speed. Subsequently, it was subjected to a forced emergency shutdown by pitching the blades from its initial position (0°) to the final position (90°) in 12 seconds. This rapid adjustment of the blade pitch angle induces a sudden change in the aerodynamic forces, leading to extreme oscillations of the WTG. These oscillations are significant as they may expose different aspects of the dynamic behaviour of the wind turbine, particularly in relation to the tower-foundation interaction.

The parametric analyses presented in Figure 6.12 to Figure 6.14 clarify the distinct roles of gap formation and interface shear strength on the global dynamic response of the wind turbine. The results show that the gap primarily governs the effective rotational stiffness at the tower base, whereas friction and cohesion primarily govern the energy dissipation mechanisms during free vibration.

Figure 6.12 compares the reference configuration without gaps with a damaged configuration characterised by a 1 mm gap and a cohesionless and frictionless interface. The introduction of the gap produces a clear increase in tower base rotation for comparable normalised bending moment levels, as shown by the markedly larger rotations in both the moment-rotation relationship and the time histories. This increased flexibility is also reflected in the free-vibration response after the shutdown, where the oscillation period becomes longer in the gap case, indicating a reduction of the effective stiffness and therefore a lower natural frequency. Regarding bending moment demand, the pre-shutdown response remains essentially unchanged between the two scenarios. After shutdown, the differences in peak moments are small but systematic assessment of the extrema indicates that the gap scenario exhibits slightly larger absolute peaks during the subsequent oscillations. The comparison of positive and negative peaks following sign inversion suggests no major asymmetry, with the dominant effect being a modest increase in the absolute maximum rather than a consistent bias towards one sign. Overall, the gap case is characterised by substantially larger rotations, a longer oscillation period, and similarly low decay rates, indicating limited changes in damping in both configurations.

Figure 6.13 isolates the influence of friction and cohesion by comparing the frictionless and cohesionless interface with an interface characterised by cohesion of 80 kPa and friction angle of 30° , while maintaining a constant gap thickness of 1 mm. The primary change is observed in the decay of the free-vibration response. The moment-rotation plot exhibits a pronounced hysteretic loop when friction and cohesion are included, indicating energy dissipation through interface shear mechanisms during gap opening and closing. This behaviour is reflected in the time histories, where both rotation and bending moment oscillations decay significantly faster than in the frictionless case. In contrast, the oscillation period changes only marginally, and the peak bending moment levels remain very similar, including the maximum absolute moment after sign inversion, which does not exhibit a consistent increase due to friction and cohesion. Therefore, friction and cohesion act mainly on damping, while their influence on extreme moment demand and natural period is comparatively limited for the configurations considered.

Figure 6.14 evaluates the effect of increasing cohesion from 80 kPa to 120 kPa for a constant gap thickness of 1 mm and a friction angle of 30° . The increase in cohesion intensifies the hysteretic behaviour in the moment-rotation response and further accelerates the decay of free vibrations, particularly in the rotation time history. The bending moment histories remain closely comparable, with only minor differences in peak values and no clear trend of increasing maximum absolute moment with cohesion. The dominant effect of cohesion is therefore an enhancement of damping and a reduction of rotation amplitudes, rather than a significant modification of peak bending moments or oscillation period.

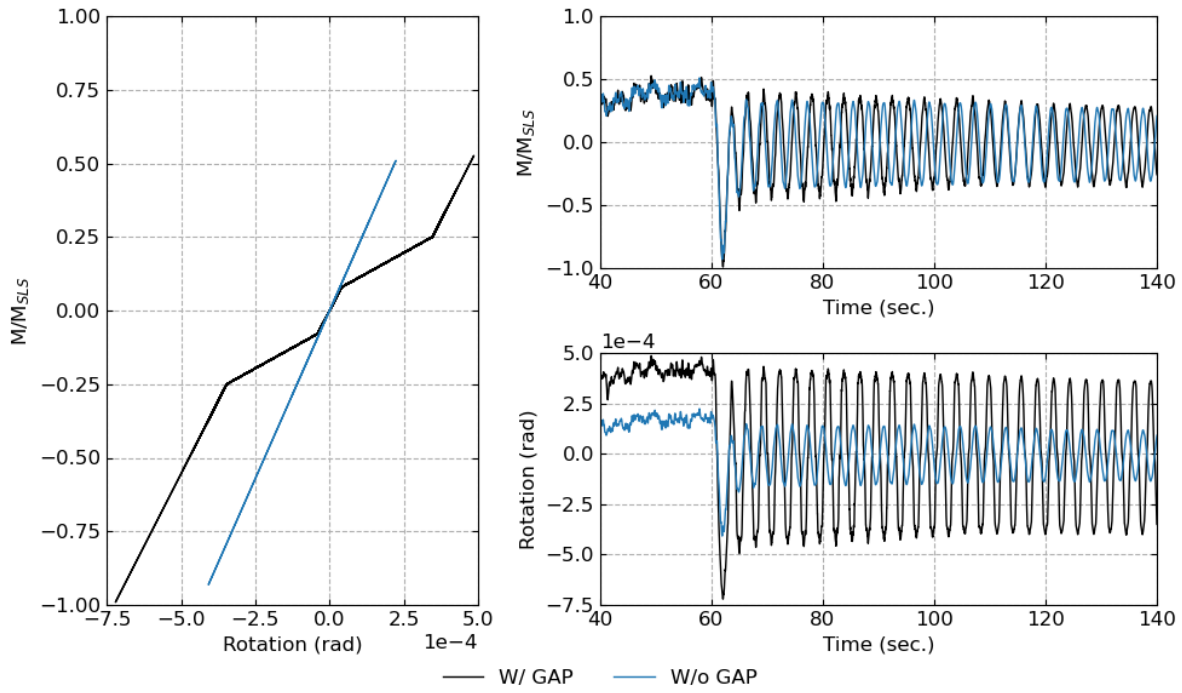


Figure 6.12 – Comparison between Scenario 1 (no gap) and Scenario 2 (1 mm gap with frictionless and cohesionless interface): Moment–rotation relationships (left plot) and time histories of bending moment and rotation at the tower base during the shutdown (right plots)

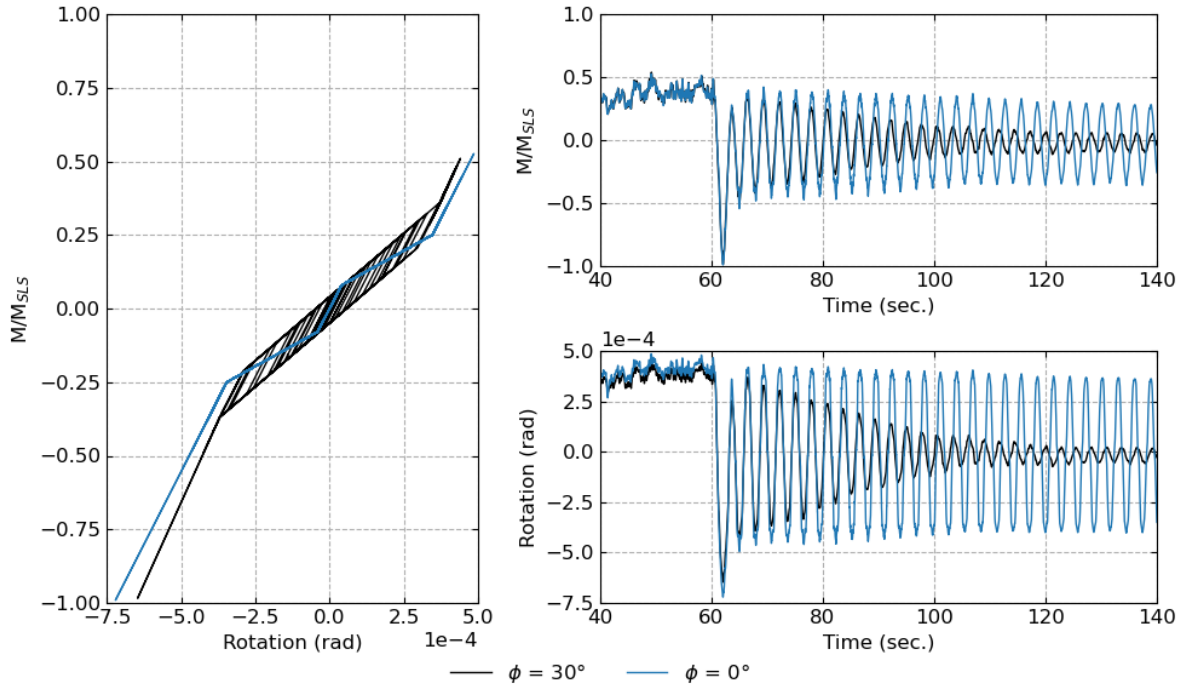


Figure 6.13 – Comparison between Scenario 2 (1 mm gap with frictionless and cohesionless interface) and Scenario 3 (1 mm gap with interface cohesion of 80 kPa and friction angle of 30°): Moment–rotation relationships (left plot) and time histories of bending moment and rotation at the tower base during the shutdown (right plots)

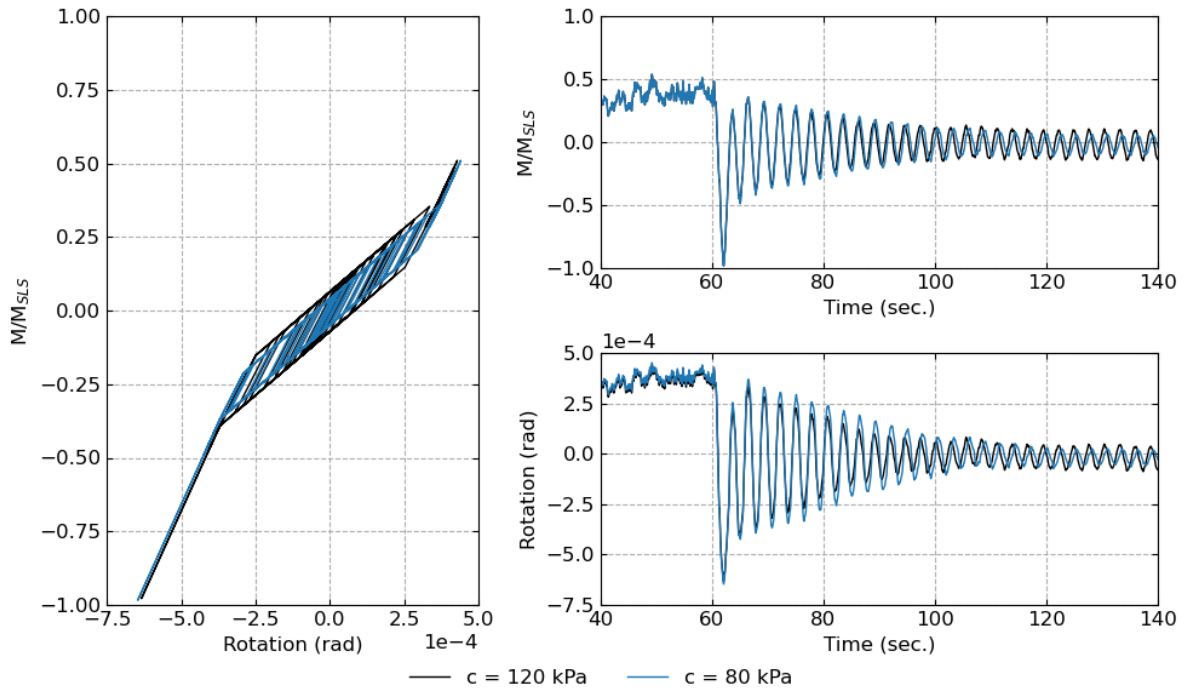


Figure 6.14 – Comparison between Scenario 3 (1 mm gap with interface cohesion of 80 kPa and friction angle of 30°) and Scenario 4 (1 mm gap with interface cohesion of 120 kPa and friction angle of 30°): Moment–rotation relationships (left plot) and time histories of bending moment and rotation at the tower base during the shutdown (right plots)

Figure 6.15 extends the comparison between the simulation with and without gap by examining the tower-top response during the emergency shutdown. In terms of horizontal displacement (Figure 6.15 (a)), the peak response immediately after shutdown is of similar magnitude in both configurations, indicating that the maximum global lateral displacement is not strongly governed by the presence of the gap within the analysed event. In contrast, the acceleration time history (Figure 6.15 (b)) shows higher peak accelerations for the case without a gap, which is consistent with a stiffer system response. In addition, the oscillation frequency decreases when a gap is introduced, reflecting the reduction in effective rotational stiffness at the tower foundation interface associated with gap opening and closing.

Figure 6.16 isolates the effect of interface friction for the foundation with gap by comparing $\phi = 30^\circ$ (with friction) and $\phi = 0^\circ$ (frictionless). The frictional case exhibits a clearer decay of oscillation amplitude in both displacement and acceleration, indicating increased energy dissipation. By contrast, the frictionless simulation maintains larger sustained oscillations over time, consistent with a reduced capacity for hysteretic dissipation at the interface. This behaviour is consistent with the hysteretic mechanisms discussed previously for the base response, in which friction changes the effective stiffness and the associated energy dissipation during cyclic contact.

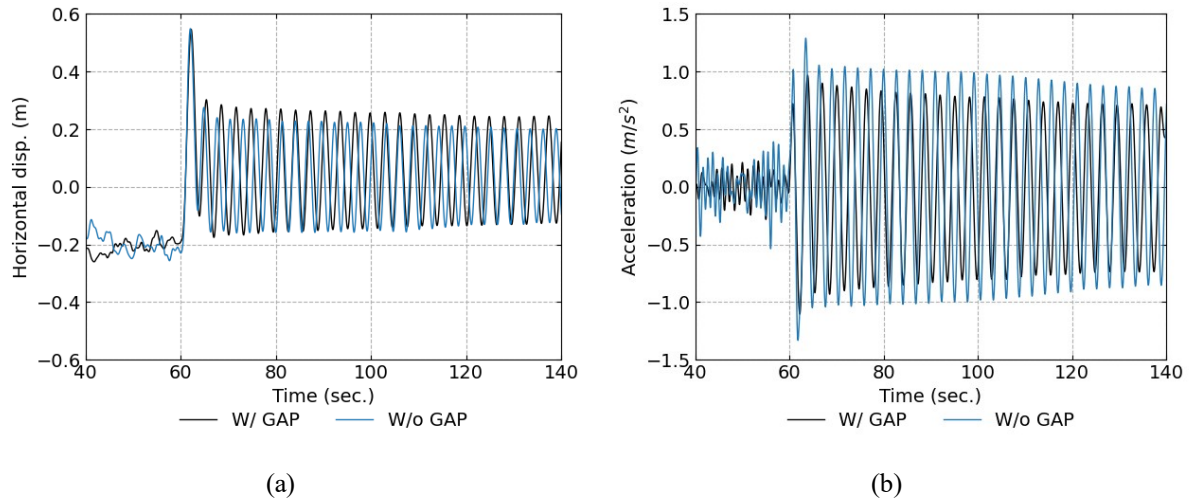


Figure 6.15 – Numerical time histories at tower top during an emergency shutdown, comparing simulations with and without an interface gap: (a) horizontal displacement and (b) horizontal acceleration. Results are shown for the model with gap (W/ GAP) and without gap (W/o GAP)

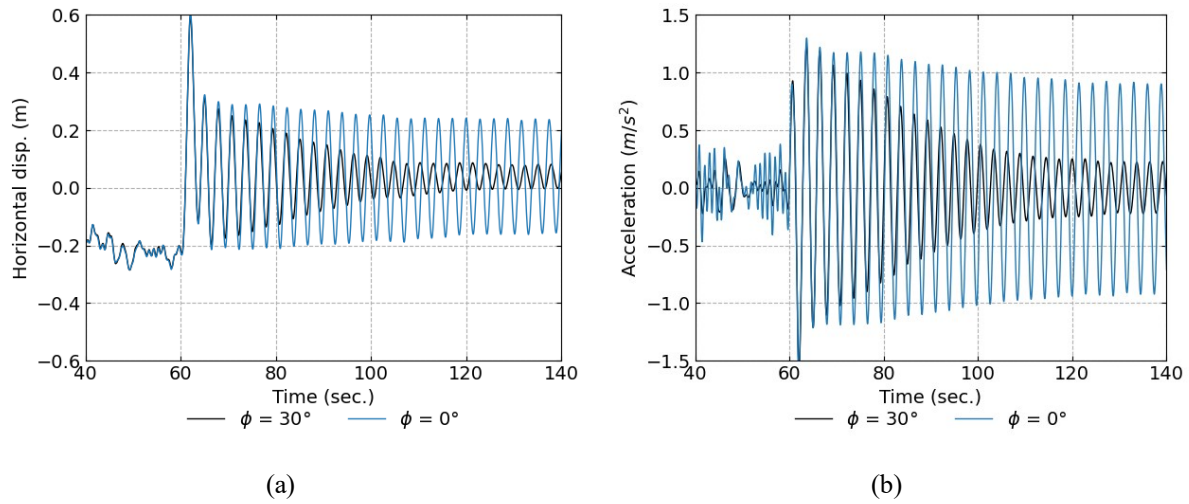


Figure 6.16 – Numerical time histories at tower top during an emergency shutdown for the model including an interface gap, comparing cases with and without interface friction: (a) horizontal displacement and (b) horizontal acceleration. Results are shown for $\phi = 30^\circ$ (with friction) and $\phi = 0^\circ$ (frictionless interface)

6.3 GAP EFFECT IN WTG BEHAVIOUR UNDER OPERATION

This section investigates the influence of damaged foundations, specifically in the form of circumferential gaps between the embedded flange and the surrounding concrete, on the operational behaviour of wind turbines.

Following the calibration of the numerical model, the subsequent step involves simulating the influence of such gaps on the dynamic and structural response of the system. To this end, three numerical simulations were conducted, employing the same wind turbine configuration as that used in the analysis presented in previously.

In the first simulation, the wind turbine operates on an undamaged foundation. In the second and third simulations, the wind turbine operates on a damaged foundation, characterised by a circumferential gap of 1 mm and 2 mm constant thickness, respectively, located between the T-flange and the embedded concrete. In all cases, the interaction between the steel and the concrete is represented using an interface model defined by a friction angle of 30° and a cohesion value of 80 kPa. The interface parameters adopted in the numerical analyses are summarised in Table 6.6. The selection of the gap thicknesses are based on the conclusions from Chapter 5.

Table 6.6 – Tower-foundation interface parameters and corresponding hardening parameters used in the simulations

	Gap thickness (mm)	Friction angle ($^\circ$)	Cohesion (kPa)	M_1/M_{SLS}	θ_2 (rad)	k_1 (N.m/rad)	k_2 (N.m/rad)	k_3 (N.m/rad)
Scenario 1	0	30	80	N/A	N/A	9.24×10^{10}	N/A	N/A
Scenario 2	1	30	80	0.08	3.70×10^{-4}	8.00×10^{10}	3.50×10^{10}	1.50×10^{11}
Scenario 3	2	30	80	0.08	5.80×10^{-4}	8.00×10^{10}	3.50×10^{10}	1.50×10^{11}

The simulations were performed considering the same turbulent wind model with the wind conditions summarized in Table 6.5.

To assess the influence of the gaps on the operational behaviour of the wind turbine, the following response parameters were examined:

- Stress resultants at the base of the tower
- Tower accelerations
- Horizontal displacements at the top of the tower
- Natural frequencies

By comparing the stress resultants with the progression of the damage in the foundation, it is possible to assess the extent to which the latter influences the former. Similarly, the accelerations measured at the top of the tower enable the evaluation of the impact of the gaps in the tower oscillations. Controlling the tower oscillations is essential to prevent malfunctioning of the mechanical components such as the gearbox and drivetrain, and to limit the accumulation of fatigue damage in the blades and other structural components under sustained cyclic loading. The limitation of the horizontal displacements at the top of the tower reduces the risk of failure of the tower internal components, such as stairs, platforms and elevator. Finally, monitoring natural frequencies is required to identify stiffness changes induced by the interface condition and to mitigate the risk of resonance, particularly in the vicinity of rotor harmonics and other operational excitation frequencies.

6.3.1 BENDING MOMENT AT THE TOWER BASE

Figure 6.17 presents the structural response at the base of the tower by illustrating the relationship between bending moment and rotation for three foundation conditions: an undamaged case (scenario 1) and damaged cases with circumferential gaps of 1 mm and 2 mm (scenario 2 and 3, respectively).

The left-hand plot in Figure 6.17 shows the moment-rotation relationship under operational loading. For the undamaged foundation, full contact between the T-flange and the surrounding concrete is maintained

throughout the analysed range, resulting in an essentially linear elastic response with high rotational stiffness. In contrast, the presence of a circumferential gap (1 mm and 2 mm) produces a pronounced reduction in the initial rotational stiffness, as evidenced by the flatter slope at small rotations. In the damaged cases, the response is characterised by an initial reduced stiffness phase associated with partial or absent contact, followed by a marked stiffness increase once the T-flange reestablishes contact with the concrete. This transition defines a bilinear response, with a floating regime at low moments and an increased stiffness regime after contact is recovered.

In particular, for the 1 mm gap case the tower transitions to full contact at a bending moment of approximately 15 MN·m, corresponding to $M/M_{SLs} = 0.37$. For the larger gap, recontact occurs at a higher bending moment and therefore at a later stage of the loading history, implying that the tower remains in the floating regime for a longer portion of the operational cycle. This delayed recontact explains the progressively larger rotations observed as the gap width increases, even though the applied bending moments are comparable.

The bending moment time history (top right plot) remains practically unchanged across the different cases, supporting the conclusion that the gaps do not materially affect the overall magnitude of the bending moment at the tower base under standard operating conditions. By contrast, the bottom right plot highlights a clear increase in tower base rotation with increasing gap width. The rotation increases progressively from the undamaged condition to the 2 mm gap scenario, confirming that the circumferential gap reduces the effective rotational restraint at the base and allows larger angular displacements for similar bending moment demand.

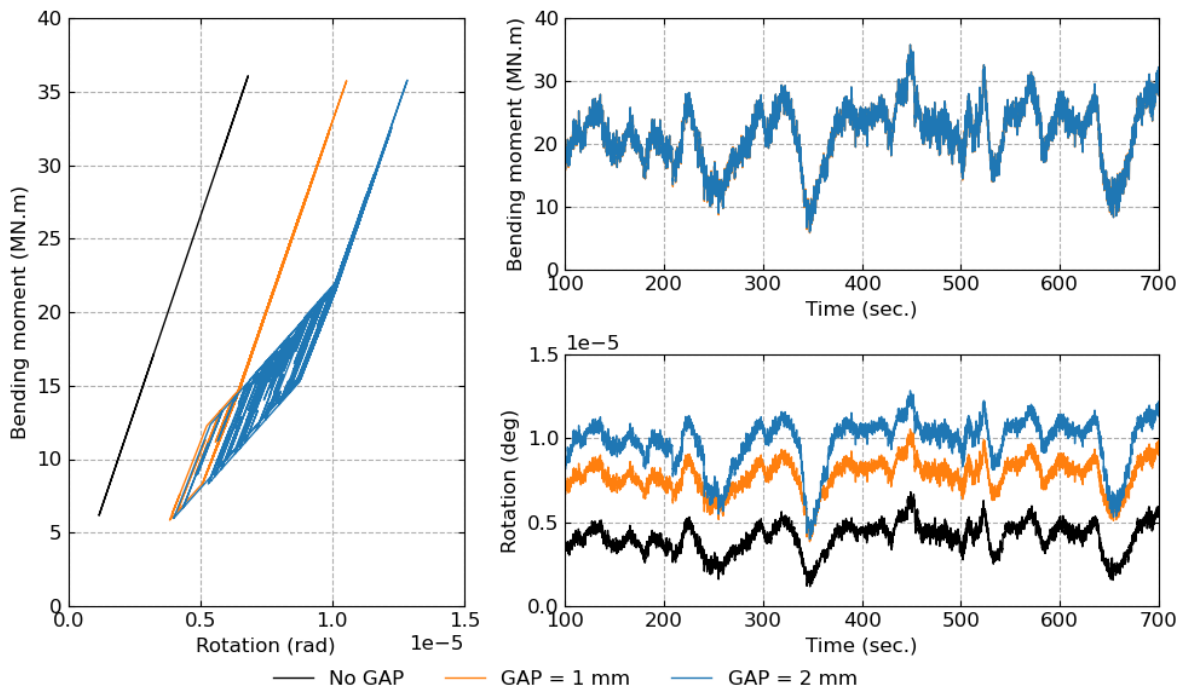


Figure 6.17 – Relationship between bending moment and rotation of the tower bottom section. Comparison between undamaged (no gaps) and damaged foundations (gaps with 1 mm and 2 mm)

6.3.2 TOWER ACCELERATIONS

The magnitude of the oscillation of the tower is quantified by computing the Root Mean Square (RMS) value of the acceleration signal over a specified time window. This method is particularly useful to identify potential variations in the dynamics of the tower resulting from the presence of gaps in the foundation.

Figure 6.18 presents the RMS values of the accelerations measured at the top of the tower, computed over a 60 seconds window for a total simulation duration of 10 minutes. The comparison includes both the undamaged and damaged foundations with 1 mm and 2 mm circumferential gap with an interface friction angle of 30° and a cohesion of 80 kPa.

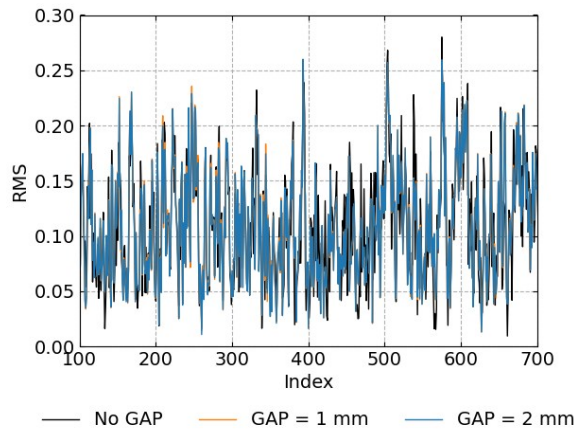


Figure 6.18 – RMS of the accelerations at the tower top section. Comparison between undamaged (no gaps) and damaged foundations (gaps with 1 mm and 2 mm)

Over the entire 10-minute simulation period, the RMS acceleration for the undamaged foundation is 0.125 m/s^2 , while for the damaged foundation with gap of 1 mm and 2 mm it increases slightly to 0.128 m/s^2 and 0.127 m/s^2 , respectively. This represents a deviation of 1.2 % to 1.9% in the magnitude of the oscillation due to the foundation damage.

Despite this minor increase, the time series indicated in Figure 6.18 reveal no systematic amplification of the magnitude of the oscillation due to the presence of the gap. Although localised fluctuations are observed in both cases, the overall amplitude and distribution of the RMS values remain comparable. This indicates that, under the examined operational conditions, the foundation damage considered does not lead to a significant increase in the tower vibration levels.

6.3.3 HORIZONTAL DISPLACEMENTS

To further evaluate the influence of foundation damage on tower deformation, the horizontal displacements at the tower top were analysed in terms of their mean value. The mean displacement was computed using 60 seconds sliding windows with 85% overlap over the 10-minute simulation records, providing a measure of the quasi-static lateral offset associated with sustained aerodynamic thrust while reducing short-term fluctuations.

Figure 6.19 presents the resulting window-mean displacement histories for the undamaged foundation and for the damaged foundations with circumferential gaps of 1 mm and 2 mm. The three cases exhibit

a very similar temporal evolution, indicating that the low-frequency variations of the mean response are primarily governed by the inflow and operational loading. Nevertheless, the damaged interface cases show a systematic shift towards larger absolute mean displacements, with the 2 mm gap producing the most pronounced negative offsets. This trend is consistent with the reduction of the stiffness at the tower-foundation interface when full contact in the embedded region is no longer maintained.

Overall, the results indicate that, under the simulated operational conditions, the introduction of a circumferential gap primarily produces a small change in the quasi-static displacement offset, rather than a qualitative modification of the displacement trend or evidence of unstable behaviour. This behaviour is consistent with a localised stiffness reduction associated with partial loss of contact, without implying a fundamental change in the overall dynamic characteristics of the tower-foundation system.

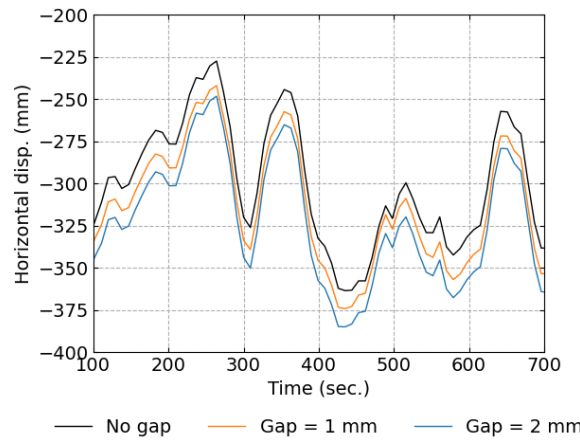


Figure 6.19 – Horizontal displacements at the tower top section. Comparison between damaged and undamaged foundation

6.3.4 TOWER NATURAL FREQUENCIES

In addition to displacement and acceleration responses, the influence of the gaps on the natural frequencies of the tower was examined. Controlling the natural frequencies of the tower is essential to ensure that they remain sufficiently separated from excitation frequencies induced by operational loads, thereby mitigating the risk of resonance phenomena.

Figure 6.20 compares normalised PSD estimates of the tower acceleration for 10-minute time series under the three interface conditions described previously. The PSDs were estimated using an FFT based Welch procedure, in which each 10-minute record was divided into segments of 60 seconds with 50% overlap and the resulting spectra were averaged. This configuration provides an effective frequency resolution of approximately $\Delta f = 1/60 \approx 0.0167$ Hz. For clarity, the spectra were not normalised by the absolute global maximum of the PSD. Instead, each spectrum was normalised by the amplitude of the dominant peak in the vicinity of the first tower mode, located on the right-hand side of the low frequency range (near f_1). This choice avoids scaling the spectra by a much larger low frequency peak on the left-hand side of the plot, which would otherwise compress the ordinate and obscure secondary peaks relevant for modal identification.

Across all three interface conditions, the identified natural frequencies remain unchanged at $f_1 = 0.34$ Hz and $f_2 = 2.85$ Hz, corresponding to the first and second tower modes, respectively. This indicates

that, within the analysed operational window, the presence of a circumferential gap up to 2 mm does not produce a measurable change in the global stiffness governing these modes. Two factors may explain the absence of detectable frequency shifts. First, any gap-induced variation smaller than the adopted spectral resolution $\Delta f \approx 0.0167$ would not be reliably distinguishable in the PSD peak locations. Second, under standard operating conditions the tower base connection is expected to maintain intermittent or near-continuous contact between the T-flange and the surrounding concrete. As a result, the effective stiffness governing the global bending modes is expected to remain close to that of the undamaged configuration for much of the time history, thereby limiting the influence of the gap on the identified modal frequencies. Consequently, the considered gap widths are not expected to have a relevant operational impact, provided that they do not lead to a critical proximity between the tower modal frequencies and rotor harmonics.

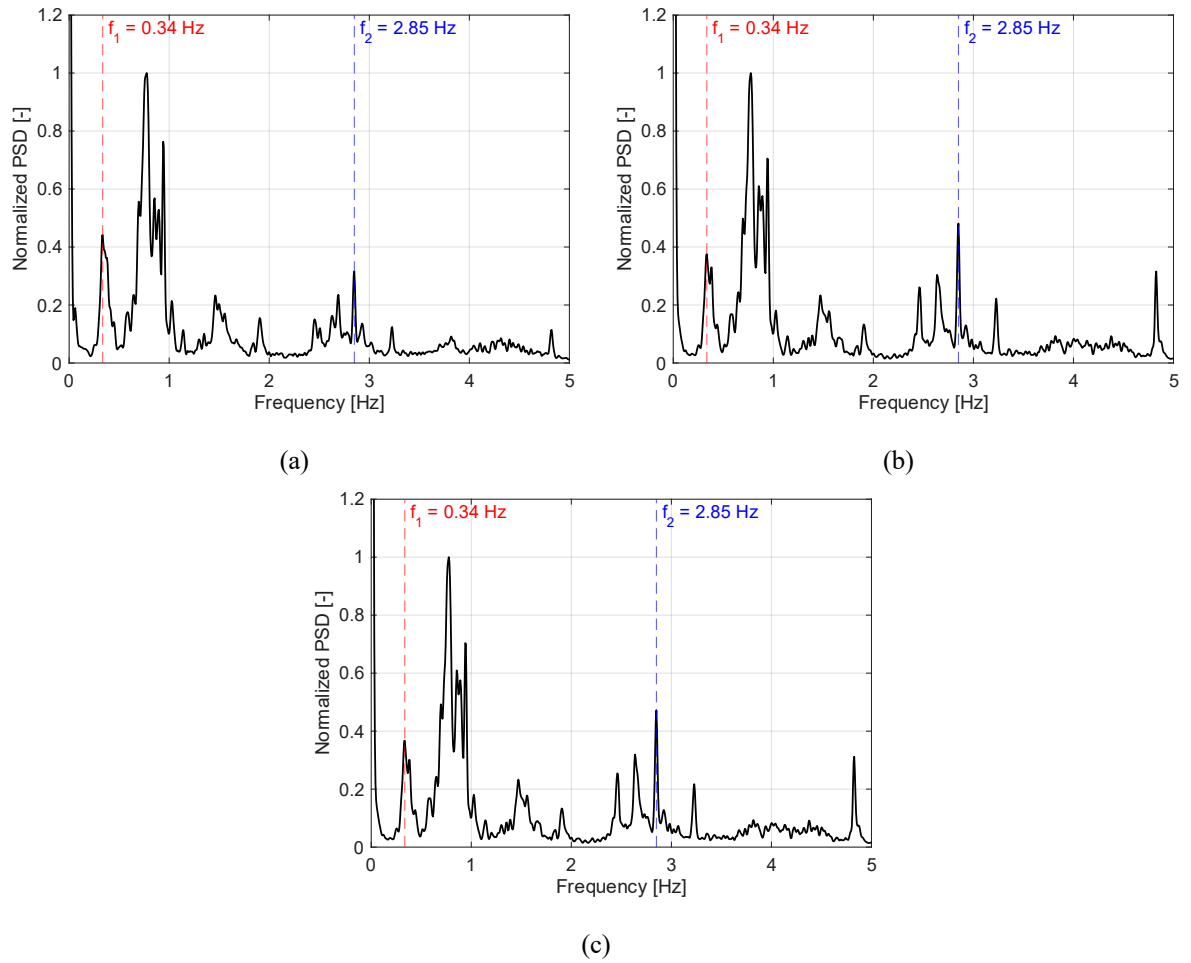


Figure 6.20 – Normalized power spectral density (PSD) estimates of tower acceleration for 10 minute time windows: (a) undamaged foundation; (b) foundation with 1 mm circumferential gaps; and (c) foundation with 2 mm circumferential gaps. In each case, the spectrum is normalized by its maximum ordinate. Vertical dashed lines indicate the identified first and second natural frequencies, f_1 and f_2

6.3.5 CONCLUSIONS

This chapter evaluated the influence of circumferential gaps at the tower-foundation interface on the global structural and dynamic behaviour of a wind turbine under operational conditions. By integrating the kinematic hardening interface model, developed in this project, into a full aero-servo-elastic

simulation, it was possible to assess the impact of gaps with thicknesses of 1 mm and 2 mm on key performance indicators. The modelling approach was validated with the measured moments and rotations during two shutdown events of the wind turbine.

The analysis of the structural response at the tower base demonstrated that the presence of gaps does not significantly change the magnitude of the operating bending moments. However, the rotational behaviour is notably affected; the loss of contact between the T-flange and the concrete induces a localised reduction in stiffness, leading to increased rotations that scale with the gap width. This response is characterised by a hysteretic behaviour where energy dissipation occurs during the open-gap phase, returning to linear elastic behaviour only upon the re-establishment of full contact.

Regarding the dynamic response at the top of the tower, the simulations revealed only minor deviations relative to the undamaged condition. The RMS of the accelerations exhibited a slight increase, ranging from 1.2% to 1.9%. In contrast, the horizontal displacement response was assessed through the 60 seconds window-mean displacement, which showed a small but systematic shift towards larger absolute offsets for the damaged interface cases, with the 2 mm gap producing the most pronounced mean displacement levels. These variations are consistent with a minor reduction in the effective stiffness and restraint at the tower-foundation interface due to partial loss of contact, while no evidence of resonant amplification or dynamic instability was observed under the simulated operational conditions.

The frequency domain analysis indicates that the presence of circumferential gaps of 1 mm and 2 mm does not lead to a measurable shift in the first two tower natural frequencies under standard operating conditions. The eigenfrequencies identified from the acceleration spectra remain stable at $f_1 = 0.34$ Hz and $f_2 = 2.85$ Hz for all interface configurations, with any potential differences being smaller than, or comparable to, the effective spectral resolution of the adopted Welch procedure. These results suggest that, during normal operation, the global dynamic stiffness governing the dominant tower modes is not significantly modified by the considered gap widths, which is consistent with a response in which the tower foundation interface is in full contact with the concrete.

In conclusion, the results confirm that while gaps up to 2 mm reduce the initial rotational stiffness and increase the flexibility of the structure, they do not compromise the operational stability or safety of the wind turbine under the examined conditions. The observed dynamic deviations are bounded and do not precipitate catastrophic failure modes, supporting the perspective that such defects, while requiring monitoring, may not necessitate immediate shutdown provided that the concrete foundation remains intact.

7 CONCLUSIONS

This thesis has addressed two critical challenges currently faced by the onshore wind industry: the need for optimised design methodologies capable of supporting increasingly large turbines, and the requirement for robust safety assessment strategies for ageing foundations, particularly those affected by defects at the interface between the steel tower and the concrete foundation. By combining extensive structural health monitoring campaigns with advanced nonlinear numerical modelling, the work has provided a comprehensive characterisation of the behaviour of Anchor Cage (AC) and Tower Foundation Section (TFS) foundation typologies, from local interface mechanics to global dynamic response.

7.1 STRUCTURAL MONITORING AND DATA INTERPRETATION

The experimental campaigns conducted at Serra del Rei and Serra da Lousã confirmed the suitability and reliability of the developed monitoring strategies for both AC and TFS foundations. The integration of strain gauges, accelerometers, inclinometers and LVDT sensors enabled the capture of the relevant structural response quantities under operational conditions and shutdown events, thereby providing a robust empirical basis for model calibration and validation.

For AC foundations, long term monitoring confirmed that prestressed anchor cage connections maintain a stable, predominantly linear moment-rotation response with high rotational stiffness at the tower base. The investigation of anchor bolt forces, based on a systematic comparison between experimental measurements and analytical and numerical predictions, demonstrated that the numerical model reproduces the measured forces with good accuracy. In contrast, the standard analytical formulation was found to overestimate force variations, potentially leading to conservative designs characterised by unnecessarily high grout and concrete strength classes and oversized steel components.

For TFS foundations, the monitoring approach proved capable of detecting and characterising interface damage at the connection between the embedded steel flange and surrounding concrete. Measurements of relative vertical displacements between the base of the tower and the concrete pedestal, combined with the analysis of moment-rotation hysteretic loops, allowed the clear identification of circumferential gaps between steel and concrete. In particular, the observed asymmetry in vertical displacements and

the nonlinear, hysteretic response in rotation were established as reliable diagnostic indicators of loss of contact. These findings demonstrate that appropriately designed monitoring systems can support the early detection and assessment of interface degradation in TFS foundations.

7.2 MODELLING STRATEGIES AND DESIGN OPTIMISATION

The comparison between standard linear design approaches and advanced nonlinear simulations revealed significant opportunities for improving both modelling practice and reinforcement design in onshore wind turbine foundations.

With regard to modelling adequacy, it was shown that conventional shell element models provide a reasonable approximation of the structural response for AC foundations, owing to the preservation of high rotational stiffness at the tower base. In contrast, these models were found to substantially overestimate the rotational stiffness of TFS foundations, by a factor of approximately 2.2, when embedded connections are present. As a consequence, standard shell-based design models are not suitable for accurately predicting the behaviour of TFS solutions, especially in the presence of interface degradation, and should be supplemented by three dimensional nonlinear analyses when such typologies are adopted.

In terms of design optimisation, the nonlinear analyses highlighted the potential for more efficient reinforcement layouts when nonlinear behaviour and realistic stress redistribution mechanisms are explicitly considered in the design process.

7.3 ASSESSMENT OF DAMAGED FOUNDATIONS

A central contribution of this thesis is the detailed characterisation of the structural performance of TFS foundations subjected to circumferential gaps at the steel-concrete interface.

A modelling strategy was implemented allowing the consideration of gaps between the T-flange of the embedded steel tower section and the surrounding concrete. The parametric study on interface mechanics showed that friction and cohesion are key parameters governing both rotational stiffness and energy dissipation in damaged foundations. The presence of gaps produces a bilinear moment-rotation response: an initial flexible branch controlled by gap opening and sliding, followed by a stiffer branch once contact is re-established between the embedded T-flange and the surrounding concrete. This bilinear behaviour is accompanied by pronounced hysteresis, which explains the increased damping observed in the monitored turbines. Good agreement with the measured moment-rotation curves was achieved.

The analyses further identified clear damage thresholds associated with gap magnitude. Gaps of approximately 1 mm lead to a reduction in rotational stiffness but do not immediately trigger material failure in the concrete pedestal under the considered load levels. However, gaps of 2 mm induce severe localised concrete crushing at the pedestal interface at load levels as low as about 32% of the quasi-permanent load. This result indicates that a gap thickness of 1 mm should be regarded as a critical operational threshold: above this value, the risk of progressive degradation, including fatigue damage and local crushing, increases significantly, and repair or mitigation measures should be prioritised.

7.4 GLOBAL OPERATIONAL IMPACT

The incorporation of the calibrated foundation interface models into global aero-servo-elastic simulations of the wind turbine enabled a direct assessment of how local foundation damage influences the overall dynamic behaviour of the wind turbine under operational conditions. For this purpose, a nonlinear model following the concepts of kinematic hardening plasticity was developed to reproduce the nonlinear moment-rotation behaviour of TFS foundation with gaps. The model showed good agreement with the measurements obtained during two shutdown events.

The analyses showed that, despite the reduction in local rotational stiffness associated with gaps up to 2 mm, the global stability of the wind turbine is not compromised under normal operation within the examined conditions. The overall bending moments at the tower base remain essentially unchanged, and the increases in tower top displacements (of the order of 7%) and accelerations (below 2%) remain moderate. These results indicate that the turbine retains adequate global performance margins, even in the presence of local interface degradation.

Under standard operating conditions, the considered gaps does not appear to affect the dynamic behaviour of the wind turbine. Any potential variations in the identified tower frequencies lie within the effective spectral resolution of the adopted frequency-domain procedure. The lack of measurable changes in the modal response suggests that the effective global stiffness remains essentially constant, consistent with the interface being largely closed and maintaining near-continuous contact during operation.

It can be concluded that foundations with circumferential gaps up to 2 mm do not, under the conditions investigated, pose an immediate risk of global collapse or dynamic instability during normal operation. Nevertheless, these foundations experience severe local stress concentrations and concrete damage at the pedestal interface, which accelerate fatigue and crushing and undermine long term durability. On this basis, a 1 mm gap threshold is proposed as a practical criterion for the initiation of repair or strengthening measures, even when short term operational safety appears to be preserved.

BIBLIOGRAPHY

1. Council, G.W.E., *GWEC global wind report 2024*. Global wind energy council: Lisbon, Portugal, 2024.
2. Council, G.W.E., *GWEC global wind report 2025*. Global wind energy council: Lisbon, Portugal, 2025.
3. (IEA), I.E.A., *Energy technology perspectives 2024*. Paris, France, 2024.
4. WindEurope, *Wind energy in Europe: 2024 Statistics and the Outlook for 2025-2030*. Brussels, Belgium, 2025.
5. Stehly, T., P. Duffy, and D. Mulas Hernando. *Cost of Wind Energy Review: 2024 Edition*. 2024.
6. Smoucha, E.A., et al., *Life cycle analysis of the embodied carbon emissions from 14 wind turbines with rated powers between 50Kw and 3.4 Mw*. Journal of Fundamentals of Renewable Energy and Applications, 2016. **6**(4): p. 1000211.
7. Hau, E., *Wind turbines: fundamentals, technologies, application, economics*. 2013: Springer Science & Business Media.
8. Krathe, V.L., *Aero-hydro dynamic analysis of offshore wind turbine-implementation of nonlinear soil-structure interaction in software fast*. 2015, NTNU.
9. Burton, T., et al., *Wind Energy Handbook*. 2011, UK: John Wiley & Sons.
10. Oliveira, G., *Vibration-Based Structural Health Monitoring of Wind Turbines*, in *Faculty of Engineering*. 2016, University of Porto: FEUP.
11. Manwell, J.F., J.G. McGowan, and A.L. Rogers, *Wind Energy Explained. Theory, Design and Applications*. 2009: John Wiley & Sons Ltd.
12. IEC-61400-1, *Wind turbines - Part 1: Design requirements*. 2019: IEC.
13. Massey, B.S., *Mechanics of Fluids*. 2012: Taylor & Francis Group.
14. Lock, C., H. Bateman, and H. Townend, *Experiments to verify the independence of the elements of an airscrew blade*. 1924: HM Stationery Office.
15. Wilson, R.E. and P.B. Lissaman, *Applied aerodynamics of wind power machines*. National Science Foundation, 1974.

16. Moriarty, P.J. and A.C. Hansen, *AeroDyn theory manual*. 2004, National Renewable Energy Lab., Golden, CO (US).
17. IEC-61400-6, *Wind turbines - Part 6: Tower and foundation design requirements*. 2016: IEC.
18. Bowles, J.E. and Y. Guo, *Foundation analysis and design*. Vol. 5. 1996: McGraw-hill New York.
19. Maunu, P., *Design of wind turbine foundation slabs*. 2008.
20. Wang, H., et al. *Experimental study on the mechanical characteristics of prefabricated dense ribs foundation for onshore wind turbines*. in *Structures*. 2025. Elsevier.
21. Esteyco. *Development of a new precast concrete foundation through braced foundations*. 2021 [cited 2021 14-03-2021]; Available from: <https://www.esteyco.com/en/proyectos/eurostars-full-scale-prototype-telescopic-tower/>.
22. DNV/Riso, *Guidelines for design of Wind Turbines*. 2002, Copenhagen: DNV/Riso.
23. fib, *Model Code 2010*. 2011: fib.
24. NP-EN-1992-1-1, *Eurocódigo 2 - Proyecto de estructuras de betão, Parte 1-1: Regras gerais e regras para edificios*, CEN, Editor. 2010.
25. Pacheco, J., et al., *New strategies for optimized structural monitoring of wind farms: experimental campaign*. *Wind Energy Science*, 2020. **5**(3): p. 983–996.
26. Loraux, C.T., *Long-term monitoring of existing wind turbine towers and fatigue performance of UHPFRC under compressive stresses*. 2018, Ecole Polytechnique Fédérale de Lausanne.
27. Pacheco, J., et al., *Experimental Evaluation of Bending Moments at the Tower of an Onshore Wind Turbine*. *EuroDyn*, 2020.
28. Pacheco, J., et al., *Use of strain monitoring data for fatigue assessment of wind turbine tower*. 10th SHMII Conference, Online., 2021.
29. Pacheco, J., et al., *Vibration-Based Monitoring of Wind Turbines: Influence of Layout and Noise of Sensors*. *Energies*, 2021. **14**(2): p. 441.
30. Oliveira, G., et al., *Modal decomposition of the dynamic response of wind turbine during one year of continuous monitoring*. *Structural Control and Health Monitoring*, 2021: p. e2754.
31. Morgan, K., E. Ntambakwa, and G. Hassan. *Wind turbine foundation behavior and design considerations*. in *Proceedings of the American Wind Energy Association Windpower Conference*. 2008.
32. Colombo, M., et al. *Limits of Current Design Approaches in the Analysis of Wind Turbine Foundations*. in *Italian Concrete Conference*. 2022. Springer.
33. Martinelli, P., et al. *Role of numerical modelling choices on the structural response of onshore wind turbine shallow foundations*. in *Structures*. 2022. Elsevier.
34. Bai, X., et al., *Modelling fatigue degradation of the compressive zone of concrete in onshore wind turbine foundations*. *Construction and building materials*, 2017. **132**: p. 425–437.

35. He, M., et al., *Structural monitoring of an onshore wind turbine foundation using strain sensors*. Structure and Infrastructure engineering, 2019. **15**(3): p. 314–333.
36. Zhou, X., H. Kong, and J.S. Dow, *Failure case study of reinforced-concrete foundations of wind turbine towers*. Proceedings of the Institution of Civil Engineers-Forensic Engineering, 2015. **168**(4): p. 158–166.
37. DIANA-FEA, *Diana Online Help*. 2019.
38. Feenstra, P.H., *Computational Aspects of Biaxial Stress in Plain and Reinforced Concrete*. PhD thesis. 1993: Delft University of Technology.
39. Rots, J.G. and J. Blaauwendraad, *Crack models for concrete, discrete or smeared? Fixed, multi-directional or rotating?* 1989.
40. Govindjee, S., G.J. Kay, and J.C. Simo, *Anisotropic modelling and numerical simulation of brittle damage in concrete*. International journal for numerical methods in engineering, 1995. **38**(21): p. 3611–3633.
41. Vecchio, F.J. and M.P. Collins, *Compression response of cracked reinforced concrete*. 1993. p. 3590–3610.
42. EN1992-1-1, *Eurocode 2: design of concrete structures – Part 1–1: general rules – rules for buildings, bridges and civil engineering structures*, CEN, Editor. 2023.
43. Borst, R., et al., *Non-Linear Finite Element Analysis of Solids and Structures*. Second ed. 2012: John Wiley & Sons, Ltd.
44. Petersen, C. and Stahlbau, *Vieweg-Verlag. Braunschweig*. 1997.
45. Bickford, J.H., *Introduction to the design and behavior of bolted joints: non-gasketed joints*. 2007: CRC press.
46. Schaumann, P. and M. Seidel. *Failure analysis of bolted steel flanges*. in *Proceedings of the Seventh International Symposium on Structural Failure and Plasticity (IMPLAST2000)*, Melbourne, Australia. 2000.
47. Mathern, A. and J. Magnusson, *Lessons Learned from the Construction, Inspection, and Defect Assessment of Reinforced Concrete Foundations for Wind Turbines*. Applied Sciences, 2022. **12**(3): p. 1443.
48. Currie, M., et al., *Structural integrity monitoring of onshore wind turbine concrete foundations*. Renewable energy, 2015. **83**: p. 1131–1138.
49. Bode, K.A. *In-Situ Experiences in Remediations of Wind Turbine Generator (WTG) Foundations with Resin and Various Cementitious Materials*. in *International Conference on Concrete Repair, Rehabilitation and Retrofitting*. 2024. Springer.
50. Bode, K.A. *Investigation in Remediation of Wind Turbine Generator (WTG) Foundations with Epoxy Resin*. in *MATEC Web of Conferences*. 2018. EDP Sciences.
51. Chen, J., et al., *Strengthening mechanism of studs for embedded-ring foundation of wind turbine tower*. Energies, 2021. **14**(3): p. 710.

52. Wang, W., et al., *Research on the strengthening and retrofitting methods of existing wind turbine foundations with embedded-ring*. IET Renewable Power Generation, 2023. **17**(16): p. 3793–3803.
53. Sobrinho, M., M. Pimentel, and F. Magalhães, *Nonlinear numerical modelling of onshore wind turbine foundations validated with experimental data*. Engineering Structures, 2025. **336**: p. 120441.
54. CEN, *EN 10025-2:2019 Hot rolled products of structural steels - Part 2: Technical delivery conditions for non-alloy structural steels*. 2019.
55. Esteyco, *Inspection of WTG foundations belonging to Lousã I wind farm. Measurement report*. 2016. p. 83.
56. Simis, A., *Ashes: aero-servo-hydro-elastic simulation* 2023.
57. Simis, A., *Engineering features summary, Ashes*. 2023.
58. Energy, G.W., *General Description. Wind turbine generator systems of the GE Wind Energy 2.x series*. 2004: Salzbergen.
59. Jonkman, J., et al., *Definition of a 5-MW reference wind turbine for offshore system development*. 2009, National Renewable Energy Lab.(NREL), Golden, CO (United States).
60. Jonkman, J.M. and M.L. Buhl, *FAST user's guide*. Vol. 365. 2005: National Renewable Energy Laboratory Golden, CO, USA.
61. Kim, N.-H., *Introduction to nonlinear finite element analysis*. 2014: Springer Science & Business Media.