ACP TR-2-2025

Cyclic Degradation in the Geotechnical Design of Wind Turbine Foundations Technical Report

A technical report prepared by the AMERICAN CLEAN POWER ASSOCIATION Wind Technical Standards Committee

202.383.2500 | 1299 Pennsylvania Ave NW, Suite 1300, Washington DC 20004 | cleanpower.org

AMERICAN CLEAN POWER ASSOCIATION STANDARDS

Standards promulgated by the American Clean Power Association (ACP), including recommended practices and technical reports, conform to the ACP Standards Development Procedures adopted by the ACP Board of Directors. The procedures are intended to ensure that ACP standards reflect a consensus to persons substantially affected by the standard. The ACP Standards Development Procedures are intended to be in compliance with the American National Standards Institute (ANSI) Essential Requirements. Standards developed under the ACP Standards Development Procedures are intended to be eligible for adoption as American National Standards.

Published by:

American Clean Power Association

1299 Pennsylvania Ave, N.W.,

Suite 1300

Washington D.C. 20004

www.cleanpower.org

NOTICE AND DISCLAIMER

The American Clean Power Association (ACP) has provided this Document for the use subject to important notices and legal disclaimers. This Document is proprietary and its use is subject to a legally binding license agreement and disclaimer ("Agreement") described herein and available on ACP's website at https://cleanpower.org/standards-development, which may be updated from time to time. Do not use this Document for any purpose unless and until you read the agreement. By viewing or otherwise using this Document, you hereby warrant and represent that you have read and agree to be legally bound by the agreement and are authorized to bind not only yourself to the agreement, but the organization for which you are accessing this Document.

Notice and Disclaimer Concerning ANSI Process

Certain ACP standards and best practice publications, of which the Document contained herein is one, are developed through a voluntary consensus standards development process. ACP administered the process in accordance with the procedures of the American National Standards Institute (ANSI) to promote fairness in the development of consensus. This process brings together volunteers and/or seeks out the views of persons who have an interest in the topic covered by this Document. The information in this Document was considered technically sound by the consensus of persons engaged in the development and approval of the Document at the time it was developed. Consensus does not necessarily mean there is unanimous agreement among every person participating in the development of this Document.

Notice and Disclaimer Concerning Accuracy of Information and Liability Concerning the Use of ACP Publications

Every effort has been made to ensure the accuracy and reliability of the data and information contained in this Document; however, ACP does not write this Document and it does not independently test, evaluate or verify the accuracy or completeness of any information or the soundness of any judgments contained in its publications. ACP disclaims and makes no guaranty or warranty, express or implied, as to the accuracy or completeness of any information published herein.

In publishing and making this Document available, ACP is not undertaking to render professional or other services for or on behalf of any person or entity. This Document and ACP publications in general necessarily address problems of a general nature. ACP disclaims and makes no guaranty or warranty, express or implied, as to the accuracy or completeness of any information published herein, and disclaims and makes no warranty that the information in this Document or its other publications will fulfill any of your particular purposes or needs. ACP does not undertake to guarantee the performance of any individual manufacturer or seller's products or services by virtue of this Document.

Users of this Document should not rely exclusively on the information contained in this Document and should apply sound business, scientific, engineering, and safety judgment in employing the information contained herein or, as appropriate, seek the advice of a competent professional in determining the exercise of reasonable care in any given circumstances. Information and other standards on the topic covered by this Document may be available from other sources, which the user may wish to consult for additional views or information not covered by this Document.

Use of this Document is strictly voluntary. ACP has no power, nor does it undertake to police or enforce compliance with the contents of this Document. ACP does not certify, test, or inspect products, designs, or installations for safety or health purposes. Any certification or other statement of compliance with any health- or safety-related information in this Document shall not be attributable to ACP and is solely the responsibility of the certifier or maker of the statement.

ACP disclaims liability for any personal injury, property, or other damages of any nature whatsoever, whether special, indirect, consequential, or compensatory, directly or indirectly resulting from the publication, use of, application, or reliance on this Document or on any of its other publications, even if advised of the possibility of such damage and regardless of whether such damage was foreseeable. In addition, ACP does not warrant or represent that the use of the material contained in this Document is free from patent infringement. ACP publications are supplied "AS IS" and "WITH ALL FAULTS."

Laws & Regulations

When using this Document, local, state, and federal laws and regulations should be reviewed. Compliance with the provisions this Document does not constitute compliance to any applicable legal requirements. In making its publications and this Document available, ACP does not intend to urge action that is not in compliance with applicable laws, and these documents may not be construed as doing so. Users of this Document and other ACP publications should take into account state, local, federal, or international data privacy and data ownership requirements in the context of assessing and using the publications in compliance with applicable legal requirements.

ACP Wind Technical Standards Committee members, at the time the standard was approved:

Company Name	Primary	Alternate
Acciona Energy USA Global LLC	Brian Mathis	Richard Heier
Advanced Insulation Systems LLC	Mike Sherman	
Alimak Group USA Inc.	Luke Metzinger	
American Renewable Energy Standards and Certification Association (ARESCA)	Brian McNiff	Robert Sherwin
ArcVera Renewables	John Bosche	Gregory Poulos
Avangrid Renewables	Christopher Morris	Agustin Melero
Barr Engineering Company	Kirk Morgan	Cordelle Thomasma
Bergey Windpower Co. Inc.	Mike Bergey	
Boulder Wind Consulting	Sandy Butterfield	
Clear Wind	Matt McCabe	
Clobotics Corporation	Nicholas Acorn	Rogers Weed
CTE WIND USA, Inc. - CIVIL ENGINEERING	Jomaa Ben-Hassine	Alexander Martin
DNV	Brian Kramak	Dayton Griffin
Elevator Industry Work Preservation Fund	Carisa Barrett	Eric Rogers
ENERCON Canada Inc.	Tarik Daqoune	
Energy Systems Integration Group (ESIG)	Cody Craig	Stanton Peterson
Exponent	Timothy Morse	Bernard Roesler
GE Renewable Energy	Mark Helton	Toby Gillespie
HSB - Hartford Steam Boiler	Linkesh Diwan	Simon Krebs
Leeward Renewable Energy	Ronald Grife	
Marmon Electrical - RSCC Wire & Cable	Damian Billeaudeau	
MotorDoc LLC	Howard W. Penrose	Branka Stemack
National Renewable Energy Laboratory	Jeroen van Dam	Paul Veers
NRG Systems	Rob Cole	Steven Clark
nVent ERICO	Tom Bendlak	Dan Holm
Pattern Energy Group Services, LP	Michael Edds	
Penn State University	Susan W. Stewart	
RE Innovations LLC	Joseph Spossey	
Sargent & Lundy LLC	David Jolivet	
Siemens Gamesa Renewable Energy	Masoud Sharifi	
Small Wind Certification Council	Shawn Martin	
Spire Engineering Services LLC	Robert Sheppard	
Terra-Gen LLC	Yuanlong Hu	
U.S. Department of Energy	Jim Ahlgrimm	Michael Derby
UL LLC	Tim Zgonena	Jason Hopkins
University of Texas at Austin	Lance Manuel	
Vestas North America	Trevor Taylor	John Eggers
WSB-Hawaii	Warren Bollmeier	

ACP TR-2 Subcommittee members, at the time the standard was approved:

Organization	Name
Barr	Rob Osburn
CTE	Jomaa Ben-Hassine
DNV	Bruno Mendes
DNV	Eric Ntambakwa (Chair)
DNV	Marcus Rasulo
DNV	Matthew Rogers
DNV	Chris Yu
EDF	Gene Bryant
EDF	Maurizio Marchetti
EDPR	Alvaro Marquez Garcia
Geocomp	Antonios Vytiniotis
Hatch	Bill Kussman
Natural Power	Chad Homuth
NV5	Amr Sallam
RES Americas	Mohamed Nofal
RRC	Ben Krause
RRC	Rohit Pant
RRC	Dongyuan Wang
RRC	Alfred Williams
RWE	Carlos Guzman
Sargent & Lundy	Mourad Riad
SC Solutions	Reyhaneh Navabzadeh
UL	Alireza Akhavan
UL	Onur Kacar
Ulteig	Zachary Grabowski
Vestas	Trevor Taylor

ACP TR-2 Advisory/Review Panel

Organization	Name
Lymon C. Reese and Associates	Shin-Tower Wang
University of California, Davis	Katerina Ziotopoulou
University of California, San Diego	John McCartney

ACP TR-2 Subcommittee Working Group members, at the time the standard was approved:

Working Group 1: Geomaterial Degradation Behavior, Cyclic Behavior, Laboratory and Field Testing

Name
Eric Ntambakwa (WG1 Lead)
Carlos Guzman
Bruno Mendes
Rohit Pant
Marcus Rasulo
Amr Sallam
Dongyuan Wang
Chris Yu

Working Group 2: Numerical Modelling

Name
Onur Kacar (WG2 Lead)
Jomaa Ben-Hassine
Bruno Mendes
Eric Ntambakwa
Marcus Rasulo
Amr Sallam
Dongyuan Wang
Chris Yu

Working Group 3: Loading Considerations

Name
Alireza Akhavan (WG3 Lead)
Carlos Guzman
Ben Krause
Eric Ntambakwa
Trevor Taylor

Working Group 4: Pile, Pier, and Anchored Foundations

Name
Matt Rogers (WG4 Lead)
Onur Kacar
Ben Krause
Rob Osburn
Dongyuan Wang
Chris Yu

Working Group 5: Shallow Foundations

Name
Bill Kussman (WG5 Lead)
Jomaa Ben-Hassine
Carlos Guzman
Ben Krause
Maurizio Marchetti
Bruno Mendes
Eric Ntambakwa
Rob Osburn
Rohit Pant
Marcus Rasulo
Amr Sallam
Dongyuan Wang

Working Group 6: Monitoring, Inspection, and Measurements

Name
Carlos Guzman (WG6 Lead)
Alireza Akhavan
Jomaa Ben-Hassine
Matt Rogers
Amr Sallam
Maurizio Marchetti
Riad Mourad

ACP TR-2 Acknowledgements

Name	
Aaron Bradshaw	University of Rhode Island
Chris Feist	University of Minnesota Eolos Wind Research Field Station
Bob Gilbert	University of Texas Austin
D. Vaughn Griffiths	Colorado School of Mines
Filipe Magalhães	University of Porto
Catalina Maldonado	Norwegian Geotechnical Institute
Helen Mitrani	University of Newcastle
Mário Pimentel	University of Porto
Mário Sobrinho	University of Porto
Ken Stokoe	University of Texas Austin
Victor Taboada	Fugro
Zack Westgate	University of Massachusetts Amherst

Table of Contents

Figures

Tables

1 Scope

The report reviews the behavior of foundation support geomaterials under cyclic loading for various onshore wind turbine foundation types and highlights the importance of design to limit geomaterial degradation. The report is motivated by the need to ensure satisfactory foundation performance during service life particularly when the foundation-geomaterial interface is subjected to a high number of cycles between compression and zero loading. Load cases are discussed with reference to the ACP 61400-6 standard. Design recommendations for different foundations and soil types are provided. General guidance is also provided for laboratory testing, numerical modelling, in-service monitoring, and issue mitigation.

2 Introduction and Purpose

The long-term performance of geomaterials supporting wind turbine foundations under cyclic loading has not historically been well investigated nor incorporated in design codes and standards for onshore wind turbine foundations. Depending on various factors, cyclic loading may result in soil strength and stiffness degradation, leading to accumulation of foundation displacements and significant reduction of soil bearing capacity. The potential for changes in the foundation stiffness has implications for turbine operation frequency and foundation loads over time, which could adversely influence wind turbine production and impact fatigue and ultimate loads on the foundation system. Therefore, it is necessary that design practices evolve to assess and address the risks associated with cyclic degradation of foundation support materials.

The American Clean Power Association (ACP) Wind Turbine Standards Committee authorized a subcommittee to develop a report that clearly identifies typical and specific U.S. national wind turbine foundation geotechnical design practice relative to cyclic degradation of foundation support materials. This report on cyclic degradation in the geotechnical design of wind turbine foundations is the product of the authorized subcommittee.

3 Symbols and Abbreviated Terms

3.1 Symbols

γ_a	average shear strain
γ_{cy}	cyclic shear strain
γ_p	permanent shear strain
γ_{td}	degradation threshold cyclic shear strain
γ_{tf}	flow threshold cyclic shear strain
γ_{tl}	linear threshold cyclic shear strain
γ_{tv}	volumetric threshold cyclic shear strain
φ	friction angle
c	cohesion
G	shear modulus
u_{cy}	cyclic pore pressure
u_p	permanent pore pressure

3.2 Abbreviations and Acronyms

CLRS	critical level of repeated stress
CPT	cone penetration test
DSS	direct simple shear
FDM	finite difference method
FEA	finite element analysis
FEM	finite element method
OCR	over-consolidation ratio
OEM	original equipment manufacturer
SCADA	supervisory control and data acquisition
SLS	serviceability limit state
VNP	Venezuelan North of Paria
WTG	wind turbine generator

4 Design Standards

The following design standards and recommended practice documents are common references for geotechnical design of wind turbine foundation and may be consulted when performing assessment of cyclic degradation of geomaterials.

- ACP 61400-6-2023, *Wind Energy Generation Systems – Part 6: Tower and Foundation Design Requirements – Modified Adoption of IEC 61400-6*
- ASCE/AWEA RP2011, *Recommended Practice for Compliance of Large Land-based Wind Turbine Support Structures*
- DNV-RP-C207, *Statistical Representation of Soil Data*, 2021
- DNV-RP-C212, *Offshore Soil Mechanics and Geotechnical Engineering*, September 2021
- DNV-ST-0126, *Support Structures for Wind Turbines*, December 2021
- EPRI, *Manual on Estimating Soil Properties for Foundation Design*, August 1990
- IEC 61400-1, *Wind energy generation systems – Part 1: Design requirements*, 2019
- NAVFAC DM-7.01, *Soil Mechanics*, 1 February 2022
- NAVFAC DM-7.02, *Foundations & Earth Structures*, 1986

5 Introduction to Geomaterial Degradation

5.1 General

Geomaterials that are subjected to cyclic loading may exhibit loss of strength and stiffness with increasing number of loading cycles due to several factors and mechanisms. The potential for strength and stiffness degradation depends on various factors including geomaterial type, degree of saturation, magnitude of applied cyclic shear stresses and strains, number of loading cycles and frequency of loading.

5.2 Geomaterial Behavior under Cyclic Loading

Geomaterials are broadly classified as coarse-grained (sand and gravel) or fine-grained soils (clay and silt). Coarse-grained soils are also commonly referred to as noncohesive soils whereas fine-grained soils are referred to as cohesive soils. The behavior of geomaterials under cyclic loading is a function of various factors and the effective stress, degree of saturation and drainage conditions are primary ones. Geomaterials behavior whether excess pore pressure can dissipate or not during loading depends on the permeability of the geomaterial and the loading frequency. If the loading frequency is relatively high and the permeability is low, geomaterials tend to show undrained behavior where excess pore pressure cannot dissipate upon loading. At small cyclic shear strains, soils will behave in an elastic manner regardless of the drainage conditions, with a small-strain shear modulus that depends on the effective stress (Hardin and Black 1968, 1969). In unsaturated conditions, the degree of saturation, matric suction, and hydraulic hysteresis play important roles in the effective stress and thus affect the small-strain shear modulus (Khosravi and McCartney 2012; Dong et al. 2016). At larger cyclic shear strain amplitudes, the drainage conditions become more important as plastic shear and volumetric strains may occur. Cyclic loading of saturated soils in drained conditions is well known to lead to a nonlinear decrease in volume with the number of cycles (Youd 1972). If cyclic loading is applied to a saturated geomaterial in undrained conditions, excess pore pressures may develop, which could lead to reduction effective stress, in turn reducing the shear strength and stiffness (Mortezaie 2012). The resulting reduction in shear strength and stiffness may be permanent depending on the duration, frequency, and amplitude characteristics of loading, and the material properties. If cyclic shearing is applied to an unsaturated geomaterial in undrained conditions, excess pore air and water pressures may develop and permanent volumetric strains may occur leading to changes in degree of saturation, both of which may lead to changes in effective stress (Rong and McCartney 2020a). Slower cyclic shearing in drained conditions may still lead to changes in volume, and smaller decreases in effective stress due to the increase in degree of saturation and effective stress (Rong and McCartney 2020b). The response of various geomaterials to cyclic loading varies depending on whether the material primarily consists of cohesive soils, cohesionless soils, intermediary (showing partially cohesive and partially cohesionless soil behavior) or bedrock materials. The following sections discuss the general behavior of the various geomaterial types under cyclic loading considered in this report.

5.2.1 Mechanism of Soil Degradation under Cyclic Loading

Figure 1 represents a typical dry shearing behavior of sands. When loose soil is subjected to shear loading, the particles have a tendency to rearrange resulting in contraction (i.e., slip-down and volume reduction) of the soil volume. Rearrangement of soil particles typically reduces the volume of the voids. If the soil is saturated and the pore water cannot escape (undrained), the tendency for volume change will generate excess pore water pressure if the rate of shearing is sufficient to lead to undrained conditions (Gibson et al. 1989). If soils have high permeability, which allows the excess pore water to rapidly dissipate, the soils subsequently densify (Van Wijngaarden 2018). When the rate of loading is sufficiently high or soils have low permeability, the generation of excess pore water pressure could result in the deterioration of stiffness and strength in sands (e.g., Liu and Xu 2013) and fine-grained soils (e.g., Ansal and Erken 1989; Okur and Ansal 2007). The buildup of excess pore water pressure may even trigger a complete loss of strength in saturated sands or may generate significant strains in clays and plastic silts if the magnitude of the cyclic loading is sufficiently high (Idriss and Boulanger 2008). A load reversal (i.e., unloading) can cause a subsequent slip-down resulting in more volumetric contraction hence continued cyclic shear loading can contribute to cumulative contraction and thus continued pore pressure increase during shearing.

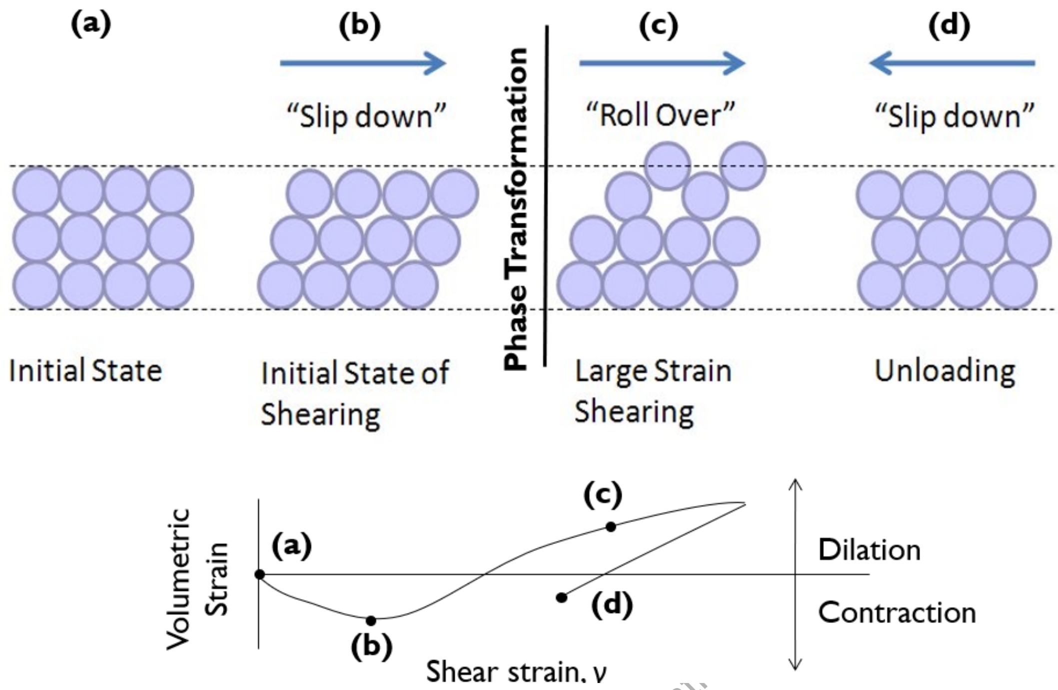


Figure 1: Granular material deformation mechanics (Vytiniotis 2012)

It is well known that the excess pore water pressure can soften the shear strength and shear modulus of geomaterials. Soil shear strength varies significantly depending on whether the material behaves in a drained or undrained manner. In drained loading for a saturated soil, pore pressures dissipate with load application while significant pore pressures may be generated during undrained loading leading to reduced failure strength. The difference in behavior in triaxial compression can clearly be demonstrated by tracking undrained and drained effective stress paths to the critical state line as shown in Figure 2, where q is the principal stress difference, and p' is the mean effective stress.

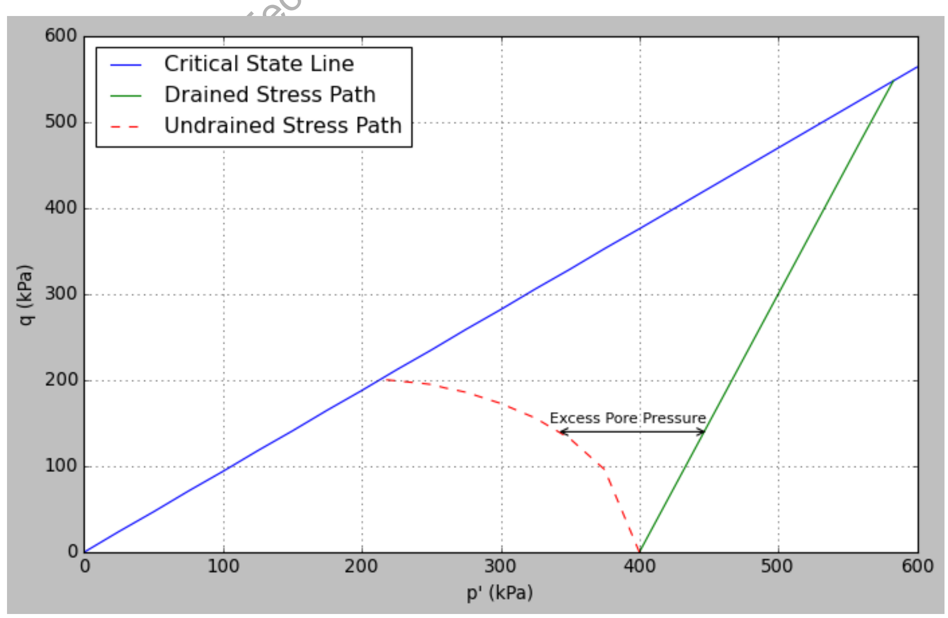


Figure 2: Comparison of drained versus undrained stress paths and pore pressure in triaxial loading

For the drained loading stress path, there is a linear relationship between increasing effective stress and shear stress while for the undrained stress path, there's a reduction in effective stress due to an increase in excess pore pressure during shearing. The drained stress path reaches the yield surface at a much higher shear stress compared to the undrained stress path, and the interpreted shear strength would, therefore, be higher for drained loading compared to undrained loading.

For cyclic loading, similar behavior is evident and depending on the frequency of load application, pore pressures may not dissipate during undrained loading and pore pressure accumulation may occur at shear stress levels below those in monotonic loading (Vaid and Chern 1983, 1985). The undrained stress path in undrained loading may therefore reach the yield surface at a lower shear stress level than in static loading as depicted in Figure 3 (Figure 6.2 from Andersen 2015).

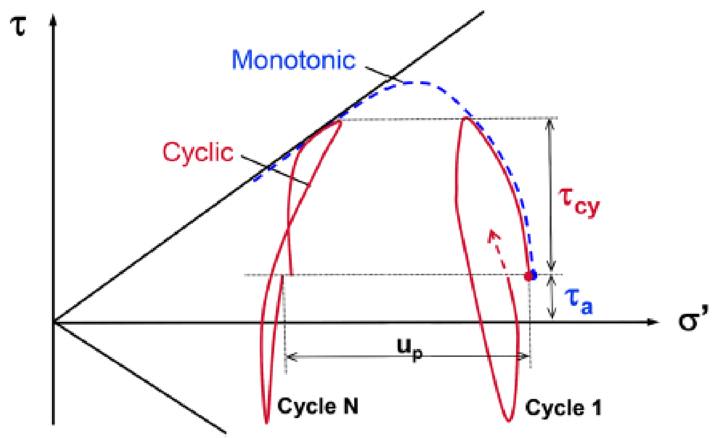


Figure 3: Effective stress paths for undrained tests (Andersen 2015)

The behavior depicted in Figure 3 demonstrates how cyclic strength can be lower than static strength for geomaterials. Assessing whether a geomaterial will behave in a drained or undrained manner when being sheared, and whether the loading is static or cyclic is therefore an important aspect of evaluating potential for degradation of geomaterials.

As cyclic loading progresses, excess pore water pressure generation may continue to increase until soil failure is triggered (e.g., liquefaction for sandy soils or cyclic softening/large plastic displacement for cohesive soils). The resulting large deformations are due to progressive reduction in shear modulus due to increased shear strains and resulting excess pore water pressure (Matasovic and Vucetic 1993; Chang et al. 2007; Khashila et al. 2021).

5.2.2 Geomaterial Cyclic Thresholds

Geomaterial degradation is a function of the applied cyclic shear strain (Andersen et al. 1988; Vucetic and Dobry 1991; Vucetic 1994). At loading levels that induce very small cyclic shear strains (on the order of about 10^{-5} or less), excess pore pressures do not develop, the soil structure remains unchanged (Mortezaie 2012) and the material essentially behaves in a linear elastic manner (Vucetic 1994). As the cyclic shear strain level increases, the behavior of the geomaterial progresses from linear elastic behavior where no pore pressures develop (and strength and stiffness remain constant) to nonlinear elastic behavior where nonpermanent soil-structure changes may develop, but strength and stiffness loss is recoverable, to nonlinear plastic behavior where permanent changes to the soil micro structure may occur due to excess pore pressure development, and strength and stiffness loss is permanent/nonrecoverable. The changes in material behavior are classified based on cyclic shears strain level boundaries or thresholds (Vucetic 1994). The threshold strains representing material behavior corresponding to linear (γ_{tl}), volumetric (γ_{tv}), degradation (γ_{td}), and flow (γ_{tf}) thresholds as depicted in Figure 4. Magnitudes of the threshold shear strains are predominantly defined by the plasticity index of the soils, nevertheless, the influence of other parameters such as OCR, confining stress, density, saturation, and soil fabric should not be entirely discounted. Various factors influence the onset of soil degradation and site-specific validation utilizing laboratory or in situ testing is therefore recommended.

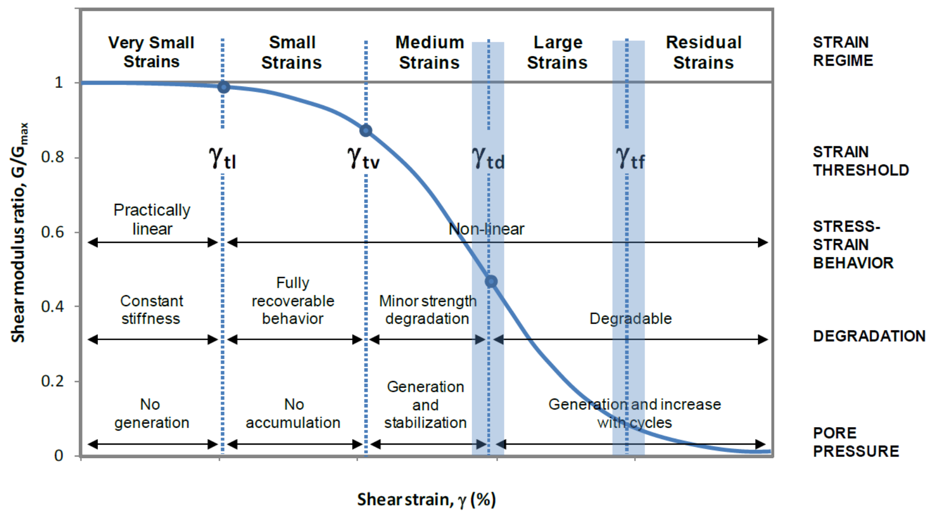


Figure 4: Shear modulus strain relationship (adapted from Diaz-Rodriguez and Lopez-Molina 2008)

Similar to the cyclic shear strain-based approach for assessing load levels that trigger cyclic degradation, a cyclic shear stress-based approach was proposed by Sangrey et al. (1969) who discuss the concept of critical level of repeated stress (CLRS) that represents a boundary between stable soil response and behavior where strength degradation occurs with increasing number of loading cycles. The CLRS concept was developed on the basis of cyclic triaxial tests that indicated that the risk for cyclic degradation is low at loading levels below the CLRS and increases above that loading level. Stress- and strain-based approaches for assessing geomaterial degradation can facilitate evaluation of wind turbine foundation response to cyclic loading and may be applied concurrently to assess sensitivity of the results to different approaches for evaluating geomaterial degradation risk.

In order to perform a reliable cyclic degradation assessment of a geomaterial, it is necessary to understand what level of cyclic shear stress and/or strain would result in strength/stiffness degradation and the corresponding softened strength/stiffness for the soil. Such an assessment may require site-specific cyclic soil testing under wind turbine loading conditions consistent with those anticipated for the foundations. The results of cyclic soil testing can then be incorporated into assessing susceptibility of the geomaterial to degradation and the associated consequences, if any.

5.2.3 Cohesive Soils

Cyclic degradation in clays is a function of various parameters including level of saturation, shear stress and strain level, plasticity index, effective stress and loading frequency. The effect of over-consolidation ratio on degradation is however less certain with some research indicating no visible effects (e.g., Mortezaie 2012) on cyclic thresholds, whereas others (e.g., Andersen et al. 1988; Vucetic and Dobry 1991; Darendeli 2001) indicate that it can be a factor for shear strength and degradation considerations.

Marine clay has been the study subject for stiffness and strength degradation since loading due to earthquakes and ocean waves/storms often induce severe undrained cyclic shear loading of deposits underlying offshore structures. Two laboratory tests methods have been suggested for assessing strength and stiffness degradation: 1) cyclic stress-controlled tests and 2) cyclic strain-controlled tests (Vucetic and Dobry 1988).

5.2.4 Cohesionless Soils

Cyclic degradation in granular soils is a function of various parameters including fabric, level of saturation, shear strain level, relative density of the soil and effective stress level (state of soil). The behavior of granular soils under cyclic loading generally follows the effective stress principle. The threshold shear strain, i.e., the shear strain that will induce volumetric changes for granular soils has been investigated by many researchers. It is typically reported to be about 10^{-4} for most sands and silty sands regardless of their relatively densities (Dobry et al. 1982; Erten and Maher 1995).

The lower the relative density of the soil and the higher effective stress level, the more contractive behavior the soil will exhibit and, hence, more degradation due to cyclic loading when such soils are saturated. Continuous accumulation of excess pore pressures in a granular soil during cyclic shearing can lead to liquefaction and complete loss of shear strength and stiffness. When soil is not saturated, or when they are allowed to drain, and the cyclic threshold shear strain is exceeded, large irreversible strains can occur that can lead to ground failure manifested as cracks and differential settlement (Rascol 2009). Granular soils may also exhibit ratcheting plasticity behavior with continuous accumulation of strains under cyclic loading even if no additional accumulation of excess pore pressures is taking place (Kammerer et al. 2002).

The susceptibility to cyclic degradation or liquefaction of granular soil has been predominantly researched for saturated loose sands or low-plastic silts. Fully saturated cohesionless soils generally have the highest susceptibility to cyclic degradation (Mortezaie 2012). However, many field and laboratory investigations show that partially saturated granular soils can also be degraded or liquefied under certain conditions, especially for soils in shallower depths with low overburden pressure. Various constitutive models, empirical approaches and frameworks have been proposed to evaluate the potential of cyclic degradation and liquefaction for unsaturated soils (Liu and Xu 2013; Zhang et al. 2016; Banerjee 2017).

5.2.5 Intermediate Soils

Intermediate soils are soils whose strength can be characterized by both cohesion and friction angle. Apparent cohesion values may be observed in the failure envelope of soils due to negative pore water pressures or from cementation that is dependent on the chemical and deposition history of the sediment. Such soils can be, for example, fine silts, clayey sands, silty sands, sandy clays. Cyclic degradation in intermediate soils is a function of various parameters, including level of saturation, shear strain level, relative density of the soil, and effective stress level.

Atterberg limits have been used to evaluate whether a fine-grained soil will exhibit a sand-like or a clay-like behavior, and generally the cyclic strength of such soil transitions to clay-like behavior when the plasticity index is about greater than 7, as shown in Figure 5. Fine-grained soils with plasticity index of between 3 to 6 will have cyclic strengths greater than similar nonplastic soils (Idriss and Boulanger 2008). Similarly, to granular soils, progressive accumulation of pore pressures can lead to substantial strength loss and eventually liquefaction. For such soils, in particular the specific soil structure that is contributing to the cohesive part of the strength may be broken down during cyclic loading leading to large strength loss and contributing to large permanent deformations. Intermediate soils are generally more fine-grained than clean sands, which can profoundly limit their hydraulic conductivity and subsequently their capability to rapidly dissipate excess pore pressures. However, their finer structure will contribute to higher levels.

Hence, when intermediary soils are present, a detailed laboratory testing program should be assessed to evaluate their particular liquefaction or cyclic softening and the broader cyclic degradation characteristics overall.

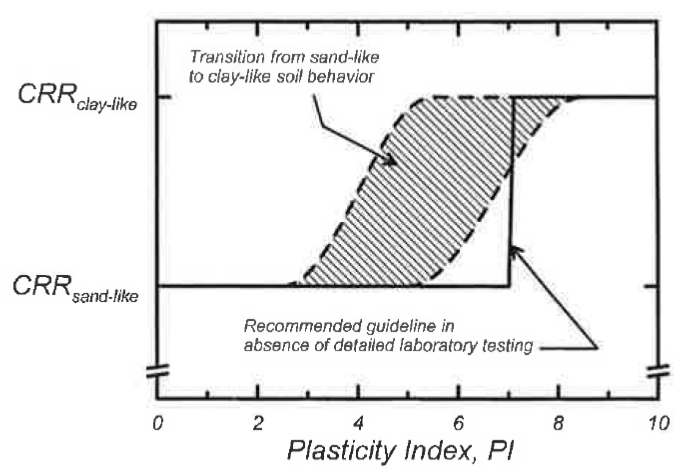


Figure 5: Illustration of transition from sand to clay-like behavior with increasing plasticity index (Idriss and Boulanger 2008)

5.3 Cyclic Degradation Assessment Framework

Designing a foundation for cyclic loading such as imposed by wind turbines requires an assessment of the risks associated with cyclic degradation of foundation support materials, and these need to be appropriately accounted for in design As discussed herein, such an assessment requires understanding of strength and deformation characteristics under combined sustained and cyclic stresses, whether applied stresses are one or two directional and whether cyclic stresses are symmetrical or nonsymmetrical with respect to the sustained stress (Seed and Chan 1966). Various approaches and frameworks can be utilized for such an assessment and should generally include representative soil strength and deformation properties, appropriate stress and/or strain levels and thresholds to provide realistic results. An example framework for assessment of the effects of cyclic loading is presented in Figure 6.

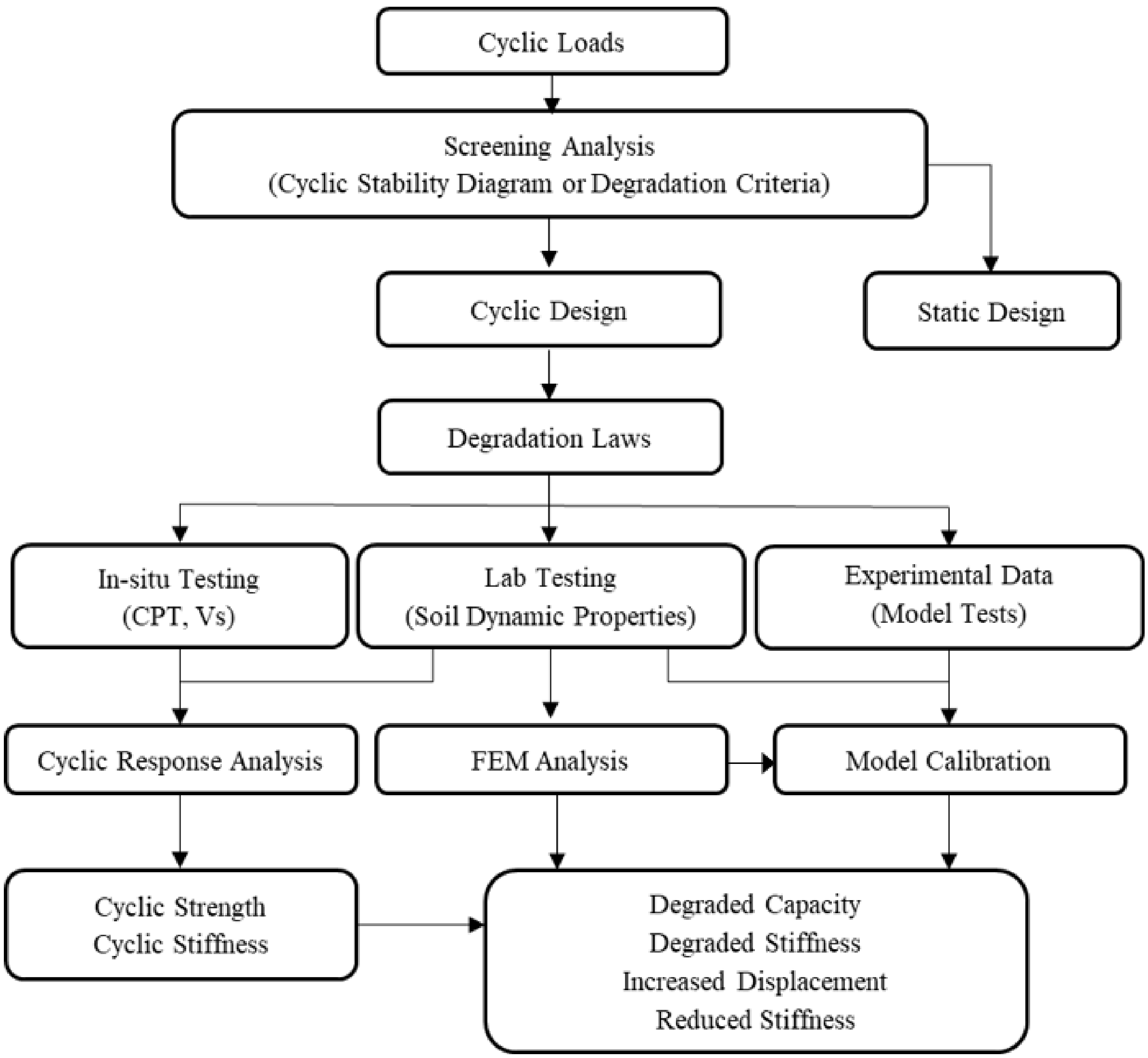


Figure 6: Example cyclic loading analysis framework (adapted from Yu et al. 2016 and Jardine et al. 2012)

This type of a framework facilitates a phased approach to the assessment of cyclic degradation starting with an initial qualitative screening of the potential risk for degradation and, depending on the results of the initial screening, progressing to either a static design (i.e., low risk for degradation) or a quantitative assessment of foundation cyclic response including advanced cyclic soil testing, finite element analyses and evaluating impact on foundation response.

5.4 Groundwater and Pore Pressure Effects

As discussed previously, under undrained conditions, volumetric changes will be prevented by the low compressibility of the water and pore water pressures are generated. In monotonic testing, soil may exhibit a peak shear stress, soften and approach the failure envelope. In cyclic testing, the soil may be loaded with a maximum shear stress that is smaller than the monotonic peak shear stress, with the load cycling with a single-amplitude shear stress, τ_{cy}, around an average shear stress τ_a (Figure 7, panel a). The cyclic loading generates pore pressures characterized by a permanent pore pressure component, u_p, and a cyclic pore pressure u_{cy} (Figure 7, panel b). The increment in pore pressure reduces the effective stresses in the soil, resulting in increased average (γ_a), permanent (γ_p), and cyclic (γ_{cy}) shear strains with time (Figure 7, panel c). The permanent pore pressure, u_p, can be used to quantify the accumulated effect of cyclic loading during a cyclic event.

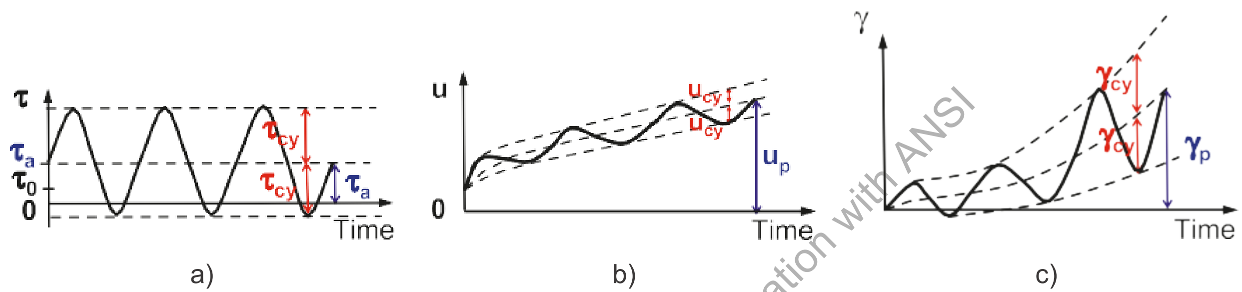
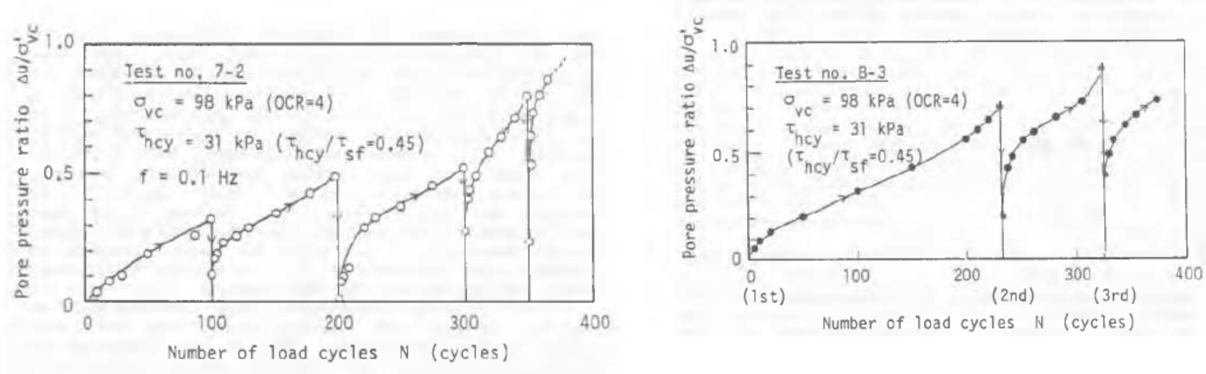


Figure 7: Pore pressure and shear strain as functions of time under undrained cyclic loading: a) cyclic and average shear stresses; b) pore pressure generation; c) cyclic and permanent shear strains (Andersen 2009, 2015)

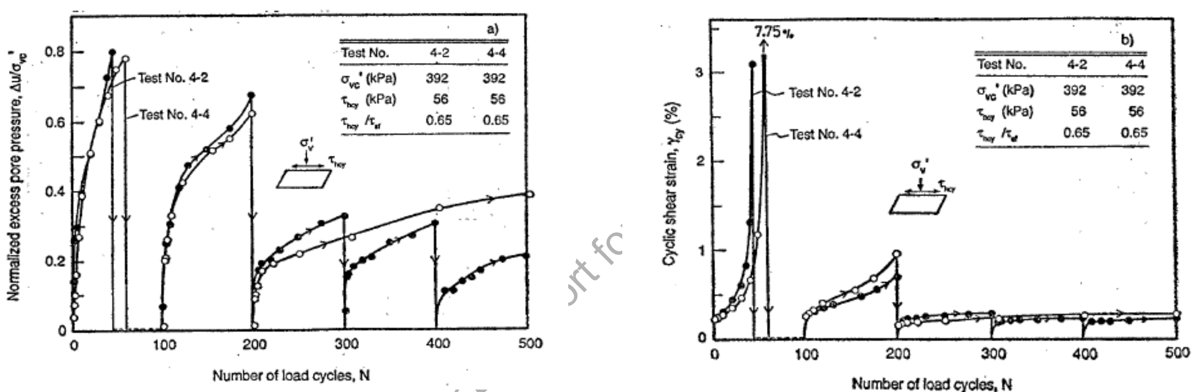
External and internal drainage conditions are significant factors in the stability of the geomaterial during cyclic loading (Mamou et al. 2017). However, below certain cyclic threshold limits, geomaterials may largely behave in an elastic manner and therefore may remain stable over large number of loading cycles under both drained and undrained conditions (Matasovic and Vucetic 1995; Mamou et al. 2017). Once the cyclic degradation threshold is exceeded, the rate of dissipation of excess pore water pressure between successive peak loads, which in turn depend on the permeability of geomaterial, will dictate the onset of plastic shear strain development and subsequent failure. During periods with lower magnitude cyclic loading, the excess pore pressures generated by cyclic loading may dissipate. Yasuhara and Andersen (1989) performed two cyclic direct simple shear (DSS) tests on over-consolidated clay (OCR = 4) with drainage, with 5 periods of undrained cyclic loadings followed by drainage after each series. For the testing with drainage periods of 60 min, the pore pressure generation increased with each successive cyclic loading series, reaching failure on the 4th series of cyclic loading. However, when the drainage period increased to 24 hours, the soil reached complete failure after the 2nd series of cyclic loading due to a rapid increase in the pore pressure and shear strain. Yasuhara and Andersen (1989) concluded that cyclic loading accompanied by drainage may have a detrimental effect on the response of over-consolidated clays since they will have a lower resistance to the subsequent undrained cyclic loading events (Figure 8).

Similar tests were performed by conducting DSS tests on normally consolidated Drammen clay, in five series of undrained cyclic loading and drainage after each series. The results showed that cyclic loading accompanied by drainage has the potential to increase the resistance of normally consolidated clays to subsequent undrained cyclic loading (Figure 9), as the cyclic shear strains and excess pore pressures decreased in the later series of cyclic loading applied in the tests (Yasuhara and Andersen 1991).



a) b)

Figure 8: Results from cyclic DSS test on over-consolidated clays with drainage (Yasuhara and Andersen 1989): a) 60 min after each series; b) 24 h after each series



Test No.	4-2	4-4
σ_{vc}' (kPa)	392	392
τ_{hcy} (kPa)	56	56
τ_{hcy}/τ_{sf}	0.65	0.65

a) b)

Figure 9: Results from tests on normally consolidated clays with several series of undrained cyclic loading and drainage (Yasuhara and Andersen 1991)

The assessment of geomaterial behavior typically requires an assumption of either fully drained or undrained behavior. In reality, geomaterials can exhibit behavior that is characterized by partial drainage and the fully drained or undrained frameworks may therefore not be appropriate for assessment of strength or stiffness. The nature of geomaterials that typically include alternative layers of granular and cohesive deposits may contribute to composite behavior that cannot be characterized as simply drained or undrained. Pore pressure generation in one layer may also influence pore pressure conditions in an adjacent layer that may otherwise be assumed to be fully drained. The behavior of geomaterials under partial drainage (or under intermittent cyclic events) depends on stress history, compressive or dilatant response (normally consolidated clay versus over-consolidated clay, loose-of-critical versus dense-of-critical sands) (Matasovic and Vucetic 1995). It has also been demonstrated that the cyclic strength of geomaterials can be a function of the assumed drainage conditions (Figure 10). An assessment of cyclic degradation should therefore be based on appropriate drainage assumptions informed by geomaterial properties and laboratory testing.

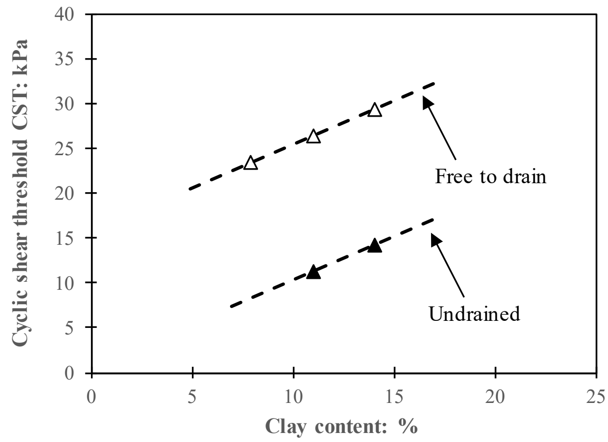


Figure 10: Effect of increasing the clay content on the cyclic shear stress threshold (adapted from Mamou et al. 2017)

5.5 Methods to Quantify Cyclic Degradation and Degradation Thresholds

Design standards such as ACP 61400-6 require that the risk for degradation of soil capacity and stiffness are evaluated as part of the foundation design. The standard acknowledges the complexity of such evaluations and historically onshore wind turbine geotechnical design in the US has been based on monotonic test data and the ACP 61400-6 accepted standards such as the “no-gapping” criterion.

Should a designer choose to perform a detailed cyclic degradation assessment, it is necessary to understand the potential for cyclic degradation to occur (i.e., what level of cyclic shear stress/strain would result in strength/stiffness degradation) and any corresponding consequence (i.e., softened strength/stiffness for the soil). Such an assessment requires a good understanding of the site-specific conditions (i.e., loading conditions, groundwater conditions, cyclic behavior of in situ soils, etc.).

Advanced laboratory testing program should be supported by sufficient laboratory testing to characterize the soil, which may include additional index testing to characterize the soil, additional consolidation testing to establish the soils stress history, and additional monotonic testing for normalization of test results. Testing for cohesive soils should be conducted on high-quality undisturbed samples to limit the impact of sample disturbance that can be fulfilled through techniques such as x-ray diffraction. Due to the difficulty in collecting high-quality samples cohesionless soil samples are often reconstituted to the in situ conditions (i.e., the field relative density).

The evaluation of cyclic degradation is significantly complicated by the turbine’s complex loading, and relatively simplified presentation in Markov matrix format. Throughout a typical turbine’s design life, it is anticipated to undergo billions of cycles and few details are provided beyond the load distribution and magnitude. Without established guidance or a better understanding of the loading history, the large number of cycles will overwhelm most laboratory testing programs and analytical, and numerical models.

In the case of cyclic laboratory testing, loading programs are often limited to a maximum of 500 to 1500 cycles of uniform magnitude, which may be more applicable to established offshore design storms that provide a distribution of loads with time and consist of durations between 1-8 hours, or earthquakes loading with an even more brief assumed loading history.

5.5.1 Strain-Controlled Testing

The response of geomaterials under cyclic loads largely depends on the cyclic stress-strain characteristics often defined by the decrease (degradation) of the soil stiffness (shear modulus G) after a number of cycles N of a cyclic shear strain amplitude γ_c. The number of cycles N affects the value of G/G_{max} due to the degradation of G with N, with the influence of N on stiffness being potentially significant, depending on the value of N, level of γ_c, and the type of soil.

When subjected to undrained cyclic loading, the overall stiffness and strength of the soil can degrade, causing the curve of G/G_{max} versus γ_c to consistently decrease as the number of cycles N increases (Figure 11).

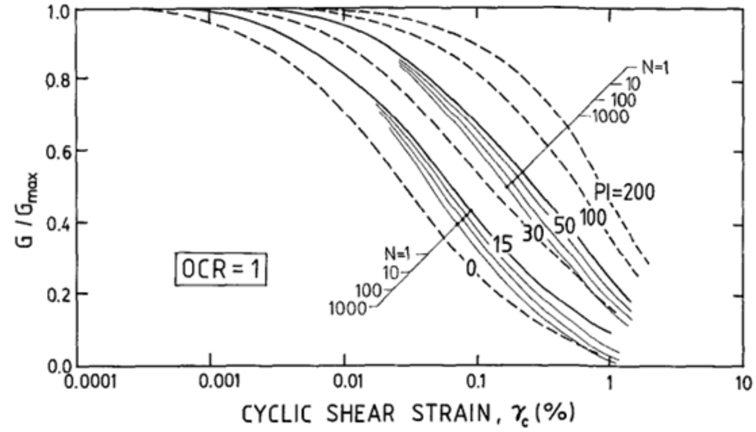


Figure 11: Shear modulus versus cyclic shear strain (Vucetic and Dobry 1991)

The cyclic stiffness degradation effect on G/G_{max} can be evaluated for a given cyclic shear strain amplitude using the concept of degradation index δ (Idriss et al. 1978). The degradation index δ describes the relative decrease of the secant shear modulus after N cycles (G_N) with respect to that in the first cycle (G_1) and is defined as:

$$\delta = G_N / G_1$$

For a given γ_c and as cyclic loading progresses, the effect of degradation accumulates and δ decreases monotonically with N. The rate of decrease of the degradation index δ with a number of cycles N can then be characterized by a single parameter called the degradation parameter t (Idriss et al. 1978), which is defined as:

$$\delta = N^{-t} \text{ or } t = \log \delta / \log N$$

The influence of N cycles on G for a given γ_c can then be evaluated directly in the laboratory using cyclic strain-controlled test results and used to characterize the geomaterial degradation for a series of γ_c caused by their corresponding load level at a given N cycles.

The degradation index δ varies with the number of cycles, shear strain γ, and OCR as demonstrated by testing results for Venezuelan North of Paria (VNP) clay (Vucetic and Dobry 1988) (Figure 12). Overall, the results indicate that degradation increases with the number of cycles and shear strain, but the degradation rate decreases with increasing OCR.

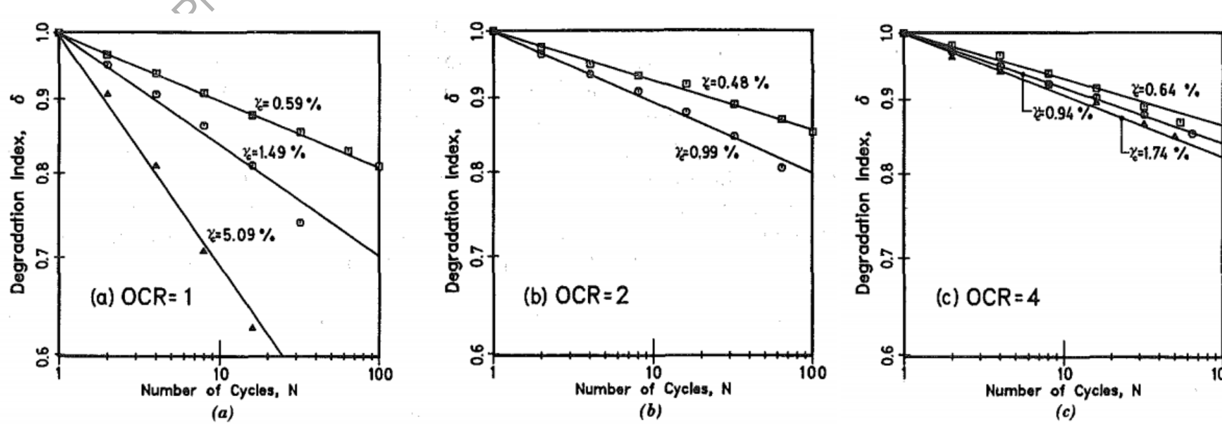


Figure 12: Degradation index, δ, versus the number of cycles and shear strain value γ and OCR of VNP clay based on constant-strain DSS test (Vucetic and Dobry 1988)

In the same study, Vucetic and Dobry (1988) also presented some findings (Figure 13) that showed the relationship between the degradation parameter t and the cyclic shear strain γ_c based on DSS testing and triaxial tests for 6 normally consolidated clays. The results also indicate that the degradation increases with increasing shear strain.

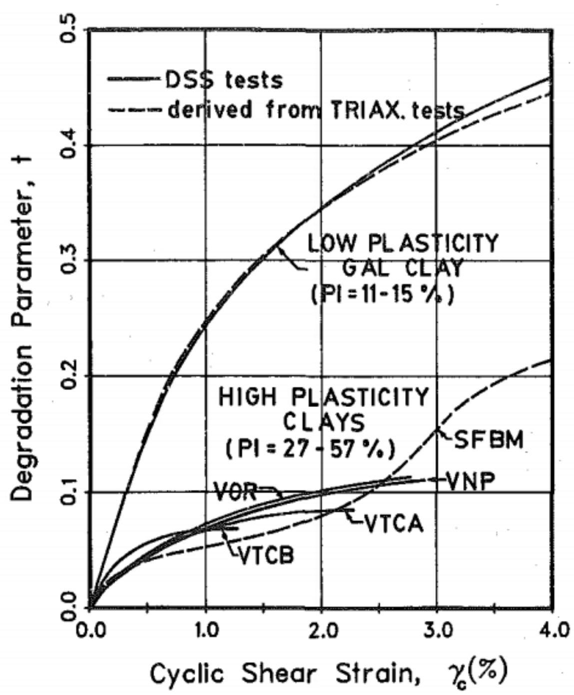


Figure 13: Degradation parameter *t* versus cyclic shear strain g_c (Vucetic and Dobry 1988)

An evaluation of a soil's cyclic thresholds as described in Section 5.2.2 may provide a basis for an evaluation of the potential for cyclic degradation to occur. Such evaluations may utilize a series of cyclic stain-controlled tests, or a series of multistage-cyclic stain-controlled tests. In the case of multistage testing care should be taken to avoid pre-shearing samples at small strains that may result in an increase in shear strength for subsequent load phases.

5.5.2 Stress-Controlled Testing

The cyclic contours approach utilizes a series of cyclic stress-controlled laboratory tests to representing a cyclic loading history as an equivalent monotonic model. Cyclic contours may be an approach to model the potential range of anticipated strength reduction and more details can be found in Andersen (2015).

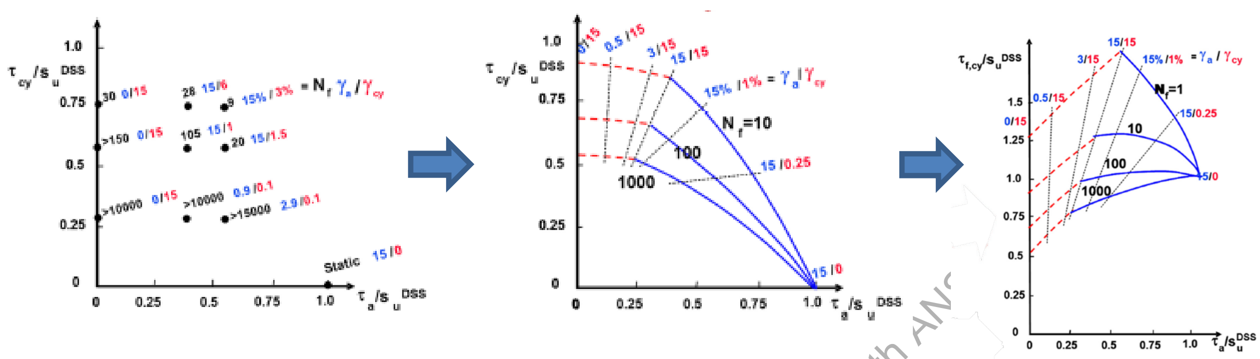
To evaluate cyclic behavior, a series of cyclic laboratory tests are conducted at various cyclic and average stresses. Tests are conducted until they achieve a specified strain failure criterion (typically τ_{cy} or $\tau_a = 15\%$), or a maximum number of cycles (typically 500–1500 cycles). The failure strain is specified or chosen as a shear strain at a level where the shear strain develops rapidly when the soil is subject to continued cyclic loading, which in some clays the failure strain can be smaller strain than 15%. For sands, contour lines for pore pressure development may be more relevant. In this case, excessive cumulative deformations may constitute a failure criterion.

The results of tests are used to construct a contour of cyclic strength (Figure 14), strain (Figure 15), and pore pressure accumulation (Figure 16). Contours should be generated based on OCR for clays, and the in situ relative density for sands. For guidance and to reduce the number of required tests for a site-specific evaluation laboratory testing programs are often based on comparison to existing cyclic contour databases such as the Drammen clay database.

Full anisotropic models may rely on a combination of triaxial extension, triaxial compression, and DSS to account for soil anisotropy. However, when using analysis software that is not capable of accounting for anisotropy, DSS tests are often used as an equivalent isotropic soil model (DNV-RP-C212).

The cyclic shear strength, $\tau_{f,cy}$, is the peak shear stress that can be mobilized during cyclic loading and is taken as:

$$\tau_{f,cy} = (\tau_a + \tau_{cy})_f$$



a) b) c)

Figure 14: Construction of cyclic strength diagrams: a) laboratory test results; b) cyclic stress contours; c) cyclic strength contours (Andersen 2015)

Cyclic laboratory tests are conducted with imposed undrained conditions where negative pore pressures are generated but this may not reflect the in situ condition and may degrade with a significant number of cycles.

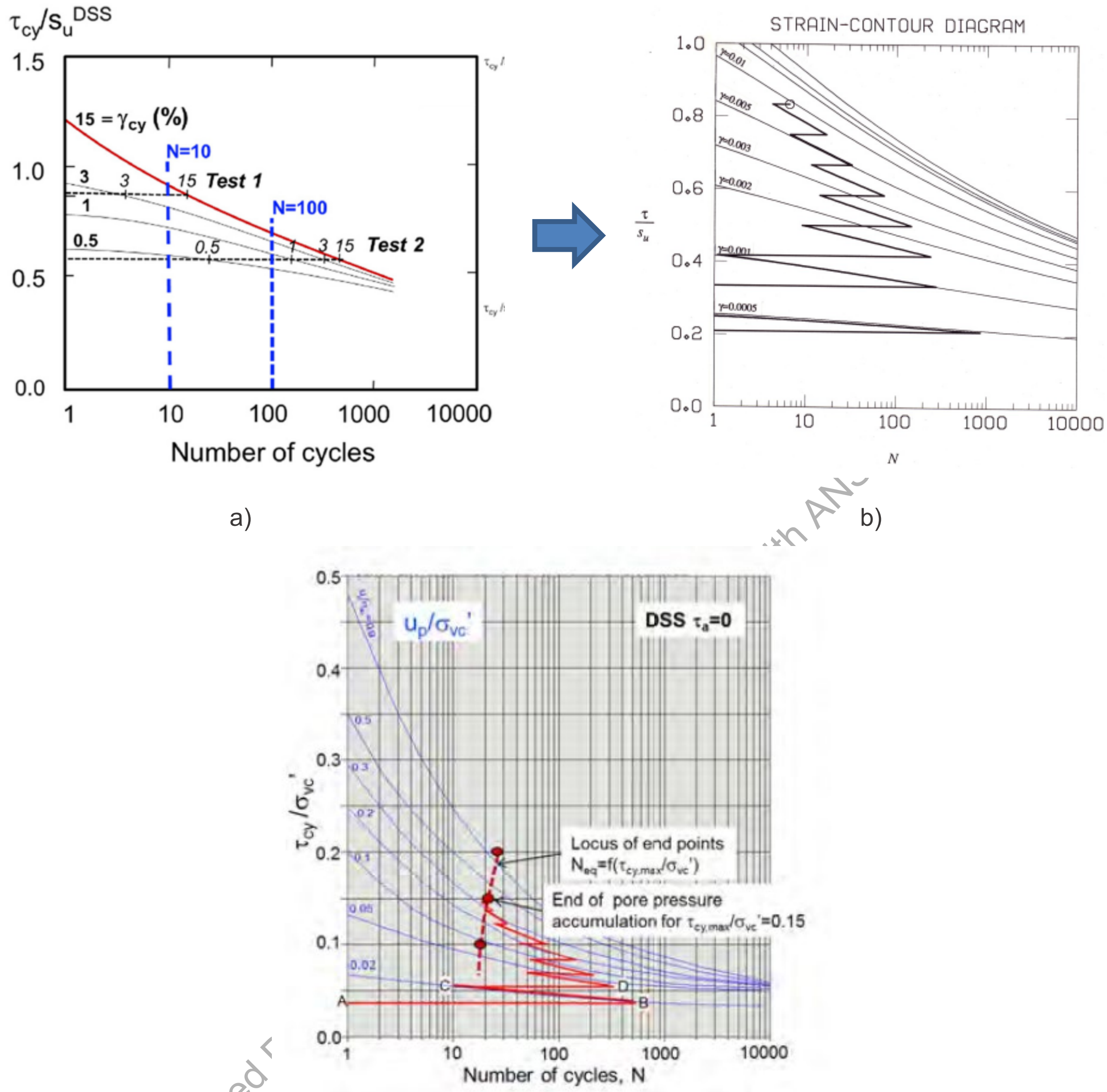
5.5.2.1 Equivalent Number of Cycles

As laboratory testing and the cyclic contours are based on a set of uniform loading conditions it becomes necessary to convert the structures loading history into an equivalent number of uniform cycles. Two approaches are proposed to estimate the equivalent number of cycles: a) strain accumulation and b) pore pressure accumulation. Both accumulation methods rely on extrapolating the results of a series at a constant average stress into strain and/or pore pressure versus number of cycles.

The strain accumulation method is generally considered the standard approach for the evaluation of fine-grained soils as it is challenging to accurately measure pore pressure in the laboratory. The pore pressure accumulation method is applicable to sands, where the dissipation of pore pressure can occur in parallel with pore pressure generation.

The strain and pore pressure accumulation contours are generated based on the results of cyclic tests at a constant average stress. The results are plotted in contours of normalized cyclic stress or pore pressure vs number of cycles and fit with contours of strain/pore pressure at different increments.

The load history is then normalized by the maximum anticipated load and scaled by a common factor on all stress amplitudes. Individual parcels of loads are then applied, generally from smallest to largest magnitude, with the assumption that the strain/pore pressure accumulation is carried over from one parcel to the next along the strain or pore pressure contours. The process is repeated with different scaling factors until the accumulation of strain and/or pore pressure at the end of all parcels equals the chosen failure criterion. It is important to note that the impact of scaling a turbines full design life (i.e., 25+ years) is unclear as published literature generally does not evaluate such a large set of loading data, and the model is typically utilized to evaluate much shorter design (i.e., 1- to 8-hour design storms).



c)

Figure 15: a) Laboratory test results; b) strain accumulation contour; c) pore pressure accumulation contour (Andersen 2015)

5.5.2.2 Pore Pressure Dissipation

State-of-the-art cyclic degradation models in sand may account for partially drained loading. Due to the difficulty in accurately measuring pore pressure generation in laboratory testing, modeling may be challenging or not possible in fine-grained material.

Partially drained behavior may be modeled the effect pore pressure dissipation by determining the amount of drainage that occurs simultaneously with the pore pressure generation during each parcel. Once the pore pressure dissipation has been determined it may be subtracted from the pore pressure accumulation.

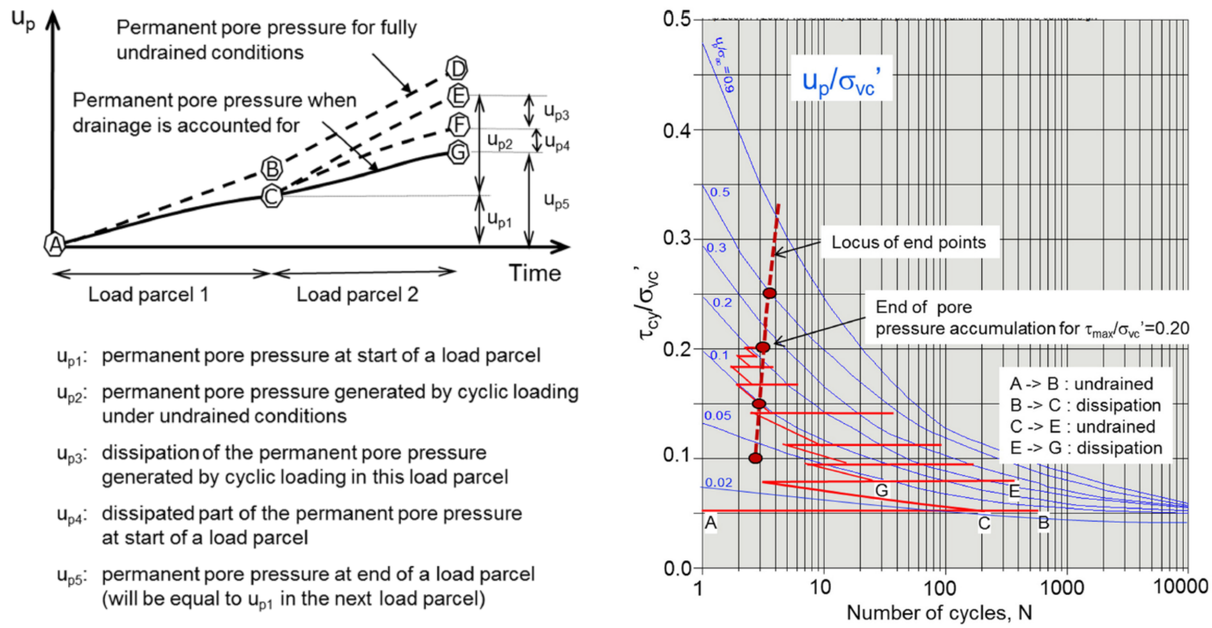


Figure 16: Pore pressure accumulation with partial drainage (Andersen 2015)

5.6 Loading Frequency

Loading frequency effects tend to diminish with increasing number of cycles and with decreasing cyclic deviatoric stress amplitude. The influence of frequency appears to be significant if relatively small numbers of cycles are considered. In general, for a given number of cycles, larger shear strains and excess pore pressures are generated at lower frequencies.

Research by Matsui et al. (1980) studied the effect of loading frequency on cyclic response of normally consolidated and over-consolidated clays, and found that for a given number of cycles, higher excess pore pressures and axial strains were generated at lower frequencies (Figure 17).

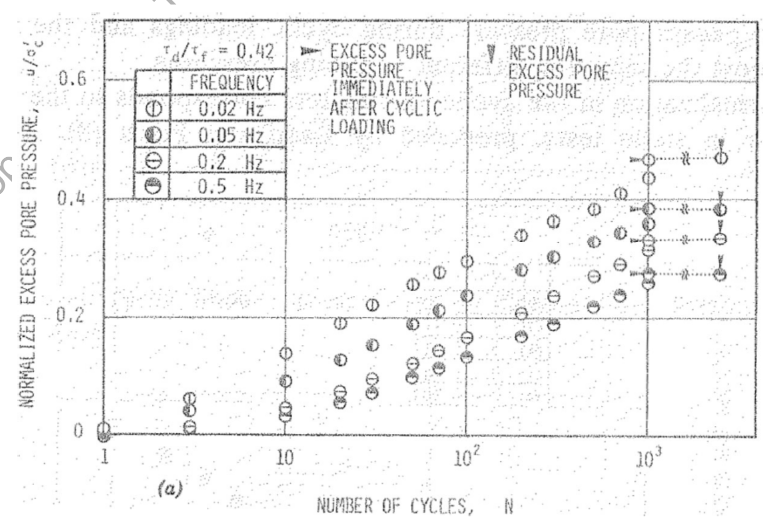


Figure 17: Effect of loading frequency on excess pore pressure (Matsui et al. 1980)

Zhou and Gong (2001) evaluated the effect of loading frequency on normally consolidated and over-consolidated clays and found that the degree of stiffness degradation is high for low loading frequencies and lower for higher frequencies. For loading frequencies less than 0.1 Hz, the testing indicated rapid stiffness degradation at relatively small number of cycles (Figure 18). However, it is clear from the results that nonlinear stiffness degradation would be expected for the typical range of loading frequencies for wind turbine foundations discussed below.

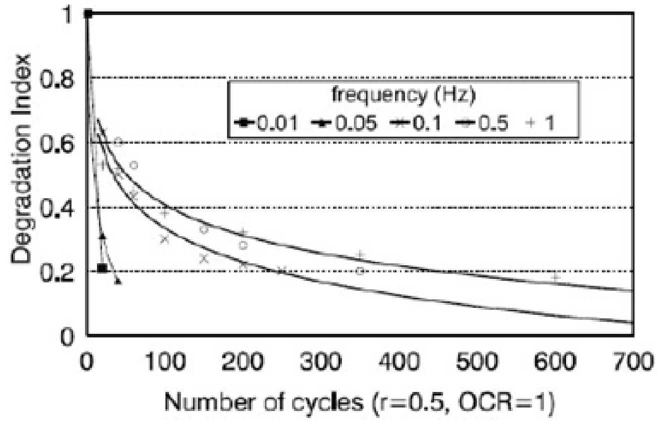


Figure 18: Effect of loading frequency on the degradation index (Zhou and Gong 2001)

Ansal and Erken (1989) also studied loading frequency effects and observed that the frequency effect in normally consolidated clay diminishes with an increasing number of load cycles and with decreasing shear stress amplitude.

For reference, natural frequency values for wind turbine structures can range approximately between 0.2 and 0.4 Hz, depending on hub height and rotor diameter, with larger size turbines (both hub height and rotor diameter) having lower natural frequencies and smaller ones having higher natural frequencies. The loading frequency effects in excess pore pressure accumulation have been measured and studied in undrained tests that do not allow for partial drainage. In field-scale conditions, during lower-frequency loading, more partial drainage of excess pore pressures would occur. Engineers should take into account the combined effect of partial drainage in reducing excess pore pressures and the rate effects of excess pore pressure accumulation in their designs.

6 Numerical Modeling for Foundations Supporting Wind Turbines

6.1 Introduction

Numerical modeling is widely used in many engineering applications for the assessment of complex problems and has gained wide usage in geotechnical engineering. In essence, numerical modeling is an idealization of the behavior of a system to capture specific aspects of the response. When appropriately applied, numerical modeling can produce realistic results of material behavior including deformation, stress distribution, dynamic response and offer insight into potential failure mechanisms of geotechnical structures. Various numerical modeling approaches are available in practice, including finite element method (FEM) modeling, finite difference method (FDM) modeling, discrete element method modeling, boundary element modeling, and meshless methods.

For geotechnical engineering applications, the use of numerical modeling requires competence in various topics including the theory behind the method used (FEM, FDM, etc.), the capabilities of the numerical platform and the constitutive models being applied, calibration of constitutive models, and a well-versed broader understanding of the principles of soil mechanics. The use of numerical modeling also requires careful interpretation and postprocessing of the results, as well as appreciating the limitations of the models and the software being utilized. As such, although numerical modeling is a useful tool for assessing geotechnical structures, it should be carefully applied with appropriate calibration of the input parameters and the results should be thoroughly peer reviewed to ensure that they are credible and consistent with expected behavior, as well as with estimates from simpler methods (e.g., closed-form solutions). Where possible, numerical modeling results may also be validated by comparing to field measurements obtained from instrumentation campaigns of in-place structures or scale models.

For the assessment of cyclic degradation of wind turbine foundations, numerical modeling offers a powerful tool to develop a better understanding of the foundation response under different loading conditions to capture realistic soil-structure interaction behavior. Compared to simplified solutions, numerical modeling can provide a better understanding of stress and strain distributions, time-dependent behavior, as well as potential failure mechanisms that should be considered. Numerical modeling also offers the ability to assess soil response considering that foundation behavior is a function of the soil continuum response (rather than discrete soil elements), which is important for most geotechnical problems that typically involve layered soils with anisotropic material properties. Since cyclic degradation of geomaterials is a function of the induced cyclic stress and strain level, numerical modeling may be utilized to obtain representative estimates of shear stress and strain levels within the materials and to facilitate assessment of potential degradation and residual strength and stiffness of the materials. Although various numerical modeling methods are available for geotechnical practice, FEM/finite element analysis (FEA) are the most used numerical modeling methods in the geotechnical assessment of wind turbine foundations. The numerical modeling discussions herein are therefore focused on FEM/FEA approaches although the considerations are generally applicable to other numerical modeling ones.

6.2 Constitutive Models

6.2.1 Choice of Constitutive Model

In numerical analysis, the relationship between the stresses and strains in the soil material are expressed with sets of mathematical relationships called constitutive models (Lade 2005). These relationships can vary in their complexity, accuracy and the number of parameters needed in defining the constitutive model. Hooke's law of linear, isotropic elasticity can be thought of as the simplest relationship regarding the stiffness of the soils (Brinkgreve 2005). Hooke's law combined with the Mohr-Coulomb strength criterion is known as the Mohr-Coulomb model and this model can be conceived as a first order model for soil behavior in general.

Over the last six decades, constitutive models have undergone considerable improvement, and several became available to address different geotechnical problems, varying from simple models with few parameters to more complex ones requiring several parameters. It should be noted that more complex constitutive models often are more capable of representing the real soil behavior, but they also require more parameters that might be challenging to obtain from conventional laboratory testing. Soils are a complex material that consist of a solid skeleton of grains that are in contact with each other, and voids filled with air and/or water shows a highly nonlinear and often anisotropic time-dependent behavior when subjected to stress and strain change (Lade 2005; Brinkgreve 2005). Brinkgreve (2005) summarizes the soil behavior aspects into six main categories as described below. Depending on the geotechnical problem at hand, some or all of these aspects can be more important, and the chosen constitutive model should be able to capture at least the most important aspects accurately. The modeler should be aware of the loading paths of interest to the problem at hand, the constitutive responses they activate, and most importantly the limitations and capabilities of the chosen constitutive model in capturing those. According to Brinkgreve (2005), the main soil behavior aspects are 1) groundwater and pore pressure, 2) soil stiffness, 3) plastic (irrecoverable) deformations, 4) soil strength, 5) time dependency, and 6) dilatancy.

When using numerical modeling for wind turbine foundations, the goal is to reproduce the soil-structure behavior expected in a real word problem. As such, the numerical modeler should make use of laboratory testing results to define the parameters required by the constitutive model selected. When lacking enough site-specific data, the numerical modeler often will rely on the broader body of published data, keeping in mind that site-specific soil behavior may not be reproduced by data in the available literature. The overall effort is best served, when the constitutive model at hand can already honor the broader body of data collected over the years and fundamental aspects of the response (e.g., suppressed dilatancy and increased contractiveness under increasing overburden stresses). For example, for an embankment built in soft clay, a critical state model might be able to reproduce the expected pressure-dependent soil strength, but for a wind turbine foundation a constitutive model that accounts for small-strain range and strain hardening is required. Additionally, if a dynamic analysis is performed to capture the cyclic nature of wind loading, the model should be able to properly perform at the small-strain range and reproduce pore pressure generation, cyclic mobility and/or cyclic strain softening.

6.2.2 Undrained and Drained Behavior

Due to cyclic loading, the soil might exhibit undrained, partially drained or drained behavior, based on its permeability and loading frequency and duration. For saturated low permeability soil subjected to short load duration, it can be anticipated that the soil will exhibit an undrained behavior (i.e., pore pressures are generated within the soil matrix). However, soil behavior is not only dependent on permeability but also on the rate of loading. If the rate of loading is slow compared to the soil permeability, it can be assumed that no significant excess pore pressures will be developed, but if the rate of loading is high compared to the soil permeability, excess pore pressures will develop due to undrained behavior. In an undrained analysis, as the volume cannot change since the water is not allowed to flow, the loading is transferred into the pore water leading to excess pore pressure generation. In a drained analysis, the water is free to flow through the voids, volume changes can occur, and the loading is transferred to the soil skeleton and no excess pore pressures develop (Lees 2016). In Figure 19, a flowchart illustrating the type of analysis that may be employed in numerical modeling is presented.

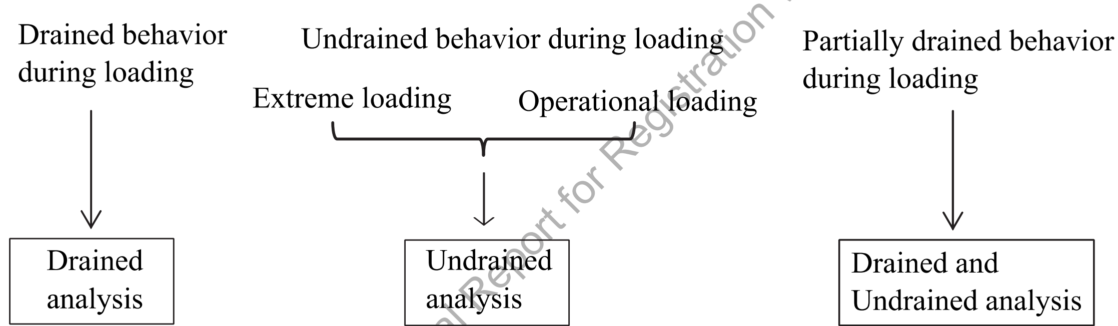


Figure 19: Selection of drained, undrained, or consolidation analyses (adapted from Lees 2016)

Under certain cyclic loading conditions, partially drained conditions might be encountered for some soil types (e.g., sands, silty sands). However, due to limitations in implementing this behavior in numerical analyses at the time of this report release, it is recommended that the numerical modelers assess the behavior under drained and undrained conditions and use the most conservative results.

Generally, there are three methods to simulate undrained soil behavior, not including consolidation analysis with a short time interval (as presented in Figure 20):

1. Effective stress analysis framework with effective parameters – use of effective stiffness and strength parameters
2. Effective stress analysis framework with undrained strength parameters – use of effective stiffness parameters and undrained strength parameters
3. Total stress analysis framework – use of undrained stiffness and strength parameters

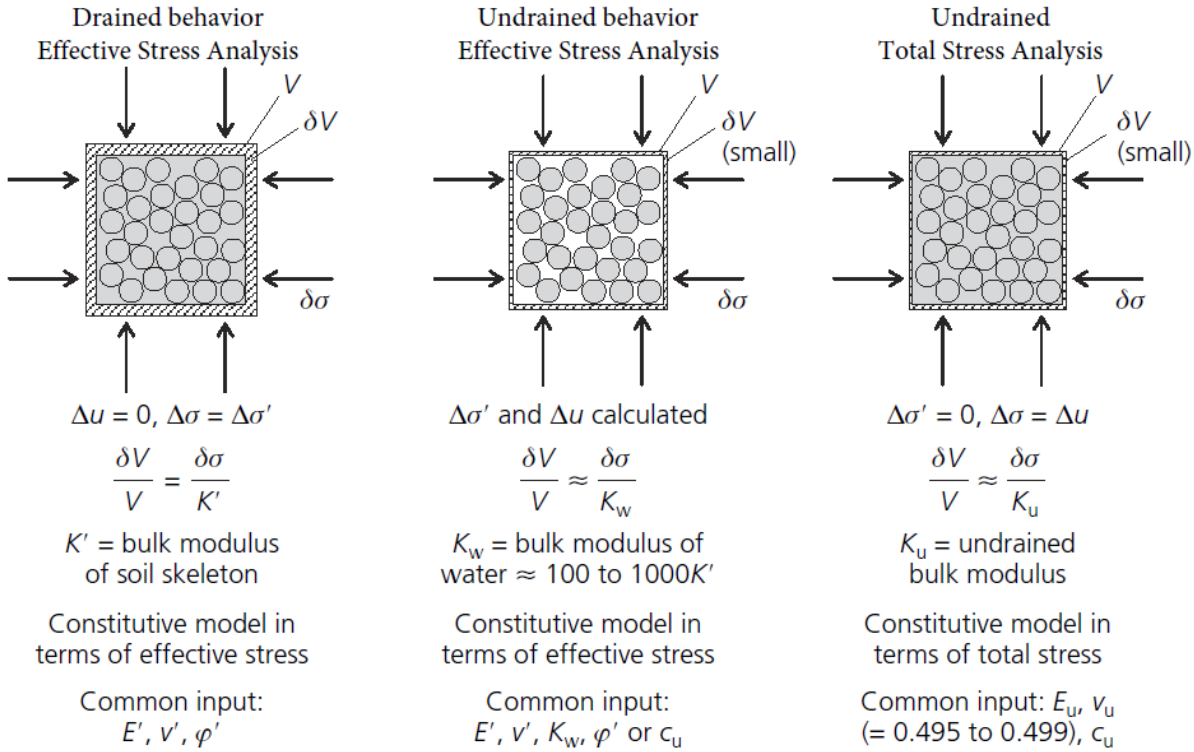


Figure 20: Methods to simulate drained and undrained soil behavior (adapted from Lees 2016)

The undrained behavior modeled by using an effective stress analysis framework combined with effective strength parameters uses a high value of bulk for the pore water, which is added into the stiffness of the soil so that volumetric strains are small and excess pore pressures are generated. This method has the advantage of providing outputs of excess pore pressure, but it is only likely to be reasonably accurate when using appropriate advanced constitutive models. As the undrained shear strength is not an input parameter, the resulting mobilized shear strength must be checked against the site-specific soil data.

Alternatively, the undrained behavior can be modeled by using an effective stress analysis framework with undrained strength parameters. As the undrained shear strength is an input parameter, it removes the risk of overpredicting the soil shear strength, as when using effective strength parameters. However, for this calculation method the excess pore pressure predictions may become highly inaccurate.

The last method to model the undrained behavior is by using a total stress analysis framework, in which undrained stiffness and strength parameters are used such as undrained Young modulus, undrained Poisson's ratio, and undrained shear strength. Total stress analysis is useful for basic constitutive models where unrealistic conditions might otherwise be predicted with the effective stress analysis approach. However, it is not suited for advanced soil models, except those formulated in terms of total stress framework such as for example the NGI-ADP constitutive model. Additionally, this framework does not give a prediction of pore pressures and thus there is no distinction between effective and total stresses.

Each method has its own advantages and shortcomings, so the numerical modeler should be familiar with the methods used to simulate the soil-structure interaction and the selection and calibration of an appropriate constitutive model. One of the most notable accidents due to misuse of numerical modeling was the collapse of Nicoll Highway in Singapore on April 2004, as the supporting geomaterial, consisting of under-consolidated marine clay, was modeled by using the effective stress analysis framework and effective parameters (E', c'-phi'ϕ') to assess its undrained behavior. The root cause analysis suggests that the numerical model overestimated the undrained shear strength of the marine clay deposit by 18% as one of the causes for the diaphragm wall collapse (Endicott 2015; Agaiby and Ahmed 2016).

6.3 Geometry, Boundary Conditions, and Interfaces

6.3.1 Introduction

Numerical analysis and simulation have a wide range of engineering applications, and consequently, there is an abundance of literature covering theory and practice. This section presents the geometry model principles of FEA, including converting the physical domain to a virtual/numerical domain, meshing, element formulation, and boundary conditions for onshore foundations supporting wind turbines.

6.3.2 Converting the Physical Domain to Virtual Domain for Foundations

6.3.2.1 General

A model representative of the geometry of the structure to be analyzed is needed to proceed with FEA for the foundations that support wind turbines. During geometry modeling, it is necessary to define the physical domain (i.e., physical geometry) for the foundations, and then convert the physical domain to the virtual (or numerical) domain to be able to perform numerical analysis. The virtual domain must therefore be a close representation of the physical problem in dimensions and orientation (Okereke and Keates 2018). Modern commercial numerical modeling software packages have built-in virtual domain generating modules to convert the physical geometry of the foundations to their numerical counterparts. Although it may not be possible to replicate the exact geometry of a real foundation, the model should be sufficiently representative, including the salient features of the domain under investigation, to reasonably capture the realistic response and minimize geometric errors.

6.3.2.2 Mesh Generation

One feature of the FEA for geotechnical engineering is that the soil and rock mass need to be modeled extending well beyond any structure of interest within the soil domain. This raises the issue about how far or how big the domain of soil and rock mass should be used to avoid boundary effects that may affect the simulation accuracy, including the stress/strain, deformations for foundations, and wave reflections for seismic analysis, etc. Although there are various recommendations in the literature, an investigation of the effect of boundary conditions may take the following into consideration:

1. The depth and the width/length of the soil/rock domain should be of sufficient size to minimize boundary distortions that may affect the simulation accuracy (Figure 21).
2. Validation of stress distributions using commonly used closed-form solutions, e.g., the Boussinesq method for pressure dissipation can be used to interpret the influencing depth of the foundation pressure.

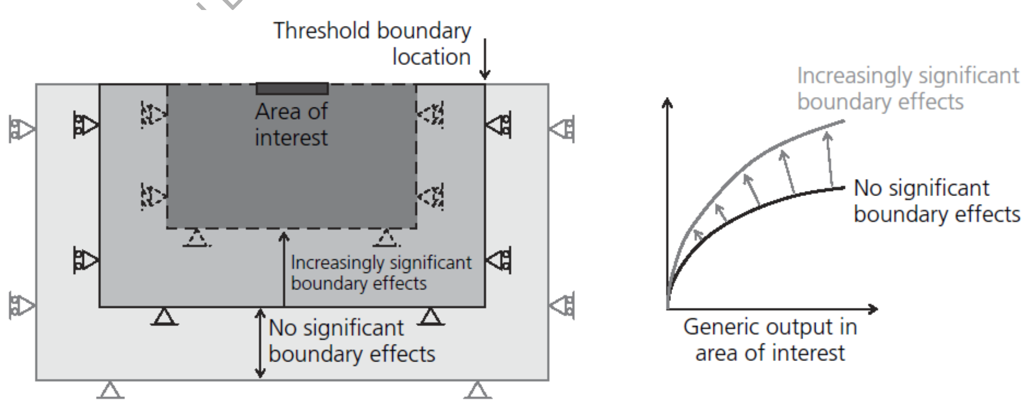


Figure 21: Boundaries size and effects (Lees 2016)

The mesh generation process replaces the geometry by equivalent element mesh, and the mesh is composed of small regions of elements (Potts and Zdravkovic 1999). Element formulation is important as it deals with a derivation that is needed to deduce displacements acting on a body that is discretized by finite elements. Different formulations apply for different types of elements, and derivation of such displacements is dependent on shape functions (e.g., Okereke and Keates 2018).

The FEM solver performs the analysis element by element, and the convergence of the numerical solution has to be reached for each element to ensure the solution is acceptably accurate. To control the process, the convergence limit is usually set to as low as it can be, but it can be changed by the user. The accuracy of the solution is dependent on the number of elements composing the mesh, as illustrated in Figure 22.

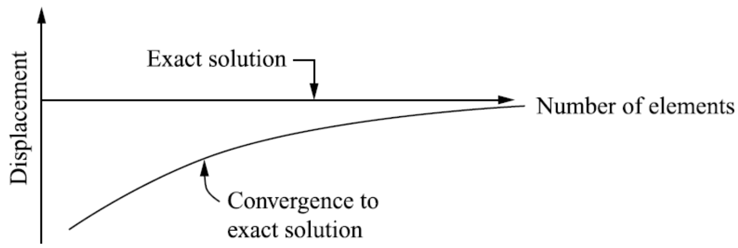


Figure 22: The relationship between number of elements and solution accuracy (Logan 2007)

6.3.3 Loads and Boundary Conditions

In performing FEA for wind turbine foundations, appropriate boundary conditions should be applied to the model. The boundary conditions represent the constraints imposed on the behavior and conditions of the nodes at the boundaries of the virtual domain. Since many engineering solutions are governed by ordinary and partial differential equations, the solutions to these equations depend on the constraints (i.e., the boundary conditions) as different boundary conditions will lead to different solutions. Thus, the boundary conditions must be well defined and satisfied and fitted into governing the differential equations. Considering the definitions and the functions of the boundary conditions, applied loads essentially are parts of the boundary conditions though it is common to distinguish them from the model boundary conditions in the application of the FEA.

The applied boundary conditions should not change the configuration of the structure of interest but provide support reactions whereas loads will cause the reactions. Below is a chart that classifies the types of loads commonly used in FEA for wind turbine foundation assessment (Figure 23):

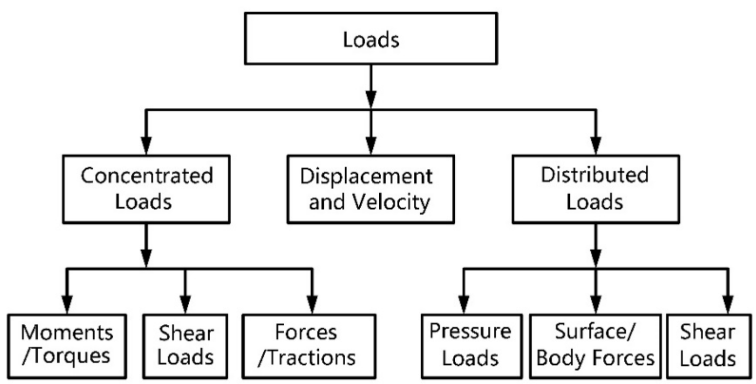


Figure 23: Types of loads for FEA

Wind turbine loads for foundation engineering analysis might be applied by resorting to different modeling techniques such as the use of surface pressures, point loads, by using a rigid body[1], or any other modeling technique. The numerical modeler should ensure the loads applied to the model are properly transferred to the foundation and soil domain, and the results obtained are appropriate.

6.3.4 Interface Modeling

Interface elements are commonly used in numerical analyses to account for the interaction along the surface of contact of two different materials (e.g., a structural member and its neighboring soil). The interaction includes static and cyclic actions. Different numerical analysis software employs different

[1] If available and if the foundation is expected to behave in such manner.

approaches and different levels of complexity to interface modeling, but the element formulation and working mechanism is similar.

For wind turbine analyses, the following conditions should be met at a minimum:

1. The soil-structure interaction should be modeled using interfaces.
2. Interfaces should allow relative movement and separation of the soil and structural members.
3. The interface material properties should reflect the reality (e.g., if the soil material is known to be degrading under cyclic loading, the interface material properties should be able to capture that behavior).

6.3.5 Geometry Idealization

Geotechnical problems can be modeled in two dimensions (2D) or three dimensions (3D). In a 2D analysis the geotechnical problem can be idealized as a plane strain or axisymmetric.

A 3D model involves the analysis of a geotechnical problem in a true 3D space (Figure 24, panel a), with displacements, strain and stresses occurring in three dimensions. For geotechnical problems symmetric in both geometry and loading, a 3D model can be idealized as a half model to save computational time (Figure 24, panel b). Caution should however be used when performing such simplifications, since both loading, and boundary conditions need to be examined such that the foundation behavior is representative of the full 3D model.

A plane strain model (Figure 24, panel c) involves the analysis of a plane/vertical slice, with the strain and displacement of the "third dimension" (i.e., axis perpendicular to the plane) assumed to be zero. As such, the strains can only occur in directions within the plane and are independent of the out-of-plane direction. The plane strain modeling technique is suited for geotechnical problems with a uniform and continuous cross-section and loading scheme over a certain length perpendicular to the cross-section. It can also be useful for gaining an understanding of distribution of stresses and strains with depth, as well as for potential failure mechanisms.

An axisymmetric model (Figure 24, panel e) involves the analysis of a plane, vertical section, except one vertical side of the plane is the axis about which the site has rotational symmetry. The horizontal axis is the radius from the axis of symmetry, and the strain perpendicular to the plane and in the circumferential or hoop direction is assumed to be zero; hence, displacement, strain, and shear stress can only occur in the analysis plane.

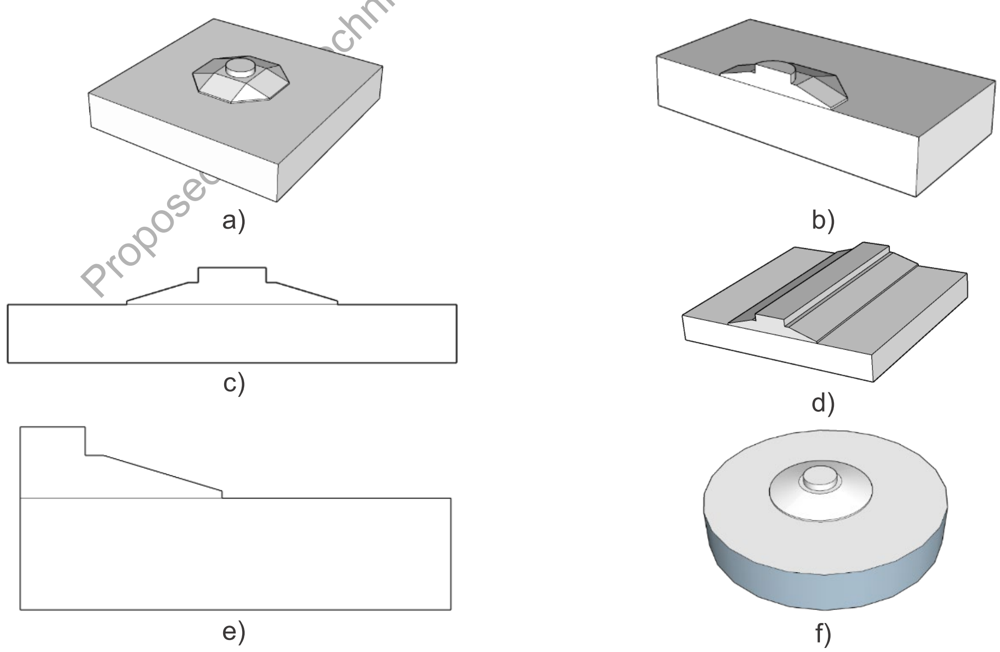


Figure 24: Octagonal foundation modeling: a) full 3D model; b) half 3D model; c) plane strain model; d) 3D idealization of a plane strain model; e) axisymmetric model; f) 3D idealization of an axisymmetric model

Although the use of 2D idealizations is useful and appealing due to its simplicity and computational efficiency (i.e., time), some geotechnical problems require a full 3D analysis. Historically, it was not practical to perform 3D modeling in standard practice due to the amount of time required to run such an analysis. However, with parallel processing technology being readily available to practitioners, the use of 3D modeling is becoming more economical and practical, and thus is the preferable option to study gapping and cyclic degradation of the subgrade material.

While it is possible to model a wind turbine foundation by using an axisymmetric model for relatively simple validations (e.g., stress or strain distribution with depth), the simplicity of the model has limitations in capturing a detailed picture of foundation behavior and is therefore not sufficient for assessing gapping and its effects. The application of plane strain analysis is feasible for some aspects of the foundation, but the approach has limitations in capturing 3D effects, which makes it less suitable for studying gapping and cyclic degradation when compared with a 3D model.

6.4 Modeling of Cyclic Behavior

Modeling cyclic behavior of the soils in numerical models is complex and requires a combination of different phenomena during cyclic loading, such as modulus degradation, damping, pore pressure buildup, and strength degradation.

6.4.1 Modulus Reduction

Soils exhibit nonlinear behavior under shear loading. The secant modulus decreases with increasing shear strain and the shear modulus at small strains is typically referred to as G_{max}. The relationship between shear modulus and strain amplitude is typically characterized by a normalized modulus reduction curve as shown in Figure 4. To model the cyclic behavior, a soil constitutive model capable of capturing the modulus and strength degradation should be implemented. There are various available software with built-in or user defined soil constitutive models that can model the modulus reduction with increasing shear strain. Some of these models are the Hardening soil model with small-strain stiffness, and the UBCSAND (Beaty and Byrne 2011), NorSand (Jefferies and Shuttle 2012), PM4Sand (Boulanger and Ziotopoulou 2023), PM4Silt (Boulanger et al. 2022), and PDMY03 (Khosravifar et al. 2018) models. The modeler should be aware of the applicability and limitations of the constitutive model selected for the assessment.

6.4.2 Damping

Soils exhibit energy dissipation upon cyclic loading. Generally, damping in soils is classified as hysteretic damping and radiation damping. Hysteretic damping is an internal damping mechanism of the soils where energy is dissipated due to the friction of the soil elements and is thus independent of frequency. Hysteretic damping is measured in the form of damping ratio, which is defined as the ratio of the dissipated energy to the stored energy in the soil element per cycle of loading. The hysteretic behavior is illustrated as follows (Figure 25):

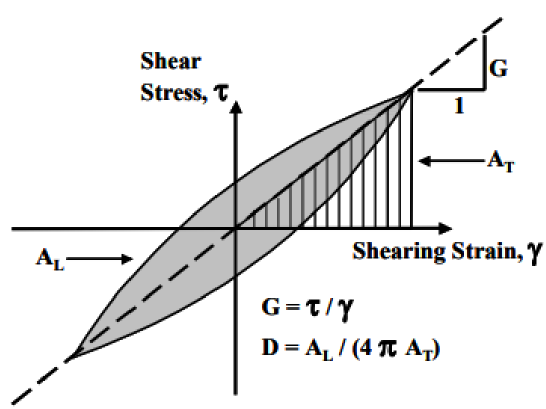


Figure 25: Typical hysteretic loop during cyclic loading (Darendeli 2001)

The nonlinearity in the stress-strain relationship results in an increase in energy dissipation and, therefore, an increase in material damping ratio with increasing strain amplitude as presented in Figure 26 (Darendeli 2001). Material damping ratio at small strains (in the linear range) is referred to as small-strain material damping ratio, D_{min}. The material damping ratio a can be measured in the laboratory from resonant column or torsional shear tests and cyclic triaxial tests. The damping ratio is mostly affected by effective stress, number of cycles, and plasticity (Darendeli 2001).

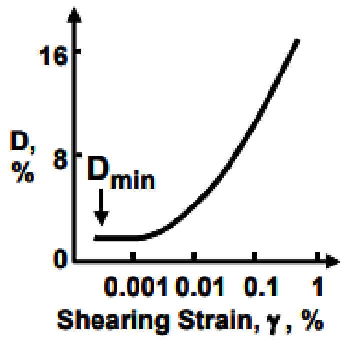


Figure 26: Damping ratio with increasing shear strain (Darendeli 2001)

6.4.3 Pore Pressure Generation

Saturated soils subject to cyclic loading under undrained conditions are prevented from volumetric changes due to the low compressibility of water compared to the soil skeleton. Therefore, part of the normal stresses carried by the soil skeleton will thus be transferred to the pore water, and the effective stresses in the soil will decrease accordingly. A detailed discussion of the pore pressure generation under cycling loading is included in Section 5.4.

6.4.4 Strength Reduction

Cyclic softening of clays is commonly understood as the reduction in soil stiffness and strength due to repeated cyclic loading. A constitutive model that can include the cyclic shear strength reduction is the preferred method when modeling cyclic loading of wind turbines. However, it may not be feasible to apply the actual number and magnitude load of cycles that the wind turbines experience during their design life in the model. Therefore, some simplifications can be made in the load cycles and magnitudes if it is shown that the results would not be significantly affected by this simplification. A detailed discussion of cyclic shear strength degradation is included in Section 5.3.

6.4.5 Feasibility of Dynamic Analysis and Simplifications

The number of load cycles throughout the design life of a wind turbine can be millions of cycles. These loads are provided by the turbine manufacturers in a format that is called Markov matrix that provides load cycle counts at different mean values and load amplitudes expected during the design life. The loads are not available as time histories and it is therefore not feasible to perform a full cyclic loading analysis in numerical modeling. Moreover, the foundations are generally large, which requires large geometries to be defined in the model and can lead to very long calculation times in a dynamic model. It is therefore necessary to obtain representative cyclic loading information for use in assessment of cyclic degradation as discussed in Section 7 herein. Laboratory test results may also be used to determine if there is a threshold number of cycles for a specific load or shear strain level beyond which the number of cycles do not change the soil behavior significantly. The results of the soil testing would then be incorporated into the dynamic analysis for assessment of potential degradation.

A static analysis may also be performed to assess the gapping percentage, expected strain levels in the foundation support material and the depth within which degradation may be expected to occur. To perform such analysis, laboratory testing should be used to characterize cyclic soil behavior and prevent subjective judgment. The static analysis can be performed in an iterative manner where the stiffness and strength parameters can be updated based on the obtained strain levels and the cyclic laboratory tests results.

6.5 Model Calibration with Foundation Monitoring

When performing assessment of in-place behavior of a foundation, numerical models can be calibrated based on acquired data such as turbine operating frequencies typically recorded within the supervisory control and data acquisition (SCADA) system for the turbine. Detailed instrumentation plans are discussed in Section 10 herein if the goal is to monitor foundation-soil interaction and cyclic degradation behavior during the operation of the turbine. The foundation instrumentation includes installing a series of sensors (i.e., stain gages, earth pressure cells, soil extensometers, tiltmeters, etc.) within/under the foundation and at tower base. System frequency data are also collected and used to estimate foundation stiffness and calibrate measurements from other sensors. The measurements from the foundation monitoring program, after thorough interpretation and verification, can be used to refine the constitutive model of foundation soil behavior and numerical analysis input settings/parameters and calibrate the output results of numerical analysis model as shown in Table 1.

Table 1: Calibration of Numerical Analysis Results Based on Foundation Monitoring

FEM Results	Monitoring Scheme
Foundation rotation	• Tiltmeters or inclinometers attached to foundation and tower
Foundation settlement	• Monument survey, settlement gages, extensometers, or inclinometers
Foundation rotational stiffness	• Measurement from tiltmeters or inclinometers attached to foundation and tower • Strain gages installed in walls of tower base • Frequency measurements from the turbine SCADA system or a condition monitoring system
Foundation bearing pressure	• Earth pressure cells installed at the interface between the foundation base and the top of subgrade • Strain gages installed at the interface that can back-calculate earth pressures
Soil strain levels	• Soil strain gages or extensometers installed at various depths within the influence zone
Excess pore water pressure	• Pore water pressure transducers installed at various depths within the influence zone
Extension of foundation gapping	• Earth pressure cells installed at the interface between the foundation base and the top of subgrade • Strain gages installed at the interface that can back-calculate earth pressures

6.6 Postprocessing and Results Validation

All numerical models need to be validated to check how accurately they represent reality (Oberkampf et al. 2002). There are differences/discrepancies between the reality and the numerical model due to several reasons. Brinkgreve et al. (2013) explain the sources of these differences as simplifications, modeling errors, constitutive models, uncertainties, software and hardware issues, and misinterpretation of results. There are several methods to validate numerical models. These methods generally are related with individual components of the whole numerical model and should be used in conjunction. One of these validation methods is simulating soil behavior measured in laboratory tests using single element models. Also, some in situ tests can be simulated, although not with a single element method. Another validation method is checking the boundary conditions and mesh size to ensure that the model results are not affected by the choices made by the user. Initial conditions of the stresses and pore pressure at the real site can also validated using measurements such as pore pressure distribution to make sure that the model initial conditions reflect the real-world conditions. A fundamental approach for verification and validating the results is to check the accuracy of the results. This can be achieved by using measurements from the real project if available or large-scale tests (e.g., centrifuge model tests), design charts, closed-form solutions, experience from similar problems, a simpler model (using a 2D model to validate a 3D model for example), and benchmarking.

6.7 Limitations

Numerical analysis is a powerful tool to model the behavior of a geosystem under prescribed loading conditions. With the advent of computers and availability of a wide range of commercial software, numerical analysis is very often the default choice for solving a complex geomaterial-structure interaction. This method too has its limitations, and therefore could lead to incorrect results, sometimes with costly consequences, if not applied correctly. Limitations of numerical analysis methods can be listed into three broad categories for geotechnical engineering: 1) limitations inherent to the principles and theories, 2) limitations imparted by users and applications, and 3) limitations in the data available to define the parameters that allow to model the soil behavior. Accuracy and applicability of numerical analysis methods can be limited by the following:

- Constitutive models used in the numerical analysis: Geomaterials exhibit a complex and heterogenous interaction under complex loading conditions. Constitutive models are mathematical representations with inherent assumptions and simplifications of such behaviors. The constitutive models chosen for dynamic numerical analyses of cyclic degradation problems should be able to capture small-strain behavior, loading-unloading cycles, and pore pressure generation. In other words, model validation should be undertaken for the constitutive model used. In addition, the parameters used in the model should be adequate and representative of the soil conditions at the site.
- Numerical analysis models: Similar to constitutive models, simplification and assumptions are made in modeling geomaterial and structural interaction such as boundary conditions, load applications, etc. Incorrect assumptions may result in inaccurate solutions. Simplified assumptions during numerical modeling also may also lead to solutions that are only partially representative of the in situ conditions.
- Errors in application and interpretation of the results: Interpretation of the numerical analysis results not only requires an advance level understanding of the underlying principles of the geomaterial behavior, loading conditions and boundary conditions, but also proficiency in application tools such as computer software and package utilized to perform such analysis. Results of numerical analysis performed by inexperienced user without any expertise in this domain often leads to false interpretation and implementations.
- Numerical analysis tools require high computational resources and advanced level of proficiency in this area. For complex problems, such as the dynamic analysis of geomaterial-structural behavior (such as for wind turbine foundations), advanced laboratory testing also needs to be performed to validate/calibrate constitutive model parameters.

6.8 Conclusion

Numerical modeling can be a useful tool in assessing the soil-structure interaction of wind turbines under static or cyclic loading. Similar to any numerical modeling in geotechnical engineering, analysis of wind turbine foundations using numerical models require a comprehensive field and laboratory investigation, selection of adequate soil model that is capable of reflecting the real soil behavior under the given conditions, application of the correct loading characteristics, validation of the results and awareness of the limitations of the used model. It should be noted that due to the foundation sizes and the number of loading cycles in wind turbine analysis, several simplifications might be required. Therefore, the user should be aware of the implications of the simplifications and whether the simplifications are affecting the results significantly.

7 Loading Considerations

7.1 General

This section presents information on foundation loads that could be considered for analysis of cyclic degradation of foundation support materials and foundation gapping during turbine operation. Given that the number of loading cycles affects the amount of degradation, a comprehensive but practical approach should be used to consider the necessary load levels and corresponding loading cycles for numerical modeling and advanced laboratory testing. Foundation load information from the original equipment manufacturer (OEM) provides data in the form of Markov matrices that give the number of load cycles for each range of load levels and can be condensed for a comprehensive and practical analysis.

7.2 Foundation Load Information

Foundation loads are provided by the OEM of the turbine and developed for the actual turbine being supported and site-specific wind conditions. The foundation load documents can also include requirements by the OEM for the foundation performance. The following shows the most frequently used coordinate system to define the forces and moments in the tower and foundation (Figure 27).

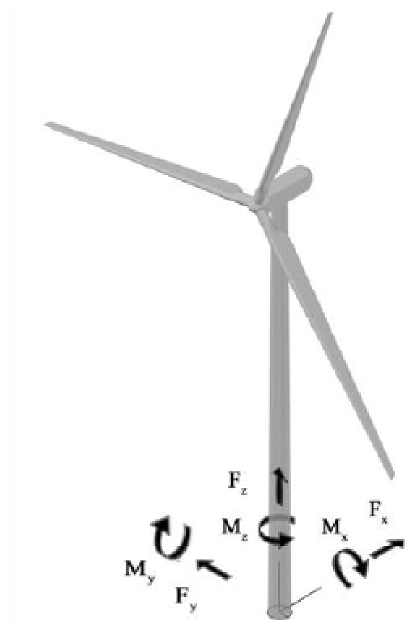


Figure 27: Coordinate system for forces and moments (ASCE/AWEA RP2011)

The forces and moments in the foundation load documents generally do not account for wind direction. Most sites have a dominant wind direction, and it appears that the most significant forces and moments act fairly close toward the same direction; thus, for simplicity it seems reasonable to assume that these significant forces and moments generally act in the same direction. However, the wind conditions at each site should be individually evaluated to verify the most reasonable assumption.

IEC 61400-1 presents a general overview of the external conditions considered in the design of a wind turbine and design checks for compliance with specific external conditions, with these generally separated in normal conditions and extreme conditions. Extreme conditions represent rare external design conditions and cause ultimate failure in the foundation or tower; therefore, these are considered outside the scope for assessing long-term cyclic degradation. Normal conditions are those that occur during normal operation of a wind turbine, resulting in serviceability load levels, which translates into a series of load levels occurring for a number of cycles, thus can be used to assess cyclic degradation and verify foundation stiffness.

7.3 Definition and Description

7.3.1 Serviceability Loads

ACP 61400-6 and DNV-ST-0126 define the normal load conditions as the serviceability load levels S1, S2, and S3 and are described as follows:

- S1 load level is the characteristic load or normal extreme load.
- S2 load with probability of 10^{-4} equal to 0.01 percentile values, equivalent to DLC 1.1 in IEC 61400-1, period of exposure to higher loads of 0.87 hours per year.
- S3 load with probability of 10^{-2} equal to 1 percentile values, quasi-permanent load level, no lift-off (gapping) load, period of exposure to higher loads of 87 hours per year.

To illustrate the definition of each serviceability load level, the Markov matrices can be used to graph the load level (mean and range value) versus percentile exceeded (or number of accumulated cycles) and each serviceability load level can be plotted into this fatigue load representation (Figure 28).

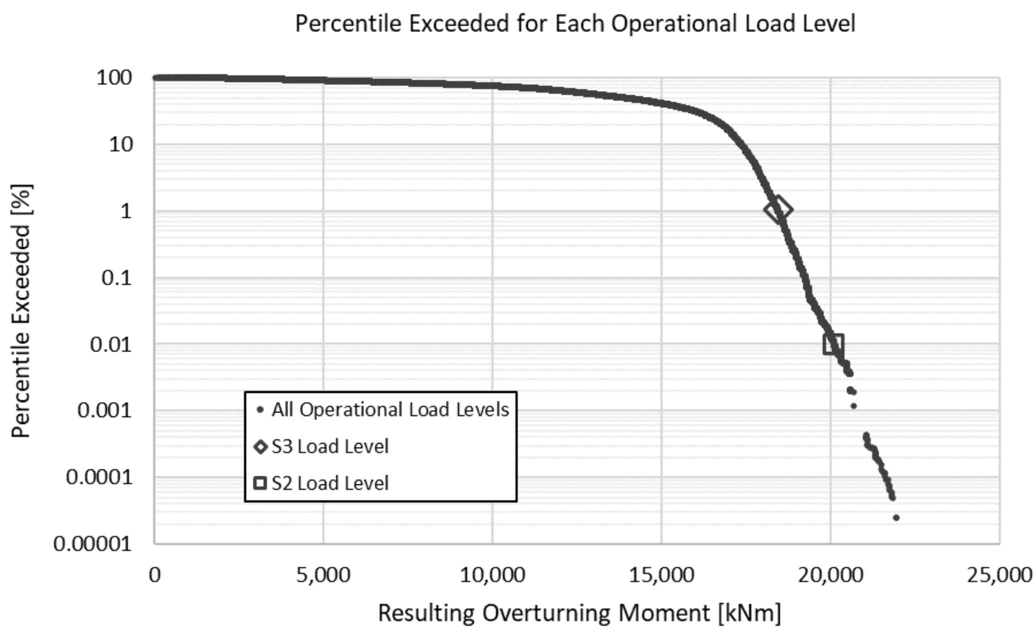


Figure 28: Illustration of serviceability load level using fatigue load data available in Markov matrix

7.3.2 Markov Matrix

The fatigue load spectrum is often presented in the form of Markov matrix. The fatigue loads are grouped into a number of similar bins (usually a few hundreds to tens of thousands of bins). Each bin includes a load level (which is the mean expected load), load range (which is the fluctuation of the load about the load level), and number of cycles (which is the expected number of occurrences of the load in the considered service life). Markov matrix is a collection of the bins such each row of the matrix represents one bin.

7.4 Approach for Using Load Data in Markov Matrix

Cyclic degradation of geomaterial is influenced by strain level and number of loading cycles. Load level, range of load fluctuation, number of cycles, and sequence of loading are among the parameters that could impact the rate of degradation. Markov matrix includes the estimated loads together with load ranges and number of cycles that are expected during the turbine lifetime. However, information on the sequence of loading is not included in Markov matrix. Therefore, if conservative but reasonable assumptions are made on the sequence of loading, Markov matrix can be used as load input for cyclic degradation assessment of geomaterial for wind turbine foundations.

A typical Markov could contain a few hundreds to tens of thousands of load bins (top graph in Figure 29), with each bin consisting of load mean, its range and corresponding number of cycles. Therefore, depending on the method of analysis, it may not be practical to conduct the analysis for all the load bins. The following two approaches can be used to reduce Markov matrix bins to a degree that is practically manageable for analysis.

7.4.1 Condensing the Markov Matrix

Condensing a Markov can be performed by identifying similar load bins and combining them to generate larger bins. Both load mean and load range should be considered in deciding whether the bins are similar and can be combined. The maximum and minimum load at each bin that are (load level) ± (half of load range) can be compared by setting a tolerance. The bins that fall within the tolerance can be combined. The combined bin can be created by averaging the load levels and load ranges separately and adding the number of cycles of each individual bin (bottom graph in Figure 29).

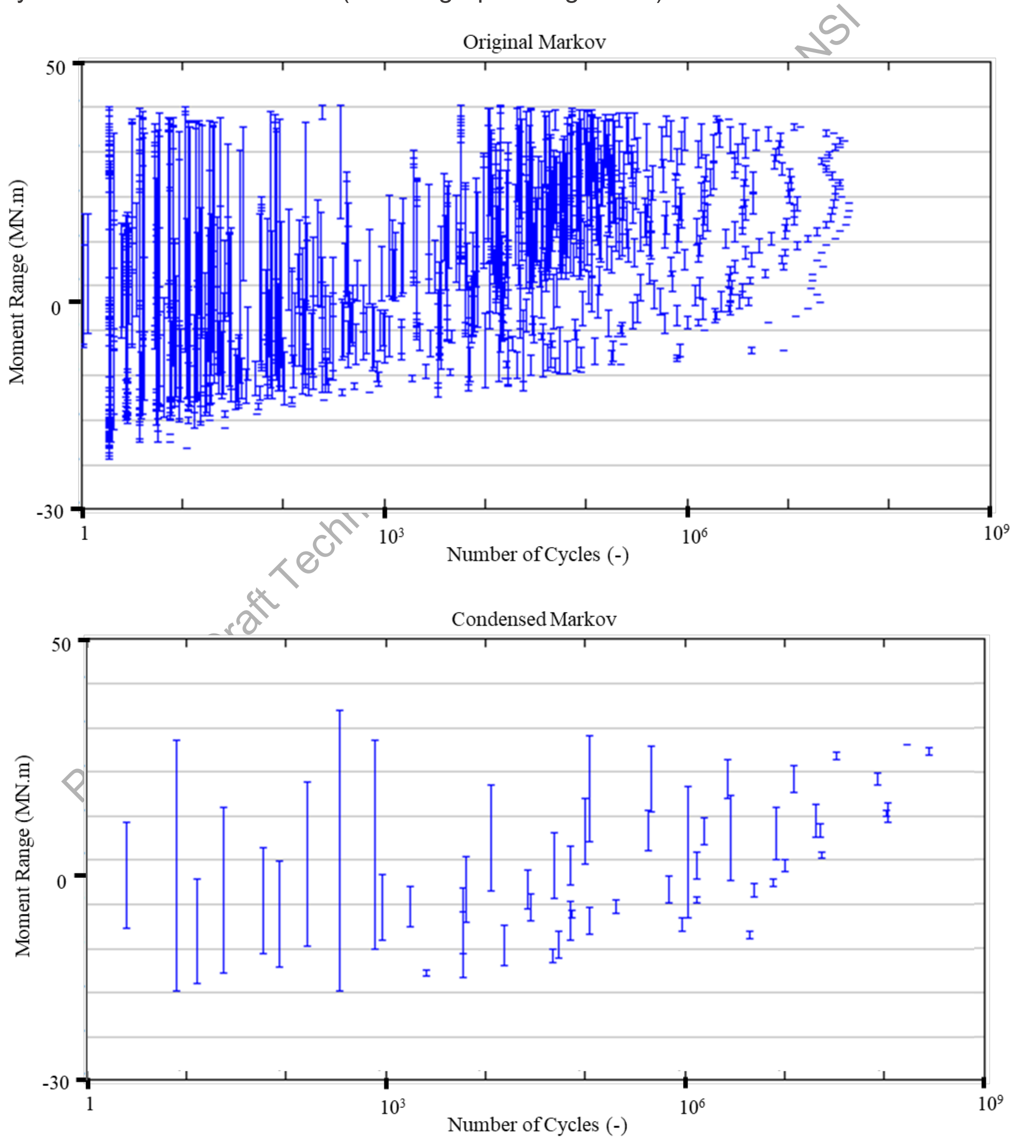


Figure 29: Original Markov matrix versus condensed Markov matrix

The tolerance level can be adjusted to obtain a reasonably balanced level of consolidation and representation. The loads in the condensed Markov matrix can be used as inputs in the foundation analysis. Alternatively, further verification can be performed on the level of loads bins with higher potential of contribution toward degradation and those that could be discarded assuming they have little or no significant impact on the overall degradation. This can be achieved through cyclic laboratory testing where the cyclic threshold strain levels and effects of number of cycles can be examined (Figure 30).

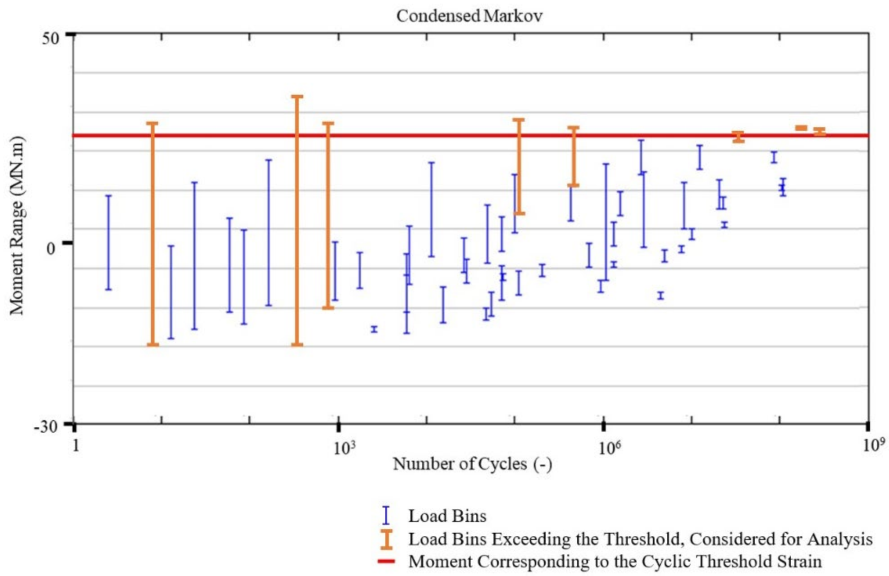


Figure 30: Condensed Markov matrix with examples for threshold values – Actual threshold values to be determined by user based on site-specific analysis

7.4.2 Dividing the Markov Matrix

Condensing the Markov may cause loss of accuracy in the analysis specially when the analysis results are sensitive to the load variation within the selected tolerance. As an alternative approach, the Markov can be divided to a few data points to be used as loads input for analysis. The result of the analysis for each outcome (strain, stress, etc.) can be used to form a correlation between the analysis outcomes and the corresponding loads. If enough data points are analyzed and a correlation is established, then the analysis results can be interpolated for all Markov bins without re-running the analysis and therefore without a need to condense. A hypothetical example is shown in Figure 31. Soil response (in this example shear strain) is determined from analysis (e.g., FEA) for the highest load in the Markov matrix (in this example moment), as well as for zero load and two intermediate load points. Then a correlation is established between the load (moment) and analysis results (shear strain).

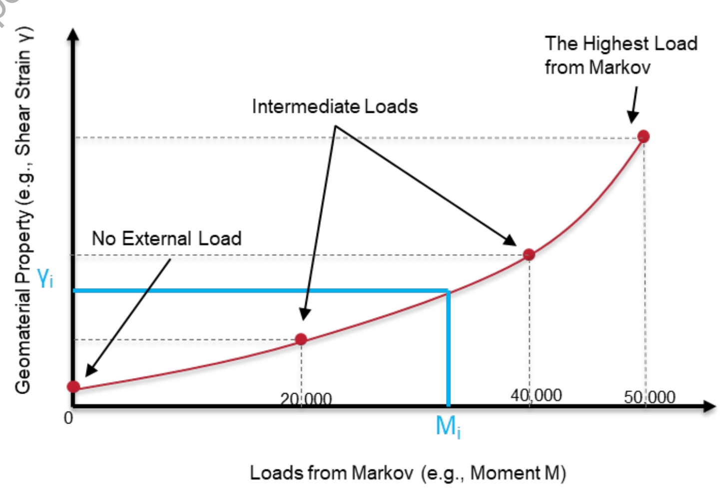


Figure 31: Example of correlation obtained between the load (moment) and analysis results (shear strain)

7.4.3 Subgrade Cyclic Evaluation Using Markov Matrix

The behavior of geomaterial under cyclic loading can be modeled by adopting an appropriate constitutive relationship for the geomaterial under cyclic loading as discussed in Section 6 herein. A degradation parameter can be defined as a function of strain and number of cycles for each desired material property of the geomaterial. Further discussion of geomaterial degradation behavior is presented in Section 5.

When a degradation parameter is defined, and a correlation is established, the approach presented in Section 7.4.2 can be used to evaluate the level of degradation using the loads in the Markov. To illustrate the method, a hypothetical and arbitrary degradation parameter is defined for a soil parameter (such as stiffness) as shown in Figure 32.

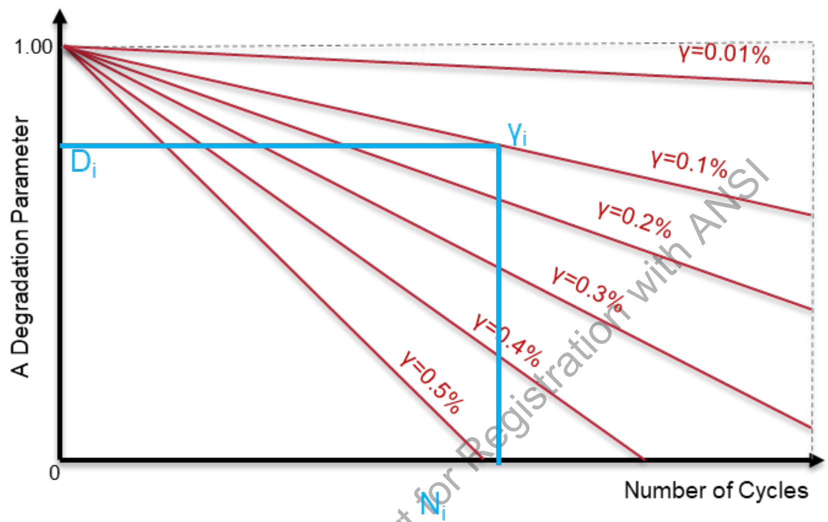


Figure 32: Hypothetical and arbitrary degradation parameter

Using the correlation established, for each row of the Markov, a geomaterial response (e.g., strain) can be obtained from the correlation similar to the example shown in Figure 31. Then knowing the strain, a degradation parameter can be obtained from a material model similar to the example shown in Figure 32. Next, a total degradation parameter can be defined by combining all the individual degradation parameters obtained for each bin of the Markov by assuming a conservative and reasonable sequence of loading. Finally, the level of degradation of the geomaterial property can be assessed and the reduced property after experiencing the full spectrum of loads in the Markov can be estimated knowing the total degradation parameter.

The foundation size, shape and position can be designed by trial and error such that the foundation design aspects calculated based on estimated reduced geomaterial properties (stiffness, settlement, bearing pressure) fall within the acceptable ranges.

8 Deep Foundations and Rock Anchors

8.1 Introduction

Deep foundations, although not as common as shallow foundations in the North American wind energy industry, are regularly considered as a foundation option for project sites that do not lend themselves to common shallow foundation approaches, or if a deep foundation concept can take advantage of site conditions and provide a more cost-effective foundation solution. This section will discuss common approaches to deep foundations and standards in use by the industry and provide guidance as to the recommended practices for assessing cyclic degradation of common deep foundations used for wind turbines.

Note that cyclic soil-structure interaction of deep foundations is by its nature highly complex and specific standards have not been developed to address how cyclic degradation is incorporated into a deep foundation design for a wind turbine. As a result, this section summarizes available documentation and standards that are available as a guide to practitioners, and provides a framework for future standards development, and identify areas that can be investigated by future research.

Deep foundations are typically used for supporting wind turbines in regions where stratum of adequate capacity is found at much greater depths, while the rock anchored foundations are also suitable for project sites where the bedrock resistance is sufficient at shallow depth.

Similar to the loading conditions for shallow foundations, a wind turbine supported on a deep foundation is subjected to long-term cyclic loading due to dynamic vibrations caused by wind loads and rotation of the blades. Section 5.2 of this guideline and DNV-ST-0126 state that cyclic effects are most significant for deep foundations installed in cohesive soils, cemented calcareous soils and fine-grained cohesionless soils (silt).

In this chapter, the cyclic behavior of geomaterial for deep foundations (including pile foundations, monopile foundations, and foundations with ground anchor system) is investigated and design recommendations are provided to limit potential degradation of geomaterial surrounding and beneath the deep foundations.

8.2 Standards for Wind Turbine Deep Foundations

Standards for deep foundations applied to wind turbines are included in the following documents:

- ACP 61400-6-2023, *Wind Energy Generation Systems – Part 6: Tower and Foundation Design Requirements – Modified Adoption of IEC 61400-6*
- DNV-ST-0126, *Support Structures for Wind Turbines* (supersedes GL Certification Rules, 2010, and DNV Riso *Guidelines for Design of Wind Turbines*, 2003)

Various codes and standards, which provide guidance for considering cyclic degradation effects in designing of pile foundations for onshore wind turbines, have been reviewed in this paper. Widely used standards for offshore wind turbines and other offshore structures (i.e., offshore oil and gas infrastructure) were also reviewed considering that the existing design codes for onshore wind turbines do not provide direct recommendations to cover the full range of impacts of cyclic loading on the soils supporting the turbine foundations. General recommendations for evaluating pile behavior under cyclic loading conditions are summarized in Table 2.

Table 2: Recommendations for Evaluating Cyclic Pile Behavior in Design Codes

Design Codes	Recommendations
DNV-ST-0126	• Laboratory testing (i.e., cyclic triaxial test and resonant column tests) for assessment of strength and stiffness degradation of turbine foundations under cyclic loading • Incorporating cyclic effects on shear strength of the soil in the applicable limit state and cyclic effects on soil shear modulus in the serviceability limit state (no detailed methods) • Cyclic pile behavior is more evident in cohesive soils than in medium to coarsely grained cohesionless soils
BSH (2007)	• Incorporating cyclic effects on shear strength of the soil in the applicable limit state and cyclic effects on soil shear modulus in the serviceability limit state (no detailed methods) • Cyclic pile behavior is more evident in cohesive soils than in medium to coarsely grained cohesionless soils
API RP 2A WSD (2000) API RP 2GEO (2014)	• Lab/on-site soil testing and pile load testing in determining the elastic properties of the soil and resistance-displacement relationship along the vertical and horizontal direction of the pile • Framework for evaluation of cyclic response of pile foundations using discrete element or continuum models

8.3 Summary of US Practice

US practice for deep foundations is summarized in ASCE/AWEA RP2011, and ACP 61400-6 addresses US-specific items for the implementation of the international standard.

8.4 Summary of Approaches and Methodologies

ACP 61400-6 and DNV-ST-0126 are typical standards used for design of deep foundations for wind turbines. Both standards indicate the risk of progressive degradation of the pile capacity or stiffness under the serviceability limit state (SLS) wind loads and specify the design requirements to limit the cyclic degradation potential of foundation support materials.

ACP 61400-6, Section 8.6.4.3 requires that soil sensitivity be identified under cyclic loading and the geotechnical investigation report should provide appropriate recommendations on the mitigation measures including limiting the mobilized shaft friction and end bearing stress to a low proportion of the pile capacity; or limiting or eliminating pile tension under the SLS load case LDD 10^{-2} (with exceedance probability of 10^{-2}).

DNV-ST-0126, Section 7.6.1 requires that the permanent cumulative deformations in the foundation support martials should be calculated as a function of the number of cycles at each load amplitude in the applied history of the SLS loads; and that separate tolerance for the permanent cumulative damages owing to the history of SLS loads throughout the design life should be specified for the deep foundations and support structures. DNV-ST-0126 also recommends that the effects of cyclic degradation be accounted for in the lateral and axial design for deep foundations, such as utilization of appropriate P-y and t-z curves considering the degradation of lateral resistance, stiffness, and unit skin friction under SLS load case LDD 10^{-2}.

8.4.1 Pile Foundations

Driven piles and drilled piers have been used as a deep foundation option in locations that require extending through a soft or otherwise unsuitable deposit to either generate additional capacity through skin friction, or end bearing into competent materials. Soil-structure interaction of pile groups is inherently complex, and as a result use of pile and pier foundations for support wind turbines has been limited to difficult soil conditions due to the relatively high cost.

Static design and basic cyclic design of pile foundations is covered by ACP 61400-6, as well as DNV-ST-0126 and numerous geotechnical references for deep foundations. Historically, pile and pier foundation designs have used the approach of limiting normal operational SLS loads on the pile or pier elements to downward compression only, and only allowing for pile uplift under extreme wind loads. In general, although it is necessary to analyze pile group behavior to model the complexities in the system, this approach has provided good performance over the history of the wind energy industry and limited the effects of cyclic degradation on typical driven and drilled pile foundations. Another typical aspect of pile and drilled shaft foundations is that translational stiffness is often the critical design aspect, rather than rotational stiffness typical with other deep and shallow foundation types.

As noted, ACP 61400-6 and other design standards and references address design of deep foundations under cyclic loading, though detailed guidance on this topic remains limited in the US onshore wind industry. In contrast, there are several references in the offshore industry that have focused significant effort in recent years around the topic of cyclic design of deep foundations. The design of foundations for offshore structures is complex and takes into account many factors not applicable for onshore structures, but the behavior of geomaterials and the interaction between the deep foundations and geomaterials is consistent.

Driven pile foundations for use in US onshore wind typically involve the use of large groups of 12.75-inch or 16-inch concrete filled closed end steel pipe piles attached to a pile cap that also houses the anchor bolt cage for the wind turbine tower. For the purposes of this discussion, driven piles are generally considered to exhibit a length-to-diameter ratio of at least 10 and often much greater. Deep foundations with large length-to-diameter ratios typically behave as slender beams, bending within the portion of the pile that is present above the neutral zone. Driven piles do not behave as rigid bodies in rotation only (as is the case for foundations with small length-to-diameter ratios).

Driven piles are further complicated by the fact that the in situ effective stress in the soil immediately surrounding the pile is altered during the installation process. While these stresses will eventually return to in situ conditions under static loading, the application of cyclic loading is that normal operational loads have the potential to create a continuous condition of elevated pore water pressures in the soil matrix. Ultimately, the capacity of driven piles under cyclic loading can be significantly less than the capacity under static loading. Further, the application of high cyclic loads on drive piles has the potential to result in permanent deformation at the pile cap resulting from accumulation of loads over the life of the structure, as well as creep associated with cyclically induced pore pressure dissipation. Further, these additional deformations related to the application of high cyclic loading can result in unexpected increases in the bending moment in the pile. Lastly, the application of high cyclic loading can lead to degradation in the soil shear strength and stiffness.

Design procedures for addressing high cyclic loading have been developed by the SOLCYP joint industry project for offshore foundation design and are currently under development under the MIDAS research project as well. Several recent references include a flowchart for design of driven piles under axial loading (referred to as the graduated design approach). Such approach includes a number of steps, namely data collection to understand the behavior of soils under static and cyclic loading through in situ and laboratory testing, local analyses to study the soil-structure interaction of a cyclically loaded element, global analyses to understand the impacts to the structure as a result of diminished soil strength and stiffness, as well as permanent deformations at the pile cap, and numerical modeling to study and simulate the interaction between the pile and surrounding soil when cycled. Interaction diagrams can be developed to understand the number of cycles a pile can sustain prior to reaching failure and such diagrams can be used as a simple and helpful screening tool.

However, at present time there is not a corollary design flowchart nor design approach using interaction diagrams to support design of driven piles under lateral loading.

Drilled pile or drilled shaft foundations have been used occasionally in the wind energy industry and are subject to the same level or greater level of complexity as driven pile foundations. Factors that influence cyclic degradation of drilled pile foundations can include the following:

- Stiffness of the connection between the drilled shaft and the pile cap
- Frequency dependence of relatively short stiff piles

8.4.2 Monopile Foundations

8.4.2.1 General

Monopile foundations have been used for wind turbine support for several decades and have been used for several generations of wind turbines. However, although there has been extensive research in recent years regarding offshore monopiles for wind turbine foundations, very little public information or research is available regarding performance of monopile piers that are used for onshore wind turbine applications due to industry confidentiality practices.

Short piers have several unique aspects to that are typically addressed during the design phase of a project:

- Dynamic behavior that falls between that of typical deep foundations such as piles and piers and that of typical shallow foundations
- Complex soil-structure interaction that is not well addressed by closed-form equations for either typical deep foundations or shallow foundations
- Potential for loading frequency to affect foundation stiffness
- Difficulty regaining rotational stiffness after exposure to extreme wind loads
- Sensitivity to construction tolerances and quality, specialized construction processes

8.4.2.2 Foundation Stiffness

Dynamic foundation stiffness should be verified based on the soil modulus adjusted for the anticipated soil strain level as discussed in IEC 61400-6. The foundation stiffness is a function of contact area, and this should be calculated for the S3 load level and any reduction from full contact should be accounted in the stiffness calculation. Detailed analysis or modeling should be performed to evaluate the range of strain and cyclic degradation around the foundation and verify that stiffness requirements are met. Static foundation stiffness, if specified by the turbine manufacturer, should be verified based on a soil modulus that makes allowance for the reduction of small-strain shear stiffness as a function of actual soil strain at S1 load level. This reduction depends on the soil characteristics and degree to which soil strength has been mobilized. The foundation stiffness should be calculated for the S1 load level including any reduction from full contact area.

Interaction of rotational and horizontal stiffness should be considered in design for monopiles. The foundation designer should coordinate with the turbine manufacturer as needed to establish requirements.

8.4.3 SLS: Long-Term Behavior

Verification of the geotechnical behavior under SLS should be performed to ensure that the foundation satisfies the serviceability criteria over the design lifetime of the wind turbine. Serviceability criteria include computations or modeling based upon advanced geotechnical laboratory testing to assess cyclic soil behavior should be included in the foundation design to verify that lateral stiffness and stiffness requirements are met, specifically for susceptible soil types identified during investigations.

8.4.4 Anchored Foundations

Rock and soil anchored foundations generally include a pile cap with post-tensioned anchors embedded into the supporting subsurface materials to resist uplift and overturning loads, as well as provide stiffness for the foundation. In order to reduce the potential for cyclic degradation of the soils, and fatigue damage of the anchors, the anchors are post-tensioned and designed to be kept in tension under normal operational load cases.

8.4.5 Introduction to Ground Anchor Foundation for Wind Turbines

Ground anchor foundations are evolved from ground anchors used for retaining structures and hold-down, which constitute a versatile construction system with many advantages in ground engineering. Ground anchors in the wind industry primarily function as foundations to support wind turbine generators (WTGs). In this regard, the dynamic aspects of wind turbine loading in anchors will be mobilized to the ground, either soils or rock masses. Thus, when choosing an anchorage foundation for WTGs, considerations must be given to the conditions of soils and rock masses that affect underground work. From the geotechnical engineering point of view, the geotechnical conditions must be sufficient to ensure the ground anchor foundation system is stable and durable. When ground investigation is completed, a comprehensive design should be performed including static and dynamic analysis, anchor capacity and load transfer length, pullout capability, overall stability, and other issues if deemed necessary.

8.4.6 Configuration and Components of the Ground Anchor Foundations for WTGs

Ground anchor foundation for WTGs typically consists of a reinforced concrete cap, round or square, with a diameter typically no smaller than twice of the diameter of the tower that support the wind turbines. The thickness of the cap should be dependent on design calculations but typically no less than 4 feet embedment in the ground. Ground anchor bars will be assembled in one or two rows near the circumference of the cap and extended and bonded in the ground through the holes preserved in the cap. When the bond strength reaches a level that meets the design requirement, post-tensioning will be applied to the ground anchor bars and a steel anchorage head is usually used to lock the bars to the surface of the reinforced concrete cap. Figure 33 is an example sketch that illustrates a ground anchor foundation configuration supporting WTGs.

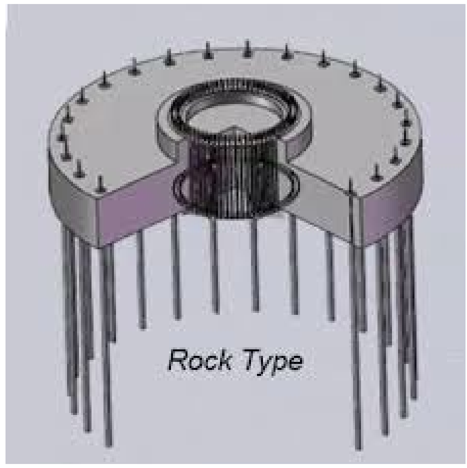


Figure 33: Ground anchor typical configuration

Anchor capacity and performance are influenced by three main factors: 1) ground characteristics; 2) installation method, particularly the method of fixing the bonding zone; and 3) the workmanship of the construction. These issues are certainly involved in the ground anchor foundation for wind turbines, which should be considered as a permanent facility, and potential problems of the developed bond between the steel tendon and injected grout being not as predicted and designed are of concern.

Creep or plastic deformation of soils will influence the performance of the anchor foundation greatly and thus the ground anchor foundation should preclude utilization of such foundation in soft soils. Creep and anchor tension loss potential must be considered carefully and supported with adequate testing of soil and rock materials and monitoring of anchor bolt tension.

8.4.7 Typical Approaches for Mitigation of Cyclic Degradation in Anchored Foundations

As noted in ACP 61400-6, due to the dynamic loading nature of wind turbines, additional means should be implemented to maintain rock anchor tension during the design lifetime of the foundation, particularly during the first 3 to 5 years of operation. Periodic anchor tension measurements or installation of permanent tension measurement devices may be utilized for select anchors for confirmation of adequate anchor tension during the design lifetime of the wind turbines.

Typical approaches for design incorporating cyclic degradation for anchored foundations have been centered around maintaining anchor tension to accommodate potential loading conditions that may affect the foundations. Commonly used measures include the following:

- Designing the foundation so that anchor tension is maintained during normal operation, as well as extreme wind loading conditions, including an allowance for cyclic degradation between to soil/rock to grout bond
- Removing all potentially compressible soil materials from beneath the anchor cap, to minimize the potential for anchor tension loss due to consolidation
- Incorporating an anchor tension monitoring and maintenance program into the operation and maintenance plan for the wind turbines
- Allowing for adjustment of anchor tension in the foundation and anchorage design, as opposed to a permanent grouted anchorage

9 Gravity Base Foundations

9.1 Geotechnical Design Considerations

Gravity base foundations for wind turbines derive their stability from resistance to sliding, overturing, bearing capacity failure, differential settlement, and insufficient soil stiffness. Foundation stability is achieved through relatively large diameter foundations embedded on the order of 8-12 feet below grade and covered with backfill soils. Degradation of geomaterial strength and stiffness due to cyclic loading may result in foundation displacements and reduction of soil bearing capacity and stiffness. For wind turbines, the reduction in foundation stiffness and/or increase in deformations can impact turbine operation frequency and foundation loads over time.

Section 5 provides discussion of the general state of practice for determination of soil stiffness and consideration of soil degradation potential in gravity base foundation analysis and design. Soil stiffness, as the primary concern relating to soil degradation is discussed in detail. Other design factors such as bearing capacity and settlement in terms of potential soil degradation considerations are also discussed briefly.

9.2 Potential Soil Degradation for Gravity Base Foundations

9.2.1 General

If using limit state design principles, the partial safety factor for loads should be consistent with the way wind turbine foundation loads are derived as discussed in Section 7. Alternative practice (such as allowable stress or working load design) may be adopted where required to maintain consistency with the reference standard for the region for which the design is being applied but this should result in at least the same level of safety as required by IEC 61400-1 or IEC 61400-2.

SLS conditions for S1, S2, S3 load levels should be as presented in Section 7.

The effect of cyclic loading on soil strength and stiffness should be addressed by considering the potential effects of soil movement due to ground gapping, effect of repeated loading on soil stiffness and degradation of soil strength due to repeated loading.

9.2.2 Geotechnical Data

Foundation design should be based on a good understanding of the ground conditions at each turbine location using geotechnical data of adequate quality and quantity. Geotechnical data should be obtained by performing sufficient in situ and laboratory testing within the zone of influence of the foundation to perform the geotechnical design (ACP 61400-6).

Degradation of competent bedrock subgrades are not expected to occur to the degree that impacts stiffness and strength through cyclic degradation.

9.2.3 Soil Stiffness Parameters

Soil stiffness should be determined as it has a fundamental effect on the behavior of dynamic structures such as wind turbines and minimum criteria for lateral, vertical and rotational stiffness should be satisfied as part of the geotechnical design. Direct or indirect techniques such as mechanical in situ tests, laboratory tests on soil samples or geophysical methods should be used where appropriate. One common method to determine soil stiffness in practice is to determine the small-strain shear modulus G_o. The soil modulus should be for a defined load or strain level associated with gravity base foundations as discussed in Section 5.2.2.

9.2.4 Soil Strength Parameters

The geotechnical evaluations should determine soil design parameters such as; undrained and drained shear strengths for cohesive soils, and internal angle of friction for granular soils, within the influence zone of the foundations. The geotechnical design parameters will be used to evaluate bearing capacity for gravity base foundations.

9.2.5 Groundwater/Soil Saturation

The presence of groundwater needs to be identified for soil degradation analyses. In addition to the effect of buoyancy on gravity base foundations, saturated soil conditions may increase the potential for some soils to degrade under dynamic loading conditions.

Monitoring using standpipes or similar should be considered if soils are susceptible to degradation under saturated conditions and water will be present.

9.2.6 Affected Soils

For gravity base foundations, typically saturated, fine-grained (clay or silt) soils are most susceptible to cyclic degradation. Other specific soils (such as liquefiable or collapsible soils) may require special consideration. See Section 5 for a more thorough discussion of soil types and potential for cyclic soil degradation.

9.3 General Design Principles and Practices

Gravity base foundations consist of a shallow base slab that derives its geotechnical resistance through equilibrium and bearing capacity of the founding soil. Soil capacity should be verified for bearing, sliding, and overturning failure modes.

9.3.1 SLS

Verification of the geotechnical behavior under SLS should be performed to ensure that the foundation satisfies the serviceability criteria over the design lifetime of the wind turbine. Soil degradation is generally considered to affect the SLS design condition. Serviceability criteria include the following:

- Compliance with the dynamic and (if specified) static rotational and lateral stiffness specified by the turbine manufacturer as the basis for the load calculations
- Control of maximum inclination and settlement of the foundation over the design lifetime of the foundation, and prevention of degradation of the soil bearing capacity or stiffness due to repeated or cyclic loading, for example accumulated generation of pore water pressures, hysteresis, creep, liquefaction or other degradation mechanism, which can ultimately lead to failure

9.3.2 Soil Stiffness

The most widely used soil characteristic to determine soil stiffness in practice is to determine the small-strain shear modulus (G_0). The most reliable methods of obtaining site-specific small-strain shear modulus involve the use of geophysical methods to measure shear wave velocity through a representative zone of influence below the foundation. Such methods include multichannel analysis of surface waves or cross-hole methods that are offered commercially in most regions. Alternatively, published correlations between other measured soil parameters and shear wave velocity may be used with caution, taking due consideration of correlation uncertainty.

The following relationship allows the small-strain shear modulus to be derived from shear wave velocities of the soil:

$$G_0 = \rho v^2$$

where

G_0 is the small-strain shear modulus;

ρ is the soil total density obtained from physical measurements;

v is the shear wave velocity.

The shear wave velocity used in the analysis should be the weighted average for the influence zone of the foundation (typically considered to be the depth corresponding to twice the foundation width). The weighted average method in Chapter 20 of ASCE 7 is a commonly used approach to obtain a representative value of shear wave velocity for a location.

The small-strain (maximum) shear modulus corresponds to very low strain levels (on the order of 10^{-6}). It is generally considered that the strain levels typical for conventional shallow gravity base foundations for wind turbines are several orders of magnitude higher. It is therefore not considered realistic to use the small-strain shear modulus to assess foundation stiffness.

Generally, wind turbine foundations operate at cyclic shear strain levels on the order 0.1% during normal operation and up to characteristic loads. Modulus reduction curves can be utilized to select a corresponding modulus reduction value for use in foundation analysis and design once an appropriate strain level is determined.

The information presented within Section 5 provides guidance for the evaluation of appropriate soil modulus and foundation rotational stiffness.

9.3.3 Soil Model

Many models have been developed to relate the nonlinear stress-strain characteristics of soil under loading. Site-specific models are not expected to be developed on a routine basis for all projects, and a generic approach may be adopted. Care should be taken to identify the characteristics of the site-specific soil conditions, which may invalidate the assumptions made in any generic stress-strain relationship.

Many models are available in published literature that account for differences in soil properties and stress history such as plasticity, voids ratio, over-consolidation ratio and number of cycles of loading. Vucetic and Dobry (1991) provide a useful review of the effect of such parameters.

The reduction of the slope of the stress/strain plot with increasing strain is indicative of reducing elastic and shear moduli. The reduction of soil shear modulus with strain level may be derived based on the following formula (Yi 2010):

$$\frac{G}{G_0} = \frac{1}{1 + \frac{R_f}{1 - R_f}\left[\frac{\gamma}{\gamma_f}\right]^{\alpha}}$$

where

G is the secant shear modulus at specific strain level γ;

G_0 is the small-strain shear modulus at $\gamma = 0$ (initial tangent value);

R_f is 1 – G_f/G_0 (assumed 0.95 in the above example) where G_f is shear modulus near soil failure;

γ_f is the soil strain near failure (assumed 0.01 in the above example);

α is the shape parameter of nonlinearity (assumed 0.95 in the above example).

9.3.4 Dynamic Rotational Stiffness

Dynamic foundation rotational stiffness should be verified based on the appropriate soil shear modulus reduced from the small-strain shear modulus as a function of the expected soil strain under the S3 load level. The expected soil strain level should be verified through computations or modeling as a part of the foundation design.

The generic formula for the calculation of dynamic rotational stiffness of a circular shallow foundation in contact with a semi-infinite uniform soil takes the form:

$$K_{R,dyn} = \frac{8 G_{dyn} R^3}{3(1 - \nu)}$$

where

$K_{R,dyn}$ is the dynamic rotational stiffness of the foundation subjected to overturning moments;

G_{dyn} is the shear modulus of the soil, reduced from G_0, to account for non-zero soil strain at load level S3;

R is the effective foundation radius in contact with the subgrade;

ν is the Poisson's ratio of the soil.

Enhancement terms may be added to the rotational stiffness formula to account for effects such as foundation embedment, limited depth to a harder stratum or increasing soil stiffness within the influence depth of the foundation.

Since the rotational stiffness is proportional to the cube of the radius of the contact area, it is highly sensitive to the contact area. If reduced foundation contact (i.e., gapping) occurs at load level S3, then the reduced foundation subgrade contact should be accounted for in determination of the effective foundation radius, R. For circular or octagonal raft foundations, the effective foundation radius may be taken as that corresponding to the area equivalent to the effective foundation contact area.

9.3.5 Static Rotational Stiffness

The static foundation rotational stiffness, where required to be checked by the turbine manufacturer, should be verified based on a soil modulus that makes allowance for the reduction of small-strain shear stiffness as a function of actual soil strain under S1 load case.

The calculation of static rotational stiffness for the foundation may take the same form as for the dynamic condition, adjusted for strain effects as shown below:

$$K_{\mathrm{R,stat}} = \frac{8 G_{\mathrm{stat}} R^3}{3(1-\nu)}$$

where terms are as defined in 9.3.4 and

$K_{\mathrm{R,stat}}$ is the static rotational stiffness of the foundation subjected to overturning moments;

G_{stat} is the shear modulus of the soil, reduced from G_0 to account for non-zero soil strain at load level S1.

If applicable, the same enhancements discussed in Section 9.3.4 may be incorporated into the static stiffness calculation with due adjustment for the differing load level.

As noted for the S3 load level, if reduced foundation contact occurs at load level S1, then reduced foundation subgrade contact should be accounted for in determination of the effective foundation radius, R.

9.3.6 Lateral Stiffness

Support structure dynamics may be strongly influenced by foundation lateral stiffness or by the interaction of foundation lateral stiffness with rotational stiffness. When specified by the turbine manufacturer, the lateral stiffness should be evaluated.

If required by the turbine manufacturer, or, in the case of a large vertical offset between the embedded foundation center of rotational stiffness and the location at which tower loads have been presented, the interaction of foundation horizontal and rotational stiffness should be evaluated.

9.3.7 Bearing Capacity

The ultimate bearing capacity of the soil formation below the foundation should be calculated from the geotechnical design values. Recommended bearing capacity should take specific account of the effect of repeated loading, and any expected degradation of ultimate bearing capacity over the design lifetime of the foundation should be addressed where relevant.

The bearing capacity of nonbedrock formation soils should be determined with reference to specific in situ or laboratory soil tests performed as part of the geotechnical site investigation. It is preferable to evaluate bearing capacity using characteristic soil properties such as undrained shear strength or angle of internal friction under applicable load conditions.

9.3.8 Settlement

The settlement potential for gravity base foundations is determined from the soil's compressibility characteristics or soil modulus, loading conditions, and soil saturation. Typically, differential settlement is compared to the allowable turbine tilt (commonly 3 mm/m unless otherwise specified by the turbine manufacturer).

9.3.9 Soil Degradation under Cyclic Loading

Potential soil sensitivity to repetitive or cyclic loads should be identified in the geotechnical investigation report. The risk of degradation of the soil capacity or stiffness should be evaluated as part of the foundation design. This risk may be addressed by fulfilling a zero-ground gap criterion, by other mitigation measures, or by adequately determining that the subgrade materials are not susceptible to cyclic degradation.

A zero-ground gap criterion can be fulfilled by proportioning the base to remain in full contact with the soil, under the S3 load level with partial safety factors for load of 1.0.

Alternative mitigation measures include limiting bearing pressures to acceptable values as recommended in the geotechnical investigation report or by replacement of sensitive soils.

If it can be demonstrated that all the following conditions are satisfied, then it is permissible that the resulting foundation design can be subjected to gapping between the foundation and underlying soil formation at the S3 load level.

- The foundation geometry is not controlled by rotational stiffness requirements or, in cases where it is, the soil modulus has been accurately determined based on in situ measurement of shear modulus for example cone penetration test or shear wave velocity measurements.
- The foundation stiffness calculation specifically accounts for any loss of contact area.
- Compliance with foundation inclination and settlement criteria are not sensitive to loss of contact area.
- The absence of high or variable groundwater conditions with the potential to lead to high pore water pressure or erosion of the soil under the foundation during prolonged cyclic loading.
- Cyclic loading is not expected to lead to a significant reduction of soil modulus such that it governs the foundation geometry.
- The soil is verified through acceptable testing or calculations as not susceptible to degradation of strength under repeated cyclic loading at the load levels being applied such that it governs the foundation geometry.

9.3.10 Stiffness Reduction

The foundation system should meet the required stiffness criteria as defined by the turbine manufacturer.

Dynamic foundation rotational stiffness should be verified based on the appropriate soil shear modulus reduced from the small-strain shear modulus as a function of the expected soil strain under the S3 load level. The foundation stiffness is a function of contact area, and this should be calculated for the S3 load level, and any reduction from full contact should be accounted in the stiffness calculation.

Static foundation stiffness, if specified by the turbine manufacturer, should be verified based on a soil modulus that makes allowance for the reduction of small-strain shear stiffness as a function of actual soil strain at S1 load level. This reduction depends on the soil characteristics and degree to which soil strength has been mobilized. The foundation stiffness should be calculated for the S1 load level including any reduction from full contact area.

Guidance on the selection of appropriate soil modulus and foundation stiffness is presented in Section 5.4.

9.4 Soil Strength Reduction Foundation Bearing Capacity/Strength Degradation

Soil capacity should be verified for bearing and sliding failure modes.

The ultimate bearing capacity of the soil formation below the foundation should be calculated from the geotechnical data presented in the geotechnical investigation report. Where the foundation is bearing directly onto fresh or slightly weathered bedrock, bearing capacity is not normally critical to the design and a reasonably conservative value may be adopted based on values obtained from literature and recommended in the geotechnical investigation report. Recommended bearing capacity should take specific account of the effect of repeated loading, and any expected degradation of ultimate bearing capacity over the design lifetime of the foundation should be addressed where relevant.

The bearing capacity of nonbedrock subgrade materials should be determined with reference to site-specific in situ or laboratory soil tests performed as part of the geotechnical site investigation. It is preferable to evaluate bearing capacity using characteristic soil properties such as undrained shear strength or angle of internal friction with application of appropriate partial safety factor on material. The recommended bearing capacity should take account of the effects of load inclination, foundation shape, depth (including the effect of sloping ground), and groundwater conditions. The effect of variations in soil properties within the zone of influence (rupture zone), either beneath or to the side of the foundation should be considered in the calculation of bearing capacity.

10 Monitoring and Testing of Foundation Stiffness Degradation

10.1 General

Loss of strength and stiffness over time causes reduction of foundation stiffness that can result in excessive foundation rotation and turbine tower tilt, thus affecting the turbine operation frequency and dynamic performance of the turbine components. Instrumentation programs can be implemented to verify the degradation of the foundation stiffness over the turbine operational lifetime. Additionally, experimental studies could be performed to better understand the stress and strain levels of the geomaterials, which contribute toward the cyclic degradation behavior of an onshore wind turbine foundation.

Measurements should consider the alignment with the predominant wind direction to increase effectiveness of capturing the maximum rotations, strains, and stresses.

10.2 Monitoring of Foundation Stiffness

One approach to verify the degradation of the foundation stiffness is by monitoring the rotation (tilt) of the foundation and magnitude of overturning moment transferred to the foundation. Direct measurements of the angle of rotation θ can be taken with tiltmeters installed at the top of the concrete level of the foundation. For the magnitude of overturning moment M, the strain ε in the walls of the tower base can be measured and used to calculate the corresponding stress σ for a given elastic modulus E. Then the stress σ can be used to calculate the corresponding overturning moment M for a given section modulus S. Both calculations should be based on material properties and cross-sectional dimensions of the tower. Temperature sensors can be used to calibrate the readings of strain ε in the walls of the tower.

$$M = \sigma \times S$$

where

$$\sigma = E \times \varepsilon$$

$$S = [\pi \times (d_2^4 - d_1^4)] / (32 \times d_2)$$

where

M is the overturning moment;

σ is the stress;

S is the section modulus;

E is the elastic modulus of tower;

ε is the strain in the walls of the tower base;

d_1 is the inner diameter of tower;

d_2 is the outer diameter of tower.

These measurements can then be used to calculate the rotational stiffness that is defined as the ratio between the overturning moment M and angle of rotation of the foundation θ ($K_R = M/\theta$). This monitoring approach verifies the overturning moment required to rotate the foundation by a certain angle. Thus, the monitoring of the foundation stiffness can be performed to assess the amount of stiffness degradation over time.

10.3 System Frequency Data

Several modern wind turbines include nacelle accelerometers and process acceleration data into system frequency estimates. If system frequency data are available, these frequency estimates can be used for several aspects of foundation performance and offer the following advantages:

- Identification of trends of foundation stiffness over the life of the turbine
- Can be used to calibrate measurements from tiltmeter/strain gauge measurements made during shorter time intervals
- Frequency measurements are a relatively inexpensive approach to screen out potential foundation stiffness issues and identify candidates for more detailed investigation and measurements

Although considerably more complicated, a wind turbine, foundation, and the supporting geomaterial can be idealized into what is effectively a series spring system. This idealization allows for numerical predictions of the system that are deemed accurate enough for understanding the behavior of the turbine dynamics.

During typical operation, the wind turbine system is dominated by what can be considered two independent springs; the wind turbine itself and the soil-foundation system rotational stiffness. This observation results in a simplified model in which the fixed base system frequency (frequency of the wind turbine without the foundation influence), the soil-foundation system rocking frequency, and the resulting system frequency can be related. OEMs typically provide turbine foundation stiffness requirements that correspond to the resulting system frequency.

Since there is a direct relation between the observed system frequency of the turbine-foundation-soil system and the rotational stiffness of the foundation-soil system, records from the SCADA system that typically include turbine frequency data can be used to infer changes in the rotational stiffness of the foundation-soil system. A downward trend in frequency is typically indicative of potential degradation in the turbine-foundation-soil system stiffness. However, numerical noise and other artifacts in the data can introduce challenges in understanding specific trends and contribute to increased uncertainty in projections of the frequency forward in time through the useful life of the turbine. Evaluation of data over a relatively long period is typically necessary to assess data quality and possible periodic trends over time. An example plot of turbine frequency data is presented in Figure 34, which shows relatively stable frequency over 7 years of operation implying consistent foundation/geomaterial stiffness.

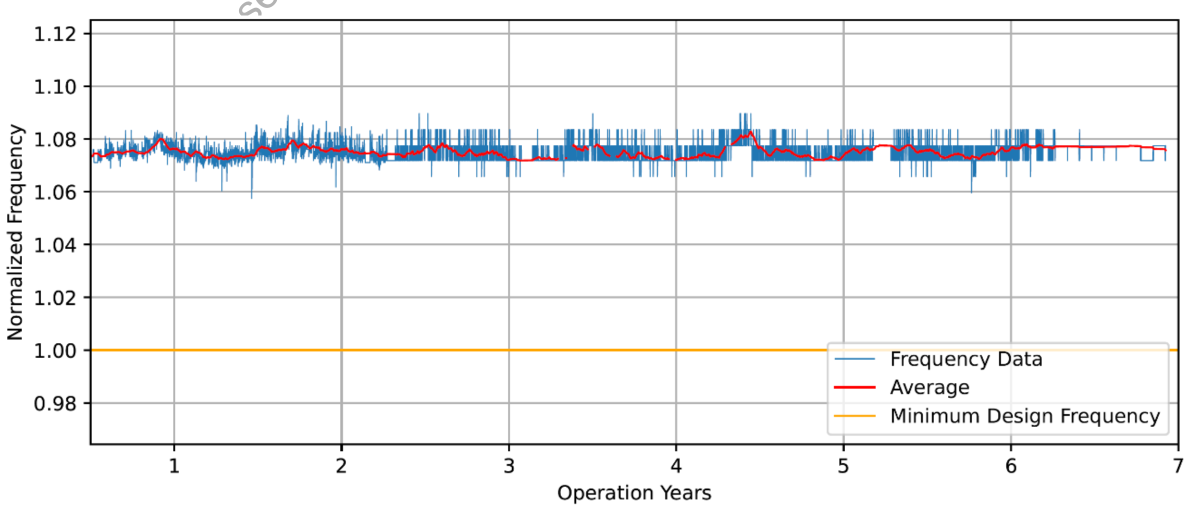


Figure 34: Example turbine stable frequency data

Figure 35, however, shows an example plot of turbine frequency data from the same wind farm but with a reduction in operating frequency within the first 3.5 years of operation, which in this case was indicative of cyclic degradation of the foundation support materials. At the 3.5-year mark, ground improvement was implemented and the turbine operation frequency stabilized for the remaining life of the wind turbine foundation. These data demonstrate how turbine frequency data can be used to infer changes in the rotational stiffness of the foundation-soil system, which may be indicative of cyclic degradation of the support materials.

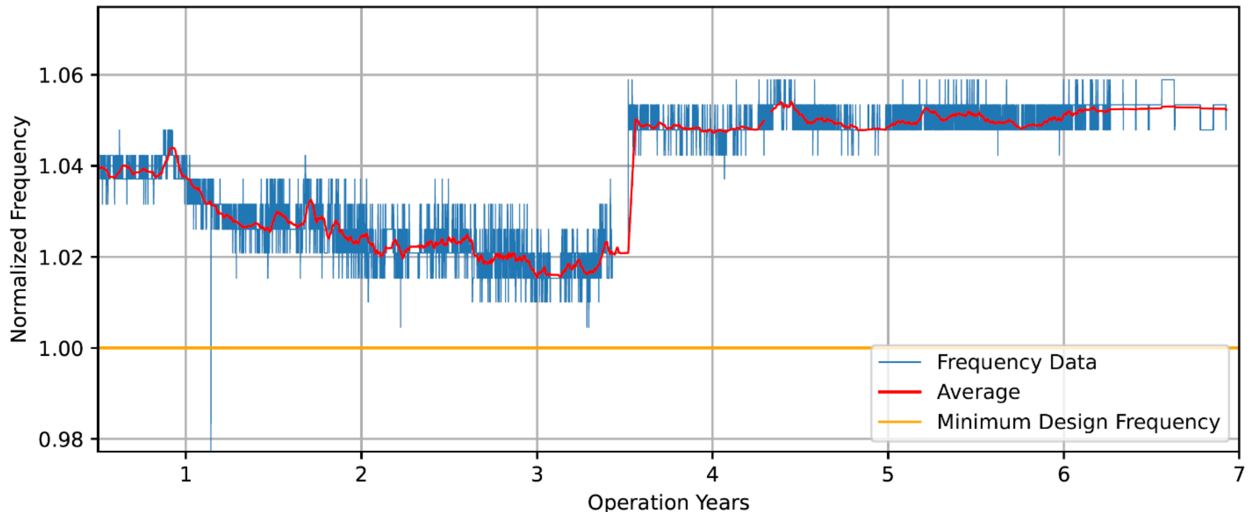


Figure 35: Example turbine frequency degradation data

In general, if high-resolution data are available, a more accurate analysis of frequency data can be conducted. However, useful results can still be obtained from 10-minute and daily averages of frequency data coming from the SCADA system.

10.4 Calibration and Verification

Although details of calibration wind turbine SCADA systems and foundation measurements are beyond the scope of this document, calibration of any foundation or turbine measurements are imperative to obtaining useful data from any monitoring system. Example of calibration techniques include the following:

- Accounting for sensor drift typical to resistance type strain gauges
- Performing a turbine yaw sweep under low wind conditions to establish baseline readings
- Cross-checking measurement data across multiple methods and/or data sources

10.5 Experimental Testing

Further experimental studies can be performed for research purposes and to improve 3D finite element model methodology. 3D finite element model methods can be used to verify the measured foundation uplift and stiffness by modeling the stress and strain levels below the foundation, since these are commonly verified using closed-form solutions.

Such experimental tests would be useful to further evaluate the stresses and strains under a foundation, depth of influence of cyclic degradation, pore water pressure generation, and other effects of cyclic degradation.

10.6 Instrumentation Types

The following types of instrumentation can be considered for possible monitoring and testing programs:

- Tiltmeters or inclinometers located at the top of the foundation to measure change in vertical level at a specific point, obtain direct measurements on the angle of foundation inclination and verify the foundation rotational stiffness.
- Strain gauges located at the base of the tower, along with temperature sensors to calibrate strain readings, to measure the strain in the walls of the tower, calculate the magnitude of overturning moment transferred to the foundation and verify the foundation rotational stiffness.
- Displacement transducers or soil extensometers located at different depths below the foundation base to measure the relative movement between two or more points, verify the maximum shear strains and estimate the depth of influence of cyclic degradation.
- Earth pressure cells located between the bottom of foundation and top of subgrade to measure contact pressures during loading events, verify maximum bearing pressures and evaluate uniformity of pressure distribution.
- Pore water pressure transducers located at different depths below the foundation base to measure pore water pressure generation during loading events and estimate the depth of influence of pore water pressure generation.

10.7 Concept for Instrumentation Layout

Figure 36 shows an example layout of turbine foundation instrumentation for collecting data that can be useful for understanding turbine foundation dynamics and cyclic response of the supporting geomaterials.

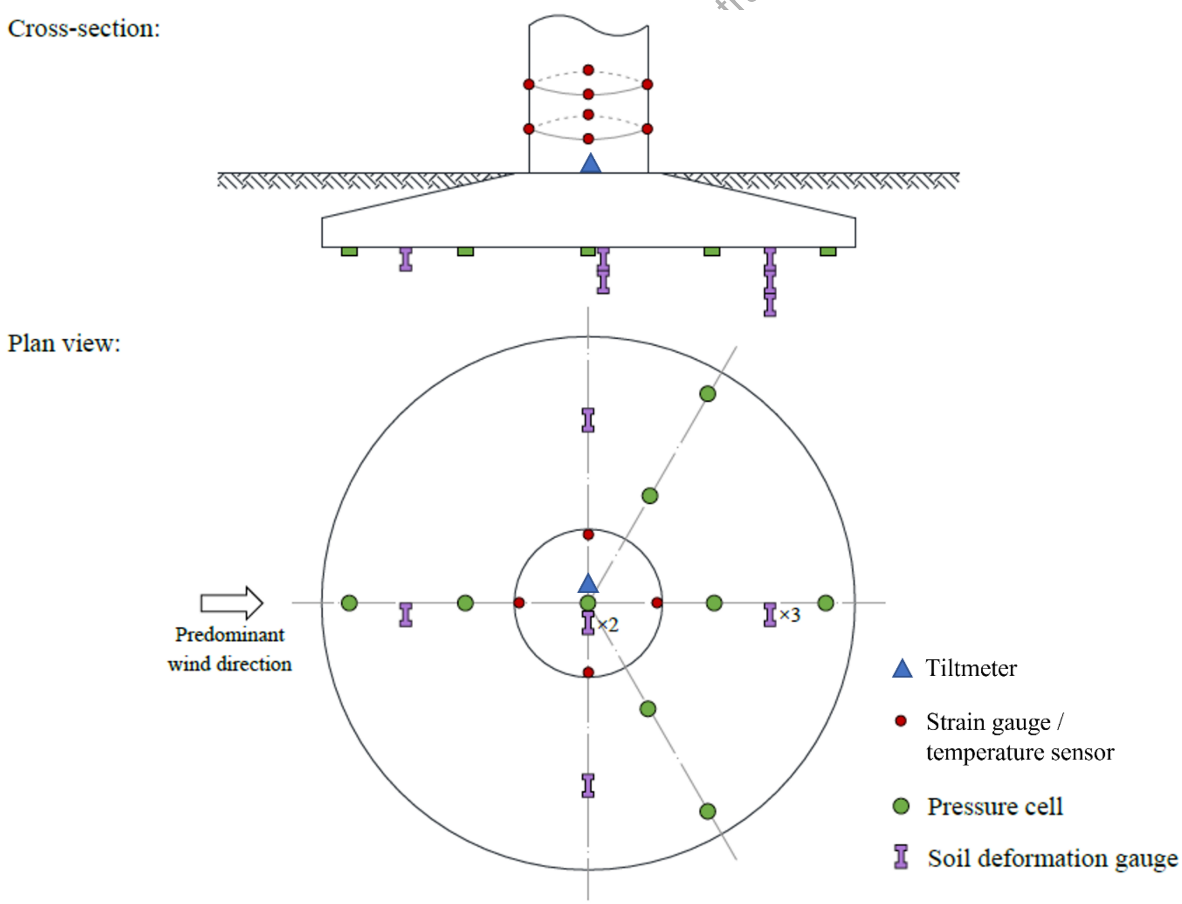


Figure 36: Example layout of instrumentation (Seymour 2018)

11 References

Title
Agaiby, S.W., and Ahmed, S.M., Learning from Failures: A Geotechnical Perspective. Proceedings of the International Conference on Forensic Civil Engineering, Nagpur, India, 2016
Andersen, K.H., Bearing Capacity under Cyclic Loading – Offshore, Along the Coast, and on Land. The 21st Bjerrum lecture presented in Oslo, 23 November 2007, Canadian Geotechnical Journal 46(5):513-535, May 2009
Andersen, K., Cyclic Clay Data for Foundation Design of Structures Subjected to Wave Loading. International Conference “Cyclic Behaviour of Soils and Liquefaction Phenomena,” Germany, 31.3-2.4, 2004
Andersen, K.H., Cyclic Soil Parameters for Offshore Foundation Design. Frontiers in Offshore Geotechnics III, Taylor and Francis Group, 2015
Andersen, K.H., Kleven, A., and Heien, D., Cyclic Soil Data for Design of Gravity Structures. ASCE Journal of Geotechnical Engineering 114(5), May 1988
Ansal, A.M., and Erken A., Undrained Behavior of Clay under Cyclic Shear Stresses. J Geotech Engrg 115(7):968-983, 1989
ASCE Wind-Induced Force Committee, Wind Loads and Anchor Bolt Design for Petrochemical Facilities, Task Committee on Wind-Induced Forces, ASCE, 1997
Banerjee, A. Response of Unsaturated Soils under Monotonic and Dynamic Loading Over Moderate Suction States. PhD Thesis, University of Texas at Arlington, 2017
Beaty, M.H., and Byrne, P.M., UBCSAND Constitutive Model, Version 904aR, 2011
Boulanger, R.W., and Ziotopoulou, K., PM4Sand (Version 3.3): A Sand Plasticity Model for Earthquake Engineering Applications, Report No. UCD/CGM-23/01, Center for Geotechnical Modelling, Department of Civil and Environmental Engineering, University of California, Davis, CA, 2023
Boulanger, R.W., Ziotopoulou, K., and Oathes, T.J., PM4SiltR (Version 1): A Silt Plasticity Model with Rate Effects for Slope Stability Applications, Report No. UCD/CGM-22/04, Center for Geotechnical Modelling, Department of Civil and Environmental Engineering, University of California, Davis, CA, 2022
Briaud, J.L, Powers, W. III, and Weatherby, D.E., Should Grout Anchors Have a Short Tendon Bond Length? J Geotech Geoenviron Eng 124:110-119, 1998
Brinkgreve, R.B.J., Selection of Soil Models and Parameters for Geotechnical Engineering Application. In: Soil Constitutive Models: Evaluation, Selection, and Calibration, Austin, January 24-26, pp. 69-98, 2005
Brinkgreve, R.B.J., Engin, E., and Swolfs, W.M., Plaxis 3D. General Information Manual, 2013
Brown, E.T., Rock Engineering Design of Post-Tensioned Anchors for Dams – A Review. J Rock Mech Geotech Eng 7(1):1-13, 2015
Burnett, D.S., A Three-Dimensional Acoustic in Finite Element Based on a Prolate Spheroidal Multipole Expansion. J Acoust Soc Am 96(5):2798-2816, 1994
Chang, W.J., Rathje, E.M., Stokoe, K.H., and Hazirbaba, K., In Situ Pore-Pressure Generation Behavior of Liquefiable Sand. J Geotech Geoenviron Eng 133:921-931, 2007
COMSOL Blog, Best Practices for Meshing Domains with Different Mesh Sizes, 2019. https://www.comsol.com/blogs/best-practices-for-meshing-domains-with-different-size-settings
Cundall, P., and Hart, R., 1993, Chapter 9, Numerical Modelling of Discontinua. In: Analysis and Design Methods, Principle, Practice and Projects, Reprinted by Elsevier 2014
Darendeli, M.B., Development of a New Family of Normalize Modulus Reduction and Material Damping Curves. PhD Dissertation, University of Texas at Austin, 2001

Title
Dassault Systems, Abaqus 6.11 Theory Manual, p. 1172, 2011
Diaz-Rodriguez, J.A., and Lopez-Molina, J.A., Strain Thresholds in Soil Dynamics. The 14th World Conference on Earthquake Engineering, 2008
Dobry, R.R., Ladd, S., Yokel, F.Y., Chung, R.M., and Powell, D., Prediction of Pore Water Pressure Buildup and Liquefaction of Sands During Earthquakes by the Cyclic Strain Method. National Bureau of Standards, Washington, DC, 1982
Dong, Y., Lu, N., and McCartney, J.S., Unified Model for Small-Strain Shear Modulus of Variably Saturated Soil. J Geotech Geoenviron Eng 142(9):04016039, 2016
Endicott, L.J., Design and Construction of Excavations in an Urban Setting – Lessons Learnt from Failures. ICGE Colombo 2015, Colombo, Sri Lanka
Erten, D., and Maher, M.H., Cyclic Undrained Behavior of Silty Sand. Soil Dynamics and Earthquake Engineering 14(2):115-123, 1995
FHWA, Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems. Publication N. FHWA-IF-99-015, 1999
Franzius, J.N., and Potts D.M., Influence of Mesh Geometry on Three-Dimensional Finite-Element Analysis of Tunnel Excavation. ASCE Int J Geomech 5:3(256), 2005
Gazetas, G., Fan, K., Tazoh, T., Shimizu, K., Kavvadas, M., and Makris, M., Seismic Pile-Group-Structure Interaction. In Piles under Dynamic Loads, ASCE Geotechnical Special Publication No. 34, 1992
Gibson, R.E., Schiffman, R.L., and Whitman, R.V., On Two Definitions of Excess Pore Water Pressure. Geotechnique 39(1):169-171, 1989
Hardin, B.O, and Black, W.L., Vibration Modulus of Normally Consolidated Clay. J Soil Mech Fdns Div Am Soc Civ Engrs 95(6):1531-1537, 1968
Hardin, B.O., and Black, W.L., Closure to Vibration Modulus of Normally Consolidated Clays. Journal of Soil Mechanics and Foundations Division, ASCE 95(6):1531-1537, 1969
Idriss, I.M., and Boulanger R.W., Soil Liquefaction During Earthquakes. Earthquake Engineering Research Institute, 2008
Idriss, I.M., Dobry, R., and Singh, R.D., Nonlinear Behavior of Soft Clays During Cyclic Loading, Journal of the Geotechnical Engineering Division 104(12):1427-1447, 1978
ISSMGE Technical Committee TC 209, Offshore Geotechnics, Proceedings of TC 209 Workshop 18th ICSMGE – Paris, 4 September 2013. “Design for Cyclic Loading: Piles and other Foundations,” edited by Alain Puech
Itasca Consulting Group, FLAC 3D 6.0 Theory and Background, p. 692, 2017
Jardine, R.J., Puech A., and Andersen, K.H., Cyclic Loading of Offshore Piles: Potential Effects and Practical Design. Proceedings of the 7th International Conference on Offshore Site Investigations and Geotechnics, Society for Underwater Technology, pp. 59-100, 2012
Jefferies, M.G., and Shuttle, D.A., NorSand: Features, Calibration and Use, 2012
Kammerer, A.M., Pestana, J.M., and Seed, R., Undrained Response of Monterey 0/30 Sand under Multidirection Cyclic Shear Loading Conditions. University of California, Berkeley, 2002
Khashila, M., Hussien M., Chekired, M., and Karry, M., On the Dynamic Soil Behavior under Triaxial and Simple Shear Modes. ASCE Int Geomech, 2021
Khosravi, A., and McCartney, J.S., Impact of Hydraulic Hysteresis on the Small-Strain Shear Modulus of Unsaturated Soils. J Geotech Geoenviron Eng 138(11):1326-1333, 2012

Title
Khosravifar, A., Elgamal, A., Lu, J., and Li, J., A 3D Model for Earthquake-Induced Liquefaction Triggering and Post-Liquefaction Response. Civil and Environmental Engineering Faculty Publications and Presentations, 451, 2018
Kim, H.K., and Cho, N.J., A Design Method to Incur Ductile Failure of Rock Anchors Subjected to Tensile Loads. Electronic J Geotech Eng 17:2737-2746, 2012
Kim, N.K., Performance of Tension and Compression Anchors in Weathered Soil. J Geotech Geoenviron Eng 129:1138-1150, 2003
Kim, N.K, Park, J.S., and Kim, S.K., Numerical Simulation of Ground Anchors. Computers and Geotechnics 34:498-507, 2006
Klar, A., and Frydman, S., Three-Dimensional Analysis of Lateral Pile Response using Two-Dimensional Explicit Numerical Scheme. ASCE J Geotech Geoenviron 128(9):775-784, 2003
Labuz, J.F., and Zang, A., Mohr-Coulomb Failure Criterion. Rock Mech Rock Eng 45:975-979, 2012
Lade, P.V., Overview of Constitutive Models for Soils. Proceedings of GeoFrontiers 2005, American Society of Civil Engineers, 2005
Lees, A., Geotechnical Finite Element Analysis: A Practical Guide. ICE Publishing, 2016
Littlejohn, G.S., and Bruce, D.A., Rock Anchors – State of the Art. Foundation Publication Ltd., Essex, UK, 1977
Liu, C., and Xu, J., Experimental Study on Effects of Initial Conditions on Liquefaction of Saturated and Unsaturated Sand. Int J Geomech August 15, 2013
Liu, X., Wang, J., Huang, J., and Jiang, H., Full-Scale Pullout Tests and Analysis of Ground Anchors in Rock under Ultimate Load Conditions. Eng Geol 228:1-10, 2017
Logan, Darly L., A first Course in the Finite Element Method. 4th Edition, 2007
Mamou, A.H, Powrie, W., Priest, J.A, and Clayton C., The Effects of Drainage on the Behaviour of Railway Track Foundation Materials During Cyclic Loading. Geotechnique 67(10):845-854, October 2017
Matasovic, N., and Vucetic, M., Cyclic Characterization of Liquefiable Sands. J Geotech Geoenviron Eng 119(11):1805-1822, 1993
Matasovic, N., and Vucetic, M., Generalized Cyclic-Degradation-Pore-Pressure Generation Model for Clays. Journal of Geotechnical Engineering 121:1(33), 1995
Matsui, T., Ito, T., and Ohara, H., Cyclic Stress–Strain History and Shear Characteristics of Clay, Journal of the Geotechnical Engineering Division 106(10):1101-1120, 1980
Mortezaie, A.R., Cyclic Threshold Strains in Clays versus Sands and the Change of Secant Shear Modulus and Pore Water Pressure at Small Cyclic Strains. PhD Thesis, University of California, Los Angeles, 2012
Narasaiah G.L., Finite Element Analysis. BS Publications, Hyderabad, India, 2008
Okereke, M., and Keates, S., Mathematics of Element Formulation. Finite Element Applications – A Practical Guide to the FEM Process. Springer, 2018
Okur, D.V, and Ansal, A., Stiffness Degradation of Natural Fine-Grained Soils During Cyclic Loading. Soil Dynamics and Earthquake Engineering 27:843-854, 2007
Pissanetzky S., An Infinite Element and a Formula for Numerical Quadrature Over an Infinite Interval. Int J for Numerical Methods in Engineering 19(6):913-927, 1983
Plaxis, 3D Manual-Reference, 2017
Potts, D.M., and Zdravkovic, L., Finite Element Analysis in Geotechnical Engineering: Theory. Thomas Telford Limited, London, 1999

Title
Potts, D.M., and Zdravkovic, L., Finite Element Analysis in Geotechnical Engineering: Application. Thomas Telford Limited, London, 2001
Rascol, E., Cyclic Properties of Sand: Dynamic Behaviour for Seismic Applications. Thesis, École Polytechnique Fédérale de Lausanne, Suisse, 2009
Rong, W., and McCartney, J.S., Undrained Seismic Compression of Unsaturated Sand. J Geotech Geoenviron Eng 47(1):04020145, 2020a
Rong, W., and McCartney, J.S., Drained Seismic Compression of Unsaturated Sand. J Geotech Geoenviron Eng 146(5):04020029, 2020b
Sangrey, D.A., Henkel, D.J., and Esrig, M.I., The Effective Stress Response of a Saturated Clay Soil to Repeated Loading. Canadian Geotechnical Journal 6(3), 1969
Seed, H.B., and Chan, C.K., Clay Strength under Earthquake Loading Conditions. Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, 1966
Seed, H.B., and Idriss, I.M., Simplified Procedure for Evaluating Soil Liquefaction Potential. J Geotech Eng Div 97(9):1249-1273, 1971
Seed, H.B., Martin, P.P., and Lysmer, J., The Generation and Dissipation of Porewater Pressures During Soil Liquefaction. Rep. No. UCB/EERC75-26, Earthquake Engineering Research Center, UC-Berkeley, CA, 1975
Seymour, S., Numerical Modelling of Onshore Wind Turbine Gravity Foundations Susceptible to Cyclic Degradation. Department of Civil Engineering, University of Cape Town, 2018
Terzaghi, K., Theoretical Soil Mechanics. New York: Wiley, 1943
Terzaghi, K., Peck, R.B., and Mesri, G., Soil Mechanics in Engineering Practice. John Wiley & Sons, 1996
Tokusho T., Yoshida, Y., and Esashi, Y., Dynamic Properties of Soft Clay for Wide Strain Range. Soils and Foundations 22(4):1-18, 1982
Vaid, Y.P, and Chern, J.C., Effect of Static Shear Resistance to Liquefaction, Soils and Foundations. Japanese Geotechnical Society, 1983
Vaid, Y.P., and Chern, J.C., Cyclic and Monotonic Undrained Response of Saturated Sands. Proceedings, Advances in the Art of Testing Soils under Cyclic Conditions, 1985
Van Wijngaarden, M., Gravity Based Foundations for Offshore Wind Turbines – Cyclic Loading and Liquefaction. Master Thesis, Delft University of Technology and the National University of Singapore, 2018
Vucetic, M., Cyclic Threshold Shear Strain in Soils. ASCE Journal of Geotechnical Engineering 120(12), December 1994
Vucetic, M., and Dobry, R., Degradation of Marine Clays under Cyclic Loading. ASEC J Geotch Eng 114(2):133-149, 1988
Vucetic, M., and Dobry, R., Effect of Soil Plasticity on Cyclic Response. Journal of Geotechnical Engineering 117(1), January 1991
Vytiniotis, A., Contributions to the Analysis and Mitigation of Liquefaction in Loose Sand Slopes. PhD Dissertation, Massachusetts Institute of Technology, 2012
Xanthakos, P.P., Ground Anchors and Anchored Structures. John Wiley & Sons, 1991
Yasuhara, K, and Andersen, K.H., Recompression of Normally Consolidated Clay after Cyclic Loading, Soils and Foundations. Japanese Geotechnical Society, 1983

Title
Yasuhara, K., and Andersen, K.H., Effect of Cyclic Loading on Recompression of Overconsolidated Clay. Proceedings of the 12th International Conference on Soil Mechanics and Foundation Engineering, Volume 1, pp. 485-488, 1989
Yasuhara, K., and Andersen, K.H., Recompression of Normally Consolidated Clay, Soils and Foundations. Japanese Geotechnical Society, 1991
Yi, F., Nonlinear Cyclic Characteristics of Soils. Proceedings of GeoFlorida, Advances in Analysis, Modelling and Design, 2010
Youd, T.L., Compaction of Sands by Repeated Shear Straining. J Soil Mech Foundations Div 98(SM7):709-725, 1972
Yu, H., Guzman, C., and Ntambakwa, E., Consideration of Cyclic Degradation in Pile Foundation Design for Onshore Wind Turbines. American Society of Civil Engineers, 2016
Zhang, B., Muraleetharan K.K., and Liu C., Liquefaction of Unsaturated Sands. International Journal of Geomechanics 16(6), 2016
Zhang, Y., Andersen, K.H., and Jeanjean, P., Verification of a Framework for Cyclic p-y Curves in Clay by Hindcast of Sabine River, SOLCYP and Centrifuge Laterally Loaded Pile Tests. In Applied Ocean Research 97, February 2020
Zhou, J., and Gong, X., Strain Degradation of Saturated Clay under Cyclic Loading. Canadian Geotechnical Journal, 2001