

Advances in Foundation Design and Assessment for Strategic Renewable Energy

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## ADVANCES IN FOUNDATION DESIGN AND ASSESSMENT FOR STRATEGIC RENEWABLE ENERGY

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### ABSTRACT

In order to meet EU legislation on emissions, significant effort is being invested into the development of cost-effective renewable power generation technologies. The two leading technologies are solar and wind power because of their potential for the lowest levelised cost of energy and for showing a growth in installed capacity and technological development. Various research findings have suggested that significant cost savings in the capital expenditure of renewable energy projects can be made through the optimisation of their support foundations, the understanding of which has formed the main goal of the research. In order to realise this potential, research into various aspects of the substructure and foundation-related technical challenges has been undertaken by the research engineer. The main focus has been related to addressing the technical challenges which apply to these structures as a result of rapid development within the area of offshore wind into increasingly deeper waters, further from shore and with larger wind turbine generators in order to increase energy yields.

One technical issue has been the deployment of grouted connections between the monopile and the transition piece, with insufficient axial capacity, resulting in conditions detrimental to wear occurring between the monopile and the transition piece. Experimental testing and a numerical model calibrated against data from structural condition monitoring of a full-scale site foundation have been developed and undertaken by the research engineer to demonstrate the potential significance of this wear to the grouted connections integrity over their remaining design life. The findings here have resulted in a methodology that can be used throughout the offshore wind industry to determine likely wear rate magnitudes.

As part of understanding the implications of the insufficient capacity, structural condition monitoring has been installed. The analysis of the collected data and comparison with design loads has formed another objective of this research in order to ascertain the robustness of the original design. The proposed model has shown good agreement between measured environmental loads, structural response and wind turbine generator thrust coefficients.

Another aspect of future development sites is to identify potential wind turbine generator foundation concepts that can provide lower capital expenditure for sites in deeper waters further from shore. Evaluation of a novel concept was therefore undertaken in order to assess its suitability as a cost-competitive offshore meteorological mast and wind turbine generator foundation. This was achieved through a detailed design review, lessons learnt from the installation, two years of operational experience of the foundation and benchmarking with traditional and other novel foundation concepts for site-specific details, highlighting its potential competiveness for lower lifecycle costs in certain site conditions.

A smaller focus of the research was to optimise the foundations for concentrated solar power parabolic trough arrays, but this was limited due to the changing nature of the energy market and reducing technology competiveness. Through a more detailed understanding of the variable wind loading, a significant optimisation of the design was possible, while still maintaining sufficient factors of safety, resulting in substantial capital savings.

#### **KEYWORDS**

Wear, Grouted Connections, Structural Condition Monitoring, Offshore Wind, Wind Turbine Foundations, Concentrated Solar Power Foundations, Wind Loading, Renewable Energy Technologies.

# **ACRONYMS / ABBREVIATIONS**

ABS	American Bureau of Shipping
CapEx	Capital Expenditure
CFD	Computational Fluid Dynamics
CSP	Concentrated Solar Power
DLC	Design Load Case
DNV	Det Norske Veritas
EC&R	E.ON Climate and Renewables
ECD	Extreme Coherent Gust with Direction Change
EDC	Extreme Direction Change
ENT	E.ON New Build and Technology
EOG	Extreme Operating Gust
ETG	E.ON Technologies (Ratcliffe) Limited
ETM	Extreme Turbulence Model
EWEA	European Wind Energy Association
EWM	Extreme Wind Speed Model
EWS	Extreme Wind Shear
FEM	Finite Element Method
FLS	Fatigue Limit State
FMEA	Failure Modes and Effects Analysis
GC	Grouted Connection
HSE	Health, Safety and Environment
JIP	Joint Industry Project
LCoE	Levelised Cost of Energy
LVDT	Linear Variable Differential Transformer
MM	Meteorological Mast
MOU	Mobile Offshore Units
MP	Monopile
MWP	Marcon Wind Power
NTM	Normal Turbulence Model
NWP	Normal Wind Profile Model
O&M	Operation and Maintenance
OEM	Original Equipment Manufacturer
OWF	Offshore Wind Farm
OPC	Offshore Pre-Construction
OpEx	Operational Expenditure
OWA	Offshore Wind Accelerator
PC	Portland Cement
PLF	Partial Load Factor
PMF	Partial Material Factor
RP	Research Package
RES	Renewable Energy Services
SCA	Solar Collector Assembly
SCADA	Supervisory Control and Data Acquisition
SCF	Stress Concentration Factor
SCM	Structural Condition Monitoring
SLS	Service Limit State

- SS
- Svensk Sjöentreprenad Technology and Innovation Transition Piece T&I
- TP
- Ultimate Limit State ULS
- Wind Turbine Generator WTG

# NOMENCLATURE

$\rho_a =$	Air density
$ ho_g$ =	Grout density
$\rho_s =$	Steel density
$\mu =$	Coefficient of friction
<i>E</i> =	Strain
$\sigma =$	Stress
$A_s =$	Swept area of rotor blades
$C_t =$	Coefficient of thrust
$C_{fx} =$	Horizontal force coefficient
$C_{fz} =$	Vertical force coefficient
$C_{my} =$	Pitching moment coefficient
$D_p =$	Pile outer diameter
$D_s =$	Sleeve outer diameter
$E_s =$	Elastic modulus of steel
$E_g =$	Elastic modulus of grout
$f_c =$	Compressive strength of concrete
$f_t =$	Tensile strength of concrete
$F_a =$	Axial force
$F_{bu} =$	Ultimate bond strength
$F_c =$	Compressive force
$F_x =$	Horizontal Force
$F_z =$	Vertical Force
I =	Second moment of area
L =	Length of the collector
$L_g =$	Length of the grout
M =	Applied moment
$M_{tp1.5} =$	Moment at TP 1.5m above top of MP
$M_y =$	Pitching moment
$p_{local} =$	Local stress
p <sub>nom</sub> =	Normal stress
q =	Dynamic pressure
$R_p =$	Radius of monopile
$R_{tpi} =$	Inner radius of transition piece
$R_{tpo} =$	Outer radius of transition piece
$t_g =$	Thickness of grout
$\tilde{t_p} =$	Thickness of MP (pile) wall or plate
$t_s =$	Thickness of TP (sleeve) wall or plate
T =	Thrust
$V_{hub} =$	Wind speed at hub height

- Cut-in wind speed
- Cut-out wind speed
- $V_{in} = V_{out} = V_{r} = V_{ref} =$ Rated wind speed
- Reference wind speed
- Upstream wind speed
- $V_{\infty} = V =$ Wind speed at pivot height
- W =Collector surface height
- Distance to neutral axis Z =

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## LIST OF PUBLICATIONS

The following publications, included in the appendices, have been produced in partial fulfilment of the award requirements of the Engineering Doctorate during the course of the research.

#### JOURNAL PAPERS

#### PAPER J1 (SEE APPENDIX C)

Dallyn, P., El-Hamalawi, A., Palmeri, A. and Knight, R., 2015. Experimental Testing of Grouted Connections for Offshore Substructures: A Critical Review. *Structures*, Vol. 3, pp. 90-108.

#### PAPER J2 (SEE APPENDIX D)

Dallyn, P., El-Hamalawi, A., Palmeri, A. and Knight, R., 2016. Experimental Investigation on the Development of Grout Wear in Grouted Connections for Offshore Wind Turbine Generators. *Engineering Structures*, Vol. 113, pp. 89-102.

#### PAPER J3 (SEE APPENDIX E)

Dallyn, P., El-Hamalawi, A., Palmeri, A. and Knight, R., 2016. **Prediction of Wear in Grouted Connections for Offshore Wind Turbine Generators.** *Structures (Under Review).* 

#### **CONFERENCE PAPERS**

#### PAPER C1 (SEE APPENDIX F)

Dallyn, P., El-Hamalawi, A., Palmeri, A., 2012. Cost-Effective Parabolic Trough Foundations for Concentrated Solar Power Plants. Proceedings of the 3rd International Conference on Engineering, Project and Production Management 2012, 10-11 September, Brighton, UK, pp. 227-238.

#### **TECHNICAL REPORTS**

#### **REPORT T1 (SEE APPENDIX G)**

Dallyn, P., 2014. Marcon Wind Power Offshore Wind Foundation Review. E.ON Technologies Limited, ETG/14/TCG/PB/706/R.

#### OTHER PUBLICATIONS PRODUCED IN THE COURSE OF THE STUDY BUT NOT INCLUDED IN THE DISCOURSE:

#### **CONFERENCE PAPER C2**

Dallyn, P., El-Hamalawi, A., Palmeri, A., Knight, R., and Morris, A., 2012. Wear in Large Diameter Grouted Connections for Offshore Wind Energy Converters. *Proceedings of the 10th International Conference on Advances in Steel Concrete Composite and Hybrid Structures*, 2-4 July 2012, Singapore, pp.638-645.

#### **CONFERENCE PAPER C3**

Dallyn, P., El-Hamalawi, A., Palmeri, A. and Knight, R., 2015. Development of Numerical Model to Predict Wear in Grouted Connections for Offshore Wind Turbine Generator Substructures. *ICMSE 2015: International Conference on Materials and Structural Engineering*, 12-15 February 2015, Kuala Lumpur.

#### **TECHNICAL REPORT T2**

Dallyn, P. and Knight, R., 2014. Assessment of Grout Wear in Robin Rigg's Wind Turbine Substructures Grouted Connections. *E.ON Technologies Limited*, *ETG/14/TCG/PB/1307/R* 

#### **TECHNICAL REPORT T3**

Dallyn, P. and Cassie, P., 2011. **Review of Carbon Trust Offshore Wind** Accelerator Programme Foundation Concepts. *E.ON New Build and Technology*, *ENT/11/TCC/PB/1708/R* 

#### **TECHNICAL REPORT T4**

Dallyn, P. and Cassie, P., 2013. **Review of Foundation Concepts for Rampion Offshore Wind Farm**. *E.ON New Build and Technology, ENT/13/TCC/PB/346/R* 

#### **TECHNICAL REPORT T5**

Dallyn, P. and Cassie, P., 2014. **Design Documentation Review of Marcon Wind's MWP Mark 2 Meteorological Mast for Södra Midsjöbanken**. *E.ON Technologies Limited*, *ETG/14/TCG/PB/705/R* 

#### **TECHNICAL REPORT T6**

Dallyn, P. and Cassie, P., 2011. Concentrated Solar Power Cost Effective Foundation Solutions. *E.ON New Build and Technology*, *ENT/11/TCC/PB/1614/R* 

#### **TECHNICAL REPORT T7**

Dubal, M. and Dallyn, P., 2012. Wind Loading on Arrays of CSP Troughs. E.ON New Build and Technology, ENT/12/CNS/PM/744/R

#### **TECHNICAL REPORT T8**

Dallyn, P. and Cassie, P., 2012. Comparison of Wind Loading Modelling Between Wind Tunnel Testing and Computational Fluid Dynamics (Project ECR 11-3) on Parabolic Troughs for Concentrated Solar Power Plants as Part of the CSP 2.1 Project. *E.ON New Build and Technology, ENT/12/TCC/PB/1052/R* 

## **1 BACKGROUND TO THE RESEARCH**

This chapter provides a short introduction to the general subject area. The context of the research in the renewable energy industry is described and the Industrial Sponsor, E.ON UK, is introduced, placing into context all subsequent chapters of the thesis and highlighting the overall goal and scope of the research.

The thesis summarises the research undertaken between 2010 and 2014 in partial fulfilment of the requirements for the award of the Engineering Doctorate (EngD) at Loughborough University (UK). The research was funded by the Engineering and Physical Sciences Research Council (EPSRC) through the Centre for Innovative and Collaborative Construction Engineering (CICE) and sponsored by E.ON UK. Data and industrial context for the research were provided by E.ON Climate and Renewables (EC&R).

The CICE is one of the research centres that forms part of Loughborough University's School of Civil and Building Engineering and was established with the aim to be focal point for addressing the industry's research and development needs through the supply of specialist staff that will bring about a step change in industry practice within the research areas of:

- Innovative Construction Technologies
- Construction Business Processes
- Advanced Information and Communications Technologies
- Sustainable Design
- Transport and Infrastructure.

The centre has supported 151 innovative EngD research projects in partnership with more than 100 different sponsoring companies. The EngD is designed to produce doctoral graduates who can drive innovation in the construction engineering industry with the highest level of technical, managerial and business competence. It is an alternative to the PhD, being better suited to the needs of industry, and providing a more vocationally oriented doctorate in engineering. At the core of the degree is the solution of one or more significant and challenging engineering problems with an industrial context.

As per the University's requirements, the EngD submission is in the form of a discourse, which sets out the aim, objectives, findings and industrial relevance/impact of the research work undertaken, and has to include a minimum of three published/accepted for publication papers (peer-reviewed), of which at least one should be in an appropriate engineering journal. As a series of mini-projects have been undertaken during the EngD research, the discourse is crucial in demonstrating coherence and adherence to a common theme. As some of the projects and outcomes are subjected to a confidentiality agreement, a redacted technical report has been included in place of a paper. These supporting publications can be found in the Appendices and the reader is guided to them throughout the thesis for more detailed information and explanations. A summary of these publications is shown in Table 1.3.

This thesis describes research from four distinct projects, each with their own individual aim and objectives, in order to meet the overall goal of optimising strategic renewable energy foundations. The relationship between these research projects, their aim, objectives, methods and outputs are shown in Table 1.2.

The thesis has five main Chapters, as required by Loughborough University's CICE regulations and follows their template, although it should be noted that this is not well suited to present and discuss the four distinct projects. The five chapters are:

• Chapter 1 discusses the general subject domain and the context of the research, including details of the industrial sponsor;

- Chapter 2 highlights the novelty of the work, aim and objectives of the projects, justification and scope for the research;
- Chapter 3 provides a brief review of research methods and details the development of the methodology (including the experimental procedures) for each research project;
- Chapter 4 provides an overarching discussion of the research development and details of the research undertaken;
- Chapter 5 highlights the key findings and implications of the research, including the contributions to the existing industrial and theoretical knowledge, critical evaluation, and recommendations for industry and future areas of research.

#### **1.1 THE GENERAL SUBJECT DOMAIN**

The European Commission (2009) has committed to a target of cutting emissions to 20% below 1990 levels and to achieve 20% of energy production from renewable sources by 2020, with "excess emission penalties" for member states which exceed these targets. The UK government has also set a target of 15% of the UK's energy production from renewable sources by 2020 (Department of Energy and Climate Change, 2011). For these reasons, a significant effort is being invested by government-funded organisations, academic institutions and industry to meet these targets. This has also motivated government support into the development of efficient zero-emission energy production technologies, which not only will help to reduce emissions, but will also provide a broader energy generation mix, adding increased security and capacity to the power networks. One area that poses a significant challenge to the widespread use of renewable energy is the relatively high levelised cost of energy (LCoE) when compared to conventional combustion power generation at current fuel prices, (see Figure 1.1), making commercial deployment economically unviable without government incentives (U.S. Energy and Information Administration, 2014; Department of

Energy and Climate Change, 2012; Fraunhofer ISES, 2013). There is therefore a drive across the European industry to reduce the capital and operational expenditure (CapEx and OpEx) of renewable energy technologies.



Figure 1 Levelised cost estimates for projects starting in 2012, 10% discount rate, sensitivities (Department of Energy and Climate, 2012)

As commercial renewable energy capture often consists of multiple smaller energy capturing devices, there are often multiple similar substructures/foundations across the field (array) of renewable energy structures on the site and therefore small optimisations can easily lead to considerable CapEx savings. Indeed, up to 30% of the CapEx cost is often related to the foundations/substructures of renewable energy devices (National Renewable Energy Laboratories, 2013 and Vallentine et al., 2009). For this reason, all of the research aims of this EngD project have been related to the optimisation of substructures or foundations of renewable energy structures.

Wind technologies are currently one of the most promising areas of renewable energy, representing the world's fastest growing energy source in terms of electricity generation (National Renewable Energy Laboratory (NREL), 2015). On the 18<sup>th</sup> of October 2014, the wind sector set a new record of meeting 24% of the U.K.'s power share over the course of a day, with peak production of 8GW (Murry, 2014). In the E.U., wind energy has been

increasing by an average of 27% per year for the last 10 years, and by 2030 it is predicted that 60% of this growth will be in the offshore market (Bilgili, Yasai and Sinsek, 2010), with the U.K. currently being the European leader in terms of installed offshore capacity (European Wind Energy Agency, 2015), Table 1.1.

Table 1.1 2014 European offshore wind installed capacity (EWEA, 2015)

COUNTRY	UK	DK	DE	BE	NL	SE	FI	IE	ES	NO	РТ	TOTAL
INSTALLED CAPACITY (MW)	4,494	1,271	1,049	712	247	212	26	25	5	2	2	8,045

Concentrated solar power (CSP) is another renewable energy technology that has the potential for significant contributions to energy production, also allowing a broader energy mix. As can be seen in Figure 1.2, approximately 20GW of power production exists in various stages of development in 20 countries (U.S. Department of Energy, 2011). There are four main commercial types of CSP systems that use the sun's energy as a heat source by concentrating the sunlight onto solar receivers. These are: parabolic trough, dish/engine, linear Fresnel reflector and power tower. Among these technologies, parabolic troughs are the most mature, and therefore have experienced significant commercial development in order to optimise the technology and reduce the overall cost of energy (Price et al., 2010; Kearney, 2007). This is due to their relatively simple and robust technology, utilising conventional steam turbines and associated plant, therefore requiring relatively low development and capital investment. In addition to this, the option of energy storage through thermal salt stores allows increased overall energy production. However, this is not the case in the UK market, due to the relatively low solar energy potential and the government subsidies resulting in photovoltaic technology having greater commercial viability. The parabolic trough technology formed one part of the research projects and is described in detail in Section 2.4.1, with pictures of the different CSP technologies shown in Figure 2.2.



Figure 2 Worldwide CSP development (U.S. Department of Energy, 2011)

### **1.2 THE INDUSTRIAL SPONSOR**

E.ON is one of the world's largest investor-owned power and gas companies, with facilities across Europe, Russia, and North America and more than 72,000 employees, generating approximately I 32bn in sales in 2012 (E.ON, 2014). In addition, there are businesses in Brazil and Turkey managed jointly with partners. E.ON's diverse business consists of renewables, conventional and decentralised power generation, natural gas, oil and gas exploration, energy trading, retail and distribution. With their broad energy mix, the company owns almost 68GW of generation capacity and is one of the world's leading organisations in the renewables market. They supply around 35 million customers with energy around the world. They have an ambitious objective: *to make energy cleaner and better wherever they operate*, striving to transform E.ON into a global provider of specialised energy solutions, which will benefit their employees, customers, and investors alike (E.ON, 2014).

To support this strategy, E.ON Climate and Renewables (EC&R) and the Technology and Innovation (T&I) divisions play a key role. EC&R contributes to the development of the renewable industry worldwide and has already invested €8,000M in renewable generation projects from 2010 to 2014. E.ON currently operates over 9GW of renewable capacity and

plans further multi-billion euro investments to grow its installed capacity. Today, E.ON is already active in generating energy from onshore and offshore wind, biomass, hydropower, photovoltaic (PV) and CSP. The T&I renewables division aims to further improve operations of existing renewable assets and to explore new technologies, such as tidal energy or advanced photovoltaic solutions, covering the main areas of:

- Development and improvement of wind generation technologies;
- Offshore renewable technologies;
- Development of concentrated solar power expertise;
- Exploring energy from biomass;
- Increasing competitiveness of Solar Photovoltaic.

All the research undertaken as part of the fulfilment of this Engineering Doctorate has formed part of projects operated by EC&R with the research funded by T&I, falling under offshore renewable and CSP technologies.

### **1.3 THE CONTEXT OF THE RESEARCH**

The overreaching objective of the EngD was to undertake research that will aid in the development of strategic renewable energy foundations with the aim of reducing the levalised cost of energy (LCoE) through optimisation and ensure a competitive advantage for the sponsoring company. Figure 1.3 highlights how the focus of the research has evolved over the course of the Engineering Doctorate, with the initially broader remit of civil engineering developments within the renewable energy sector leading to four distinct research packages that relate to offshore wind and CSP foundations. The detailed aim and objectives for each of these packages can be found in Chapter 2. This development has been necessary to meet the changing requirements of the company as the financial viability of the renewable technologies

changed, either due to improvements in efficiency, lowering costs or changing government incentives (e.g. E.ON's decision to focus investment on PV rather than CSP solar technology as a result of reducing unit cost and increased efficiency of the PV panels enabling a lower LCoE to be achieved within Europe). Only the areas labelled RP1 to 4 were research of sufficient novelty and high impact to warrant being covered within this thesis.



Figure 3 Research context

RP	AIM	OBJECTIVE	METHODS	OUTPUT	
wind	ear ler sed vind	Gain knowledge and understanding of the current testing used by the grout supplier.	Meeting, interviews	Presentation DNV+Densit	
RP1 - Effect of grouted connection wear on offshore v turbine foundations	sk to offshore tegrity due to we I connection und pically experienc ion of offshore v	Develop an understanding of grout behaviour and operational conditions experienced by the grout during loading of offshore wind turbines, to determine factors that might influence grout wear.	Lit. review Data analysis	JP1, CP2	
	Identify the ri foundation in of the grouted conditions ty during operat	Develop relationships between the factors that affect wear and the wear rate of grout.	Experimental testing Case study Numerical model	JP2, JP3, CP3, T2, Presentation DNV JIP+WOFF	
RP2-Examining loads in offshore wind turbine foundations	of loads urbine	Understand the initial design cases used for the design of the foundations that have the condition monitoring instrumentation installed on them	Lit. review	_	
	standing or standing of the standard to the st	Understand actual loading experienced over monitored periods based on analysis of the condition monitoring data	Case study Data analysis Statistical analysis	JP3, CP3	
	aprove the under cienced in offsho lations.	Develop a relationship between structural loads experienced and wind data to provide the tools to determine the potential load experienced over the life of the plant based on historical wind data	Numerical model		
	To ir expe foun	Determine the conservativeness in design loads provided by the WTG OEM.	Numerical model	T2	
RP3-Assessment of the SS Mk. 2 foundation concept Sufficiently de-risk the	isk the ept for 1 farm	Understand the commercial maturity of the meteorological mast MWP foundation.	Lit. review Data analysis	T5	
	tently de-r Mk. 2 tion conce ment in a ercial wing	Understand the remaining risks associated with the MWP foundation for use as a WTG foundation	Lit. review Data Analysis	T1, T4, T3	
	Suffici MPW founda deploy comm	Determine the optimal structural monitoring Ling of the campaign to de-risk the MWP foundation Discussion		T5, T1	
: solar n	ons	Determine foundation types and design currently used as parabolic trough foundations.	Lit. review	_	
araboli s' desig	ssign of oundati ar field	Understand the relationship between wind load and location within the solar field.	Lit. review, Numerical model	C1, T6	
timising para	mise the de ic trough f, out the sol	Determine if the foundations typically used can be optimised.	Lit. review, Numerical model, Case study		
RP4-OF trough f	To optir paraboli through	Understand the possible uncertainty in the wind loading used for the foundation optimisation.	CFD analysis	Τ7, Τ8	

#### Table 1.2 Research map

PUBLICATION	REF.	STATUS	Synopsis
Experimental Testing of Grouted Connections for Offshore Substructures: A Critical Review	J1, App. C	Published, peer reviewed journal, <i>Structures</i> , Vol. 3, pp. 90-108	Critical review and evaluation of findings from laboratory testing undertaken on grouted connections under axial, bending or combined or dynamic loading, presented along with implications and recommendations based on the findings
Experimental Investigation on the Development of Grout Wear in Grouted Connections for Offshore Wind Turbine Generators	J2, App. D	Published, peer reviewed journal, <i>Engineering</i> <i>Structures</i> , Vol. 113, pp. 89-102	Detailed laboratory methodology and results for grout wear testing presented for samples with vary conditions relevant to offshore wind farms. Including non- corroded, corroded, submerged and above water, millscale and shot blasted
Prediction of Wear in Grouted Connections for Offshore Wind Turbine Generators	J3, App. E	2 <sup>nd</sup> round of peer reviewed journal <i>Structures</i>	Summary of experimental findings, detailed development of numerical model to predict wear, calibration with case study structural condition monitoring data and results of wear across a case study wind farm presented
Cost-Effective Parabolic Trough Foundations for Concentrated Solar Power Plants	C1, App. F	Published, peer reviewed conference, Engineering, Project and Production Management 2012	Summary of literature review on CSp technologies, wind loading and foundation types, development of wind loading numerical model based on publically available wind tunnel data, evaluation of foundation design based on industry case study and optimisation potential
Marcon Wind Power Offshore Wind Foundation Review	T1, App. G	Published, peer reviewed report	Summary of development, met mast detailed design review, proposed structural monitoring campaign and evaluation as an offshore WTG foundation, along with recommendations for future work / engagement presented
Wear in Large Diameter Grouted Connections for Offshore Wind Energy Converters	C2	Published, peer reviewed conference, Advances in Steel Concrete Composite and Hybrid Structures	Sample preparation, laboratory testing equipment design, manufacture and initial methodology for determination of wear presented
Development of Numerical Model to Predict Wear in Grouted Connections for Offshore Wind Turbine Generator Substructures	C3	Published, peer reviewed conference, International Conference on Materials and Structural Engineering	Analysis of case study structural condition monitoring data, along with initial development and calibration of model to predict wear.

#### **1.3.1** OFFSHORE WIND

In order to decrease the LCoE, developers are siting larger wind turbine generators (WTG) further offshore, to exploit higher wind yields. For example within E.ON's offshore wind portfolio, it can be seen in Figure 1.4 that there is a strong positive trend of increasing turbine size and water depth.



**Figure 4 E.ON's offshore wind portfolio, water depths and turbine size vs. start of operation** Current fixed-base foundations are expected to be cost-competitive up to water depths of around 60 to 70m, after which floating foundations are likely to be the preferred solution. The choice of fixed-base foundation is affected by WTG size, water depth, soil properties of the seabed, wind and wave conditions. So far, there is no single foundation type suitable for all kinds of conditions. Among the various types of foundations, the most utilised are monopile, jacket and gravity foundations. Other types of foundation, especially for deepwater application (greater than 40m), are still at a prototype stage and further research and development are necessary to achieve cost-efficient solutions suitable for commercial production and installation.

At present, there are five types of foundations considered financially viable for deployment in water depths greater than 15m, shown in Figure 1.5, and represent the medium term development that are likely to be commercially deployed up to 2025. These are:

- 1. Concrete gravity base;
- 2. Monopile;
- 3. Suction bucket;
- 4. Tripod;
- 5. Jacket.





To accommodate increasing water depths found further offshore and larger WTGs, the substructures and foundations are becoming larger, to an extent which is beyond limits of previous testing and experience, resulting in significant knowledge gaps within the industry and academia. Some examples include representative fatigue curves, corrosion protection effectiveness, large diameter pile-soil interaction and large diameter pile wave loading. Through participation in industry-led technical working groups for both fixed and floating foundations (e.g. the Carbon Trust's Offshore Wind Accelerator Program, Wind Offshore Foundation Forum and the Energy and Technology Institute's wave hub demonstrator programme), along with the technical and commercial review of various current and future foundation concepts as part of the Engineering Doctorate, considerable knowledge has been acquired on the current and future challenges faced in the offshore wind energy industry.

Various research projects across the industry and academia are addressing these challenges and highlighting further knowledge gaps, such as:

- Wave Impact on Fixed turbine structures (WIFI);
- Pile Soil Analysis (PISA) for soil-structure interaction of large diameter monopiles;
- Det Norske Veritas Joint Industry projects (DNV JIP) 1 and 2 on grouted connections;
- Wind Offshore Foundation Forum (WOFF), for various challenges arising during the operation;
- Structural and Lifecycle Industry Collaboration (SLIC), focused on improvement to fatigue S-N curves for conditions relevant to offshore wind turbine structures.

In addition, with increasing operational experience of aging plants, more cases of unexpected structural behaviours due to inaccuracies, lack of modelling or inappropriate assumptions are being reported. The axial capacity of grouted connections (GC) used in monopile (MP) foundations was identified as an important area where inappropriate scaling of factors and design equations were used beyond the limits from which they were originally derived.

#### **1.3.1.1 Existing offshore wind foundation challenges**

Currently, 75% of all offshore WTG installations are founded on monopiles, and the most widely used connection method is a grout annulus between the transition piece (TP) and the MP (EWEA, 2014). A typical arrangement of an offshore WTG MP with a straight-sided GC without shear keys is shown in Figure 1.6. The concept of GCs has been extensively used in the oil and gas and offshore wind energy sectors, as they offer an efficient connection between the piles driven into the seabed and the topsides substructure, while accommodating relatively significant installation tolerances. However, in comparison to grouted connections

used in oil and gas platforms, offshore WTG connections have considerably lower radial stiffness, with pile diameter to thickness ratios greater than 85, compared to 45 typically found in oil and gas GCs. Furthermore, lower connection length to diameter ratios exist in WTG connections, having generally 1.5 times pile diameter overlap compared to oil and gas connections with up to six times overlap. There is also a significant difference between the relative amount of moment to axial loads, with WTG GCs typically experiencing twice the proportion of moment to axial force when compared to a quarter in oil and gas connections.



Figure 6 Typical layout of an offshore WTG GC

The load transfer mechanism experienced by the grouted connections during operation is shown in Figure 1.7. The axial capacity of the connection is developed mainly as a result of surface irregularities between the grout and the steel creating a shear resistance from the friction and mechanical interlocking of the irregularities. In addition, the initial capacity also consists of a small contribution from bonding between the grout and steel. The connection also experiences a compressive force between the grout and the steel when transferring the bending moment which increases the axial capacity of the part of the connection in compression.



Figure 7 Simplified WTG structure bending and grouted connection load transfer mechanism

In 2009 it was reported that unexpected settlements had occurred at Egmond aan Zee wind farm in the Netherlands, in the GC between the MP and TP. This event culminated in September 2009 with DNV issuing a letter to offshore wind farm (OWF) operators and other stakeholders, highlighting concerns that there may be insufficient axial capacity of the grouted joints. This can be explained by the overturning moment creating an axial force as a result of the lever arm around the centre of the connection's diameter, which at some point overcomes the axial capacity of the GC, resulting in relative displacement between the steel and the grout. An OWF, owned and operated by EC&R has been shown to be affected by this issue. The lack of axial load transfer between the grout and steel has resulted in significant relative movement between the TP and MP, which has been shown by condition monitoring to be occurring under large moment events resulting in large compressive stresses between the grout and MP. Combined with presence of water being observed between the grout and MP outer surface, this unexpected movement and compressive stress have the potential to promote wear. Hence, a better understanding of this phenomenon is of key importance to determine

the risk of this failure mode to the long term integrity of the structure. This has formed the basis of Research Package 1 (RP1).

A consequence of the insufficient axial capacity has been a progressive downward movement of the TP relative to the MP, which has resulted in the temporary installation jacking brackets coming into hard contact with the top of the MP on numerous foundations, as shown in Figure 1.6, and therefore transferring dynamic loads they were not designed for. Onsite structural condition monitoring (SCM) has therefore been installed to determine the implications of this unexpected settlement in the GC through measurement of strain, displacement, inclination, acceleration, humidity, temperature and oxygen. Such data have been used to better understand the loads and behaviour actually experienced by the offshore WTG foundation, and therefore establish any conservativeness in design loads. Analysis of the conservativeness can allow possible assessment of financial viability in replanting the foundation, or extension of the service life beyond the original design life of the foundations, increasing profitability of the asset. This has formed the basis for Research Package 2 (RP2). Conservativeness in the magnitudes of the strain responses experienced would also help to reduce the impact of the incorrect assumption of the corrosion environment used in the original design of the foundations. Across the industry it was assumed that the internal atmosphere of the transition piece would be a sealed compartment, therefore restricting corrosion as the quantity of oxygen is depleted by the oxidisation of the steel. In-air stress cycle amplitude to number of cycles to failure (S-N) curves were therefore applied for the steel fatigue calculations. In reality investigation through inspections, corrosion coupon analysis, along with analysis of atmospheric monitoring data undertaken during the course of the EngD have demonstrated that the compartments were not sufficiently sealed for oxygen depletion to occur. There was also no consideration of the need to inspect the top of the grouted connection or for the significant condensation that was found to be experienced within the TP. Therefore the steel is actually undergoing free corrosion and for a given stress magnitude, the S-N curve for free corrosion results in a significantly lower number of cycles until failure than the in air curve. This results in a significantly reduced fatigue life, with potential for some locations to be lower than the design life.

### **1.3.1.2 Future offshore wind turbine foundations**

In order to ensure cost competiveness and the lowest LCoE, E.ON and the offshore wind industry need to understand which novel and existing foundation concepts offer the lowest potential CapEx for their particular development portfolio site conditions, while being sufficiently developed to minimise the commercial risk. As part of this, technology tracking activities on various concepts have been undertaken as part of the Doctorate research. Through representing E.ON on the Carbon Trusts Offshore Wind Accelerator (OWA) Programme foundations technical working group, detailed knowledge of some of the leading concepts for future foundations has been gained. This also enabled opportunities to improve the company's in-house knowledge of offshore foundation design considerations, optimisations and areas where knowledge gaps could be filled with novel research. Some examples of the more developed concepts evaluated as part of the Carbon Trust's ongoing OWA programme, started in 2009, are shown in Figure 1.7.



Figure 8 Examples of concepts evaluated as part of the Carbon Trust's OWA programme (Sykes and Villiers, 2010)

One concept that showed potential for lower LCoE is the MWP MK. 2, making it to the final stages of the OWA selection process and winning a competitive tender for deployment as a meteorological mast (MM) at one of E.ON Nordic's development sites in the Baltic Sea. The foundation concept represents a novel solution that can be floated out and self-installed through the use of a jack-up mechanism. It therefore has the potential to be redeployed at a later date and if proved to be successful as a MM, could also be scaled to accommodate a WTG. Due to its self-jack-up nature it has great potential to minimise health, safety and environmental (HSE) risks, by reducing offshore activities and limiting environmental impact. The concept also has the potential to reduce CapEx and OpEx, as only a small sea tug is required for installation rather than expensive heavy lift vessels. Based on this potential, more research into the detailed design of the foundation and its suitability to E.ON's development sites was required and formed the third Research Package (RP3).

#### **1.3.2** CONCENTRATED SOLAR POWER

As part of developing E.ON's CSP market share, EC&R invested 275M in a 50% joint venture with the Spanish multinational corporation, *Abengoa*, to develop and operate two 50MW CSP plants in Spain (Helioenergy 1 and 2), which came into commercial operation in 2011 and 2012 respectively. One third of capital costs of these plants are in the solar field; given that for a typical 50MW site there are around 1,000 foundations, any cost reduction delivered through their optimisation has the potential to result in significant capital cost savings. The major factor that influences the structural design of the solar collector assembly (SCA) and foundation is the wind loading and therefore a thorough appreciation of its behaviour across the solar field is important. The SCAs must be able to withstand the wind loading imposed on them while still maintaining their optical accuracy. It has been shown that there is a considerable shielding effect by the first row of SCAs, (Naeeni and Yaghoubi, 2007; Holze et al., 2010 and Hosoya et al., 2008). The foundation design will thus vary

according to the field position of the SCAs, with the outer foundations being significantly larger in order to accommodate the higher wind loads generating larger overturning moments, which are then transferred to the foundations. There is therefore a need for development of suitable cost-efficient foundation designs for various soil types and load conditions, and research was required to better understand the complex wind loading on the array of the solar tracking parabolic troughs, allowing for optimisation of foundation design. This formed the research undertaken in Research Package 4 (RP4).

# 2 AIM AND OBJECTIVES

This chapter details the overreaching goal of the engineering doctorate, along with the aim and the objectives for the different research packages that have been undertaken.

The overall goal of the research was to support the development of strategic renewable energy foundations, helping to reduce the LCoE, with a competitive advantage for the sponsoring company E.ON. As discussed in the previous chapter, the research carried out can be broken down into four distinct research packages, detailed below.

## 2.1 RESEARCH PACKAGE 1 – EFFECT OF GROUTED CONNECTION WEAR ON OFFSHORE WIND TURBINE FOUNDATIONS

### 2.1.1 BACKGROUND

A result of the lack of understanding and rapid development, detailed in Chapter 1, were some of the issues that resulted in the insufficient axial capacity of the straight-sided GC used on offshore WTGs. Ascertaining the long term implications of this issue over the design life of the plant was of critical importance. A series of joint industry projects were set up to determine the cause of the insufficient capacity and produce revised standards for the design of GCs (Det Norske Veritas, 2010a). As a result, the revised design standards (DNV-OS-J101:2012) recommended nominal contact pressure to be less than 1.2MPa in order to limit the consequences of an abrasive wear failure mode. Robin Rigg, an offshore wind farm located in the Solway Firth, was chosen as a case study for part of the research undertaken as it has been affected by insufficient axial capacity and is owned and operated by EC&R. As part of RP1, a review and analysis of the specifications for Robin Rigg against the design equations of the time, indicated nominal contact stress greater than 1.2MPa. This, along with discussions within operator foundation forums (WOFF), suggested that the wear failure mode was potentially a source of concern, but that no research had previously been undertaken to
ascertain the significance of wear for the conditions found in the offshore WTG GCs. The literature review (published as J1) also highlighted the lack of experimental testing fully representative of the conditions experienced by the GCs during operation. It was therefore deemed necessary to increase the understanding of the developers, designers and standard bodies of the grout wear failure mode by undertaking research in this area. The research forms part of a larger grout failure modes and effects analysis (FMEA) project for the GC, which is funded by T&I due to the significant number of currently operational sites that utilise GCs across the industry.

## 2.1.2 AIM

Identify the risk to offshore foundation integrity due to wear of the grouted connection under conditions typically experienced during operation of offshore wind turbines.

### 2.1.3 OBJECTIVES

**Objective 1:** Gain knowledge and understanding of the current testing used by the grout supplier.

**Objective 2:** Develop an understanding of grout behaviour and operational conditions experienced by the grout during loading of offshore wind turbines, to determine factors that might influence grout wear.

**Objective 3:** Develop relationships between the factors that affect wear and the wear rate of grout.

## **2.1.4 JUSTIFICATION**

It was of critical importance to E.ON to ascertain the risk to structural integrity of the 62 foundations of the Robin Rigg wind farm, as these supported the WTG's with a combined capacity of 180MW along with two offshore substations. In addition, the elastomeric bearing remedial solutions proposed to address the insufficient axial capacity still relied on the grout

integrity to transfer the large bending moments and do not eliminate the relative movement between the MP and TP. To determine the risk of wear to the GC over its design life, it was necessary to verify the material specifications of the composite elements that make up the connection, as these would affect the structure's behaviour and wear rates. It was also necessary to understand what previous testing of the material had been undertaken to establish if any relevant testing related to wear and the operational conditions experienced by the material existed. It was therefore necessary to engage with the supplier of the grout material, which led to the development of Objective 1.

As well as understanding the materials of the connection and previous material testing, it was also necessary to identify the loads and conditions experienced in order to determine if previous testing had been undertaken under relevant conditions and therefore establish if any knowledge gaps and validation existed. By ascertaining the operational conditions experienced it would also help to identify variables that would be required to be investigated by the experimental wear testing. This led to the development of Objective 2.

With the knowledge gaps identified and the appropriate variables and conditions understood, the review of previous GC and wear testing highlighted a lack of relevant testing and therefore it was necessary to develop a test arrangement capable of applying the relevant conditions to representative samples of the material. Testing would enable appropriate wear rates for the conditions experienced by typical offshore WTG GCs to be determined, and in particular for those affected at the E.ON site by the unexpected continual relative movement between the MP and TP.

The nature of a WTG means that the loads experienced by the structures will vary both spatially and temporally due to the variation of wind speed, direction, wave height and direction and turbine operation, on a macro-scale between different sites and micro-scale around the wind farm. As a result, the compressive stresses and relative displacements experienced by each GC will also vary. Ideally direct measurements of these stresses and displacements at various locations around individual GCs throughout a wind farm would be required to provide the inputs to predict wear. However, the cost of such extensive structural monitoring over the scale of a typical commercial wind farm, with more than 20 of such structures, would be prohibitive from a commercial perspective, particularly when retro-fitted offshore. Development of a numerical model was therefore required to take advantage of existing readily available data (wind speed, direction, power generation, etc.) and apply the appropriate wear rates to allow for the prediction of the magnitude of wear over the design life of the WTG GC, in order to determine the risk to foundation integrity. This was the justification for the third objective.

## 2.2 RESEARCH PACKAGE 2 – EXAMINING LOADS IN OFFSHORE WIND TURBINE FOUNDATIONS

### 2.2.1 BACKGROUND

The design of the offshore wind turbine foundations installed at Robin Rigg, along with the majority across the industry, are based on Det Norske Veritas offshore standard for design of offshore wind turbine structures, DNV-OS-J101:2004 and subsequent updates. The basis for design is to check the structural capacity against ultimate, fatigue and service limit states, (ULS, FLS and SLS respectively). For the ULS, design load cases (DLC), as recommended in IEC 61400-3: Wind turbines, Part 3 – Design requirements for offshore wind turbines, are screened covering 570 governing cases, to determine which have the potential to impose the largest magnitude loading to the WTG structure and foundation. These DLCs include various wind, current and wave conditions, along with the WTG operating states. The current design approach requires that for each of these cases, the aerodynamic load generated by the WTG is incorporated to the loads induced by inertia of the WTG structure from wave loading

provided by the foundation designer (DNV, 2012 and Ramboll, 2007a). This results in a design loop where factors of safety to account for inaccuracies in modelling of wave loads on the foundation and the WTG wind loads are compounded. When these loads are inputted into the foundation designer's numerical models, additional partial load factors and material factors are applied and a check is performed to ensure that the applied stresses are less than the permissible yield stress of the material. For the FLS design, detailed fatigue analysis is undertaken consisting of dynamic time domain simulation with calculation of the fatigue damage induced by wind and wave probability distributions (consisting of superimposed wave and wind response time series) with post-processing to establish the total fatigue damage. Based on the yearly directional probabilities, the fatigue damage is then scaled to yearly damages. The fatigue damage is determined by the S-N curve approach incorporating the appropriate stress concentration factors (SCF) (DNV, 2010b) with the accumulation of damage ascertained based on the Miner's summation rule (Miner, 1945). As with the ULS approach, the wind loading time series is provided by the WTG original equipment manufacturer (OEM), and assumptions are made on the sites' characteristic wind speeds and direction probability distributions (DNV, 2012 and Ramboll, 2007b). All of this conservativeness inevitably has the potential to result in significant overestimation of the loads applied to WTG structures during their operational life, and ultimately in overengineered structures, with greater capital cost than necessary. There is therefore the potential for optimisation of new designs or the possibility that life extension at the end of the 20-year design life can be undertaken without significant increase in operation and maintenance (O&M) costs.

As a result of the unexpected settlement, structural condition monitoring (SCM) systems have been installed across the industry to monitor the behaviour of the grouted connection to determine the implication of this settlement and provide a better understanding of the structural behaviour. Some of these systems have indicated that the fatigue loads being measured in the TP are up to 30% lower than the design model predicted for the same conditions (Dong, 2012).

## 2.2.2 AIM

To improve the understanding of loads experienced in offshore wind turbine foundations.

### 2.2.3 OBJECTIVES

**Objective 1:** Understand the initial design cases used for the design of the foundations that have the condition monitoring instrumentation installed on them.

**Objective 2:** Understand actual loading experienced over monitored periods based on analysis of the condition monitoring data.

**Objective 3:** Develop a relationship between structural loads experienced and wind data to provide the tools to determine the potential load experienced over the life of the plant based on historical wind data.

**Objective 4:** Determine the conservativeness in design loads provided by the WTG OEM.

## 2.2.4 JUSTIFICATION

In order to assess whether there was conservativeness in the design approaches currently adopted, an understanding of these methods and how they were applied to the foundations with SCM installed was required.

The SCM installed on two of the WTG structures at EC&R's OWF offered the opportunity to demonstrate if there was conservativeness based on a case study of an operational wind farm's design for the site conditions the structures are subjected to. This would not only help to relieve other structural issues due to the choice of design S-N curves, but provide more evidence to substantiate the hypothesis that the stresses experienced by the offshore WTG

structures are considerably lower than current design predicts (DONG, 2012). The use of the SCM data allowed the direct measurement of the strain and therefore calculation of the utilisation of the steel for the actual conditions experienced by the WTG structure. Comparison with the measured environmental data would allow for derivation of relationships between key environmental properties and structural response. This enables a comparison to be made for given design conditions and historical weather data available for the site to be used to predict structural response for a given period.

# 2.3 RESEARCH PACKAGE 3 - ASSESSMENT OF THE SS MWP MK. 2 FOUNDATION CONCEPT

## 2.3.1 BACKGROUND

The MWP MK. 2 concept first appeared in its early stages of development as part of the initial Carbon Trust OWA Programme foundations competition in late 2009. It was one of the final seven concepts, but was not pursued further due to higher CapEx than any other design at £0.95million/MW, including both fabrication and installation costs of the foundation.

EC&R Nordic subsequently engaged Marcon Wind Power (MWP) to design and deploy an 80m high meteorological mast (MM) at their Södra Midsjöbanken site in a water depth of approximately 15m through a competitive tender. The MM foundation was deployed in April 2012, Figure 2.1.



Figure 9 Installed Södra Midsjöbanken Meteorological Mast (EMMA)

The basis of the concept is a three-legged jack-up foundation that floats to site, with the turbine or meteorological mast pre-installed at the harbour. The Y-shaped hull is designed to provide sufficient righting arm for stability, allowing it to be transported in reasonable sea conditions. After transporting the platform to the desired location, the legs are lowered to the sea floor, the hull jacked up out of the water, the water tank compartments ballasted to provide sufficient penetration of the legs into the sea bed and dead weight for operational conditions, before being jacked up to achieve the desired air gap. The key advantages are:

- Three-legged jack-up structure results in minimal environmental impact, with no piling or drilling noise and complete removal at decommissioning;
- Potential for integrated installation with complete construction onshore, eliminating the need for expensive heavy lift vessels, resulting in minimal installation vessel spread requirements and minimising time offshore;
- Minimising offshore lifting activities and time spent offshore reduces the health and safety risks;
- Potential for serial fabrication benefits through modular design;

• Relocation possible after two year meteorological monitoring campaign for the MM case.

The self-installing, re-deployable nature of the concept results in installation and transportation load cases driving the design, as well as the operation load requirements associated with traditional offshore MM and WTG foundations. Therefore adding additional complexity and introducing greater risk into the design process.

Given the advantages mentioned, the foundation concept could offer considerable potential benefits at future E.ON sites through cost savings as a result of installation optimisation. Therefore facilitation of knowledge transfer and ensuring optimum data capture of this demonstration project, as well as to investigate its potential as a full scale offshore WTG foundation were identified as beneficial and necessary to understand the risk of the concept.

# 2.3.2 AIM

Sufficiently de-risk the MPW Mk. 2 foundation concept for deployment in a commercial wind farm.

# 2.3.3 OBJECTIVES

**Objective 1:** Understand the commercial maturity of the meteorological mast MWP foundation.

**Objective 2:** Understand the remaining risks associated with the MWP foundation for use as a WTG foundation.

**Objective 3:** Determine the optimal structural monitoring campaign to de-risk the MWP foundation.

# 2.3.4 JUSTIFICATION

When introducing a new concept to market for commercial operation, a WTG foundation needs to have undergone testing and development to validate its performance and demonstrate its ability to achieve a cost-competiveness level of risk. From the initial conception, generic designs will be undertaken for representative site soil and loading conditions to determine if the concept is structurally feasible. If feasibility is demonstrated then more detailed design looking at fatigue and WTG stiffness requirements are carried out to produce estimates of material quantities and therefore approximate CapEx. Other studies looking at the sensitivity of the foundation to increased water depth, variable site conditions and next generation WTG sizes (6MW+) will all help to inform developers of potential suitability for the site-specific conditions of their development portfolio. Comparison of these findings with those for other concepts for the same conditions from studies undertaken across the industry through the OWA and E.ON, then allows benchmarking and insights into the potential competiveness of the concept. Assessment of the MWP concept as a MM and WTG foundation was therefore required to enable informed decisions to be made in supporting the development of the concept to the next step in commercial maturity or through to commercial deployment, so that cost savings can be realised for the company.

The novel nature of the MWP structure means it falls outside of the current industry and academic knowledge and specific design standards, therefore requiring further de-risking to ensure modelling assumptions and the resultant material quantity estimates are correct. This is important for the MWP structure as, unlike current offshore WTG foundations, the design load cases are not just governed by operation, but installation and transportation as well, due to its self-floating and installing nature. The large CapEx required along with the difficulty the industry is facing in securing demonstration sites, is prohibitive to full scale WTG foundation validation in relevant site conditions. A common development step in the offshore wind industry is therefore to deploy a MM foundation, as this imposes significantly smaller loads than full scale WTGs and so result in smaller and therefore significantly cheaper foundations. In addition, the industry standard for meteorological monitoring campaign is two years to make sufficiently accurate wind yield assessments, and so the commercial risk to a development project is minimised. The fabrication, installation and operation of these

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smaller scale foundations help to de-risk the relevant aspects and improve fabrication and cost estimates of further foundations. The assessment of this information then allows for the next financial decision to be made on the deployment of a full scale demonstration. This is required as the dynamic nature of the loads of a full scale WTG turbine are significantly different to those of a MM, and the fatigue and stiffness requirements of the foundation become the design drivers. Unfortunately at the time of deployment of the MWP foundation and MM, SCM was not installed. This decision had been taken due to the purely commercial nature of the project and previous incorrect advice by the concept designer that SCM would not be beneficial. However, retrofitting of SCM to the MWP foundation would have the benefits of:

- Structural validation;
- Stability validation during transport;
- Model validation of wave loading, soil-structure interaction, wind loading and FE models;
- Understanding conservativeness in design;
- Validation of design assumptions;
- Determining potential extension of service life;
- Potential broadening of transportation, installation and site conditions where the structure can be deployed.

However, the cost of retrofitting SCM to an offshore structure is expensive; therefore significant justification is required through assessment of the commercial competiveness as a WTG and MM foundation to demonstrate the return on investment.

In order for SCM instrumentation to be placed in the correct locations to extract useful information on the structure, a detailed knowledge of the design of the structure is required. This allows for areas with high stress utilisation to be identified and targeted for strain gauge locations, as it will ensure maximum signal to noise ratio, increasing the quality of the data. These areas are also the areas where detailed design will have been focused on and so there will be sufficiently detailed information to compare the monitored response with the model predicted response.

# 2.4 RESEARCH PACKAGE 4 - OPTIMISING PARABOLIC SOLAR TROUGH FOUNDATIONS' DESIGN

## 2.4.1 BACKGROUND

There are four main commercial types of CSP systems that use the sun's energy as a heat source through concentrating the sunlight onto solar receivers. The four types are parabolic trough, dish/engine, linear Fresnel reflector and power tower, as shown in Figure 2.2, with a comparison given in Table 2.1. The capital cost is only provided for the technologies that have been developed on a commercial scale (International Renewable Energy Agency, 2012).



TECHNOLOGY	EFFICIENCY (%)	GROUND WORKS	50MW PLANT AREA (ACRES)	CAPITAL COST (€/KW)
<b>POWER TOWER</b>	17	Intermediate	590	Low (6250)
PARABOLIC TROUGH	14	Extensive	240	Low (7500)
<b>DISH / ENGINE</b>	30	Minimal	500	High (N/A)
LINEAR FRESNEL Reflector	11	Extensive	120	Low (N/A)

Table 2.1 Comparison of typical CSP technologies

A joint venture between E.ON and Abengoa was undertaken to develop E.ON's constructional and operational experience in CSP generation. Of the CSP technologies Abengoa had in development for commercial production, parabolic trough technology was chosen for further research and development over power towers. This was because Abengoa's power tower projects at 20MW were considered too small to be economically viable, with operating and maintenance costs only becoming reasonable at greater system sizes of 30MW+. Therefore unlike parabolic troughs where scaling is just an issue of increased field area and capacity, power towers would require significantly bigger towers or multiple smaller towers to be developed on a larger scale, introducing added expense and complexity and making the economies of scale less beneficial. Parabolic troughs therefore offered the greatest potential for lowest LCoE of the CSP technologies.

Parabolic trough systems consist of parallel rows of troughs that have single axis tracking of the sun. They are curved in one axis to focus the sun's light onto an absorber tube that contains heat transfer fluid. This transfers heat via an exchanger to form steam to drive a conventional steam turbine power generation system.

### 2.4.2 AIM

To optimise the design of parabolic trough foundations throughout the solar field.

Figure 10 Four main commercial types of CSP plants: Abengoa Solar's PS10 power tower, Seville, Spain (Top left) (Abengoa Solar 2010); Sandia 10-kWe dish/String system (Top right) (Kearney, 2007); Linear Fresnel Reflectors at Kimberlina Solar power station, California, USA (Bottom left) (Areva Solar, 2010); Flagsol parabolic troughs at Kramer Junction (SEGS V), California, USA (Bottom right) (Kearney, 2007).

## 2.4.3 OBJECTIVES

**Objective 1:** Determine foundation types and design currently used as parabolic trough foundations.

**Objective 2:** Understand the relationship between wind load and location within the solar field.

**Objective 3:** Determine if the foundations typically used can be optimised.

**Objective 4:** Understand the possible uncertainty in the wind loading used for the foundation optimisation.

## 2.4.4 JUSTIFICATION

Given the large proportion of CapEx required for the solar field in a CSP plant, i.e. 31 to 35% (Vallentine et al., 2009), which for a typical 50MW parabolic field equates to around 1200 foundations (Abengoa, 2010). This would provide a huge potential to reduce the overall cost by optimising the design of the SCAs and foundations. The challenge arises in reducing material, weight, simplifying manufacture, and assembly while maintaining structural rigidity, as efficiency of the collectors is highly dependent on the optical accuracy (Kolb and Diver, 2008).

In order for a particular foundation type to be chosen and optimised, an understanding of the complex loading conditions experienced by the parabolic trough is required. It was found that limited work had been carried out to obtain a clear knowledge of the interaction of the troughs within an array and even less work to validate Computational Fluid Dynamic (CFD) models. The most extensive research to date had been carried out by the National Renewable Energy Laboratory (NREL) in the USA (Hosoya et al., 2008). It has also been shown that there is a considerable shielding effect by the first row of SCAs, (Naeeni and Yaghoubi, 2007; Holze et al., 2010 and Hosoya et al., 2008). This highlighted the variability of the loading and

therefore the potential requirement for foundation design to be varied according to the field position of the SCAs and so justifying the need for development of suitable cost-efficient foundation designs for various soil types and load conditions.

Based on reviews of current solar farms in operation, it could be seen that there is a variety of foundation types in use. There is not only variation in design concepts from plant to plant, but also within the site with perimeter foundations and inner field foundations to cater for the varying wind loading across the site. In order to establish the potential for optimisation, a detailed understanding of the specifications and foundation types commercially used were required to enable the foundation capacities for given soil conditions to be determined. Due to the significantly variable nature of the wind load across the solar field, there was large uncertainty in the wind loading, which resulted in conservative safety factors used in the foundation and SCA design. The variability of the wind load is due to the turbulence and shadowing of upstream collectors, the variable nature of the wind's velocity and direction and the complex parabolic trough shape, the angle of which varies throughout the day to track the sun. During high wind speeds they are also angled into a stow position to reduce the wind load experienced. By having a more detailed knowledge of the wind load variance across the solar farm for the various orientations of the SCA throughout the day, there is potential to reduce the highly conservative factors of safety, leading to optimisation of the design and material cost savings. Given the limited availability of detailed information on the wind loading of SCA relevant to the specifications and spacing of the SCA used at Helioenergy, additional work was undertaken to improve the accuracy of the wind load through CFD modelling.

# **3 ADOPTED METHODOLOGY**

This chapter includes a review of research methodology to determine the suitability of available approaches to meet the research aim and objectives. The methods adopted for each of the research objectives are then outlined along with their applicability and benefits. The specific methods adopted are then described and justified with reference to how they meet the research objectives.

## 3.1 METHODOLOGICAL CONSIDERATIONS

The research undertaken as part of the Engineering Doctorate falls within the applied area of research, in order to have practical applications within industry. The research involves solving problems, which in the majority of the research projects were open in nature, as they involved complex situations with multiple variables. The main priority is to ensure that the research maximises the chance of realising the objectives.

The work undertaken has consisted of four distinct research topics and in turn, four packages, each with their own aim and objectives. The research methods detailed in Section 3.3 concern the techniques available and that are employed in the research topics to meet the requirements of the objectives. The research methods chosen for each research topic are listed in Table 3.1. As well as the physical experimental work, a number of numerical investigations were also necessary as part of the research. This involved the development of computational and analytical representations of the phenomena being assessed in order to accurately capture and represent them. Statistical analysis involving the interpretation of data, normally in numerical form, was used to summarise and describe the collected data. These techniques were also used to identify and investigate patterns in the data in order to draw conclusions about the population under study with due consideration to the uncertainty and randomness in the observations.

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 Table 3.1 Research methods used in the Engineering Doctorate

## 3.2 METHODOLOGY DEVELOPMENT/REFINEMENT

This Chapter provides an explanation of why each type of methodology is applied to the research and provides a description of the development and refinement of the methodology of each research package where applicable.

# **3.2.1 RP1 - EFFECT OF GROUTED CONNECTION WEAR ON OFFSHORE WIND TURBINE FOUNDATIONS**

Investigation of previous research undertaken related to wear and experimental testing of grouted connections through a literature review, (J1), Appendix C, and meetings with industry experts highlighted a lack of detailed knowledge of the behaviour of the GCs, not only for the scale and size of actual structures, but also under the loading and environmental conditions of operation. Therefore experimental type-testing, involving the design and development of equipment and protocols (see section 2, J2 for details), was undertaken by the research engineer to quantify applicable wear rates and provide qualitative information on the wear and grout degradation. The testing apparatus (Section 3.3, Figure 3.3) was designed based on BS EN 1993-1: Eurocode 3 - Design of Steel Structures and integrated into an available testing frame, allowing variables of compressive stress, relative axial displacement, wetness, surface finish and grout properties to be tested within ranges representative of those found in offshore

GCs used in WTG foundations. The ranges were based on the findings of a review of the design documentation and analysis of SCM installed on two foundations at a case study site of E.ON's OWF, Robin Rigg. Experimental methods used for establishing material characteristics, behaviours and properties were in accordance with UK and/or European codes of practice.

Upon commencement of testing it was found that the originally proposed logging frequency for data acquisition resulted in excessively high quantities of data which could not be successfully exported using the StrainSmart® Software from the Vishay Precision Group 7000 data acquisition system. Refinement of the logging technique was therefore required in order to minimise the data quantity for a given test period, while maintaining sufficient data quality. This was achieved through reducing the logging frequency from the default value of 100Hz, to a frequency that was demonstrated to result in less than 1% error, 5Hz, Figure 3.1. Through the addition of window sampling, where data was recorded for 300s every 3600s (i.e. five minutes every hour), the quantity of data was reduced to 0.42% of the original quantity. The duration of the window sample was chosen so that a sufficient number of complete load cycles were captured, namely 10 cycles at 0.3Hz. The duration between windows was chosen to ensure sufficient resolution of data for the determination of non-linear trends and variations due to wear development.



Figure 11 Comparison of sampling frequency on accuracy of recorded test frame axial load

During testing of the first two samples it also became evident that under higher loads the lateral stability of the samples was insufficient, resulting in testing being stopped before the maximum load increments were reached. This was a result of fabrication tolerances required to allow for the compressive force to be applied while the samples were undergoing relative movements, resulting in a rotation if the axial friction across the width of the sample was not even. The implication of this was demonstrated through the analysis of the first sample test data for the four horizontal LVDTs fixed to the samples, shown in Figure 3.3.



Figure 12 Variance of horizontal LVDT (1-4) results before (top) and after (bottom) installation of lateral restraint

The analysis shown in the top graph of Figure 3.2 showed that under higher loads the variation of the results increased. Lateral guides, shown in Figure 3.3, were therefore

Adopted Methodology

incorporated into the sample to improve lateral stability and resulted in a decrease of the variation of results as shown in the lower graph of Figure 3.2.

In order to apply the wear rates derived from the experimental testing programme to ascertain the wear over the design life of a GC, given the viability of the loads experienced offshore, the development of a numerical model was required. This model needed to be able to determine the magnitude of the variables (local relative displacement, compressive stress and environmental conditions) from readily available site data (i.e. SCADA data), which were found to influence wear from the experimental testing. This therefore enabled the correct wear rate to be computed over the duration of the input data. As this tool was to be used to predict wear over the design life of the foundation, based on the variation of available historic environmental data, the output needed to be scaled appropriately to account for the full 20year design life.

## 3.2.2 RP2 - EXAMINING LOADS IN OFFSHORE WIND TURBINE FOUNDATIONS

A literature review was undertaken in order to highlight and quantify any potential for optimisation of design, as well as identifying the novelty of the research. A review of design code recommendations for the design of offshore wind turbine structures identified areas where possible sources of over-conservatism may exist. Through discussion with offshore wind farm operators and developers, the resultant likely magnitudes of these conservatisms were identified. Review of the initial design documentation for the foundations with SCM installed and the WTG specifications was required in order to understand the loads that were expected based on the design, and the environmental and structural factors that would affect them.

Initially it had been envisaged to compare the DLCs required by IEC 61400-3 (International Electrotechnical Commission, 2009) with monitored cases to allow a comparison of the

measured stresses with the stresses predicted by the designers. However, the review of the design documentation highlighted that only limited information was available within the issued design documents and focused on DLCs 2.3 and 6.1 only, as they provide the largest bending and horizontal loads respectively and are therefore design driving. The details of these DLCs are shown in Table 3.2, from which it can be seen that due to these DLCs requiring either an electrical fault or a 1 in 50 year event, the probability of them occurring within the period of available SCM data on either of the foundations was very small. This was also compounded by the limited period of SCM data available due to the relatively short period the SCM had been in operation and that there was only a very small duration of high frequency data available. In addition to this, the ability to acquire the design outputs for the other load cases which would occur on a regular basis was limited due the legal proceedings resulting from the insufficient axial capacity failures of the grouted connection involving the installation and design contractors. Therefore a comparison was made with the available information based on the WTG specifications (Vestas, 2009). The strains experienced by the structure could then be compared to the indirectly predicted strains based on the turbine and structure specifications. Although this proposed methodology would not allow the design driving load cases to be validated, it enabled the steady state typical operational loads to be compared by the research engineer to provide an indication of the conservatism in the loads assumed.

Design situation	DL C		Wind condition	Other conditions	Type of analysis	Partial safety factors
1) Power production	1.1	NTM	$V_{\rm in} < V_{\rm hub} < V_{\rm out}$	For extrapolation of extreme events	U	N
	1.2	NTM	$V_{\rm in} < V_{\rm hub} < V_{\rm out}$		F	*
	1.3	ETM	$V_{\rm in} < V_{\rm hub} < V_{\rm out}$		U	Ν
	1.4	ECD	$V_{hub} = V_r - 2 \text{ m/s}, V_r,$ $V_r + 2 \text{ m/s}$		U	N
	1.5	EWS	$V_{\rm in} < V_{\rm hub} < V_{\rm out}$		U	N
2) Power production plus occurrence of	2.1	NTM	$V_{\rm in} < V_{\rm hub} < V_{\rm out}$	Control system fault or loss of electrical network	U	N
Taun	2.2	NTM	$V_{\rm in} < V_{\rm hub} < V_{\rm out}$	Protection system or preceding internal electrical fault	U	A
	2.3	EOG	$V_{hub} = V_r \pm 2 \text{ m/s and}$ $V_{out}$	External or internal electrical fault including loss of electrical network	U	A
	2.4	NTM	V <sub>in</sub> < V <sub>hub</sub> < V <sub>out</sub>	Control, protection, or electrical system faults including loss of electrical network	F	*
3) Start up	3.1	NWP	$V_{\rm in} < V_{\rm hub} < V_{\rm out}$		F	*
	3.2	EOG	$V_{\rm hub}$ = $V_{\rm in}, V_{\rm r} \pm 2$ m/s and $V_{\rm out}$		U	N
	3.3	EDC	$V_{hub} = V_{in}, V_r \pm 2 \text{ m/s}$ and $V_{out}$		U	N
4) Normal shut down	4.1	NWP	$V_{\rm in} < V_{\rm hub} < V_{\rm out}$		F	*
	4.2	EOG	$V_{hub} = V_r \pm 2 \text{ m/s and}$ $V_{out}$		U	N
5) Emergency shut down	5.1	NTM	$V_{hub} = V_r \pm 2 \text{ m/s and}$ $V_{out}$		U	N
<ol> <li>6) Parked (standing still or idling)</li> </ol>	6.1	EWM	50-year recurrence period		U	N
	6.2	EWM	50-year recurrence period	Loss of electrical network connection	U	A
	6.3	EWM	1-year recurrence period	Extreme yaw misalignment	U	N
	6.4	NTM	$V_{hub} < 0,7 V_{ref}$		F	*
7) Parked and fault conditions	7.1	EWM	1-year recurrence period		U	A
8) Transport, assembly, maintenance and repair	8.1	NTM	$\mathcal{V}_{maint}$ to be stated by the manufacturer		U	Т
	8.2	EWM	1-year recurrence period		U	A

 Table 3.2 Design load cases (International Electrotechnical Commission, 2005)

The abbreviations used in Table 3.2 are:

F is fatigue; U is ultimate strength; N is normal; A is abnormal and T is transport and erection.

### 3.2.3 RP3 - Assessment of the SS MWP MK. 2 FOUNDATION CONCEPT

In order to assess the maturity of the concept as a mobile MM and WTG foundation, a review of documentation related to the foundation concept was undertaken. This allowed for an understanding of the principal design considerations for the concept, and additionally identified areas where there was potential for these principles to be outside offshore wind design standards (e.g. the correct fatigue considerations) and therefore represent potential areas of increased risk. The design review and deployment of a MM structure, at Södra Midsjöbanken OWF development project in the Swedish Baltic Sea, highlighted that sufficiently detailed design studies had been undertaken for the concept to potentially be scaled to support a full-scale WTG. The concept was therefore benchmarked against other novel and existing foundation concepts through a conceptual design study for site specific conditions of one of E.ON's future development sites. This, along with the review of other assessments of the concept at alternative site conditions, highlighted the potential for lower CapEx than traditional foundation concepts. As a result, the commercial interest in understanding the certainty in these operational and installation structural responses and design assumptions, along with the availability of a deployed structure, warranted the deployment of SCM. The detailed design review of the MM documentation was therefore used in order to establish key structural areas for monitoring.

## 3.2.4 RP4 - Optimising parabolic solar trough foundations' design

A literature review was undertaken in order to determine the knowledge available on the design of CSP foundations. This highlighted the complex nature of the environmental wind loading applied to the foundation due to the complex shape of the parabolic troughs within an array of the solar field, exacerbated by their changing orientation as they tracked the sun. This highlighted the need for a detailed assessment of the loads applied to the foundations at Helioenergy, and so site-specific conditions were applied to drag and pressure coefficients

derived from small-scale wind tunnel testing undertaken by NREL (Hosoya et al., 2008), as described in design methodology Section of Appendix F. These loads were then applied to typical foundation types and specifications, which had been determined by a detailed design review of the foundations currently used commercially at the case study solar farm by the research engineer. Numerical analyses were then undertaken by the research engineer, highlighting some foundation types and locations where the foundation capacity had exceptionally high factors of safety, which thus indicate potential material and cost savings. The numerical analyses then proceeded with the optimisation of foundation sizes until a minimum factor of safety was reached.

Applying the small-scale wind tunnel test coefficients to collectors with different spacing and curvatures to those at Helioenergy potentially led to inaccuracies. Therefore given the degree of optimisation being dependent on the certainty in the load, more detailed analysis of the wind loading using CFD software was undertaken for Helioenergy's collector specifications.

## 3.3 METHODS/TOOLS USED

This Chapter provides a high level description of methods and tools used for each research aim.

# **3.3.1 RP1 - EFFECT OF GROUTED CONNECTION WEAR ON OFFSHORE WIND TURBINE FOUNDATIONS**

As described previously, an experimental type testing campaign has been undertaken to quantify wear rates for varying normal compressive stress, surface conditions, grout material properties and wetness. The experimental equipment and protocol designed and developed by the research engineer to achieve this is shown in Figure 3.3. The set up replicates the wear conditions experienced offshore by introducing relative vertical movement between the grout and steel surfaces (indicated by the green line) while in compression. The relative

displacement is achieved through the inner steel plate of the sample being fixed to the test frame, while the grout is bonded to the outer steel plates using shear keys and these are forced to move in the vertical plane by the test frame actuator. The compression is provided through tightening of the bolts on the compression plate. The single degree of freedom mounting brackets and beam allow the compression to be transferred though the sample while the outer plates and grout undergo displacement. The details of the test equipment and methodology employed can be found in Section 2.1 and 2.2 of the published journal paper (J2), included in Appendix D.



Figure 13 Experimental test arrangement

The modelling process that was undertaken by the research engineer is summarised in Figure 3.4. The architecture of the numerical model developed by the engineer to enable the variability of the loads experienced, both with time and location accounted for, is described in Figure 3.5. Further, the details of the development of the model can be found in Section 3 of the journal paper (J3), Appendix E.



Figure 14 The modelling process (reproduction of Mihram, 1972)

Based on correlations of the variables measured from the SCM and SCADA systems, it was found that the model needed to incorporate the SCADA measured variables of wind speed, direction and WTG power production as inputs. Before relationships could be derived, initial screening of both data sets was undertaken by the research engineer to determine a suitable period for the analysis in terms of data quality, and to minimise any drift effect due to settlement of the TP relative to the MP. As a result, a three-month time series from January to March 2012 of SCADA and SCM data was synchronized and analysed. The time series outputs from both systems were then correlated for conditions of constant wind direction aligned with the instrumentation orientation during power or non-power generation of the WTG. The wind speed was correlated against the measured strain and displacement responses of the SCM installed on one of the offshore WTG foundations in order to develop relationships between the inputs and the structural response. More detail on the data processing and SCM and SCADA systems can be found in Section 3.2 of J3. The structural responses were then transformed into an equivalent normal stress and displacement within the GC, based on simple bending theory, design stress concentration factors and normal compressive stress equations provided in DNV-OS-J101, as described in Section 3.3 of J3. The derivation of these relationships and transfer functions by the research engineer then allowed the appropriate experimental wear rate to be applied and wear computed for a given period of data. The details of this methodology can be found in (J3), Section 3.4, Appendix E.



Figure 15 Architecture of wear numerical model

## 3.3.2 RP2 - Examining loads in offshore wind turbine foundations

In order to understand the design strain responses, a review of the WTG OEM specification (Vestas, 2009) was undertaken by the research engineer to provide information on the thrust coefficients during power production between the cut-in and cut-out wind speeds. Design

equations based on (Burton et al., 2011) and the WTG specifications were then used to establish a thrust at hub height across a range of wind speeds. Beam bending equations, hollow section theory and the WTG specifications were then used to determine the strain response at the SCM instrumentation level. The details of this methodology can be found in section 3.2 of J3, Appendix E, and Section 4.2.1 of this thesis. The monitored structural responses were based on strain response data from January 2012 to March 2013, from gauges located on the inside surface of the TP wall, 1.5m above the top of the MP installed on foundation K1, as shown in Figure 3.6. Definitions of the instrumentation abbreviations used in Figure 3.6 can be found in Table 3.3. These gauges (SGA-V) are oriented in the vertical direction to measure axial strain of the structural steel of the TP wall and are located equidistantly around its circumference at 60° intervals.



Figure 16 SCM layout

W-SX-Y-Z						
Foundation H4						
VV location K1						
X Bracket location 1-6						
SGA - Axia	al strain					
Y Gauge type RD - Radial di	splacement					
VD - Vertical d	isplacement					
Z Orientation V - Ver	tical					

 Table 3.3 Instrumentation abbreviations

At the foundation location, information provided by the SCADA system was used to derive the wind loading experienced by the foundation, based on 10-minute average wind speed, wind direction and power production data. After suitable data analysis was undertaken to correct for SCM data variables, relationships between the SCADA and SCM data were then established through correlations of the key variables under constant wind direction as described in Section 4.2.2. A comparison of the SCM measured responses could then be made with the predicted response.

### 3.3.3 RP3 - Assessment of the SS MWP MK. 2 FOUNDATION CONCEPT

The methodology employed for the detailed design review has been the comparison of design standards with the design basis used for the MM foundation. This comparison undertaken by the research engineer allowed for any discrepancies in the design assumptions and methodology to be queried with the designer and therefore assist in assessing the suitability of the foundation for its current site conditions. The detailed design review produced as part of this research was issued in 2013 to the designer, Marcon Wind Power (MWP), for consideration, as some of the concerns raised were likely due to reporting discrepancies and incomplete information provided due to the sensitive nature of the intellectual property. A workshop was then organised by the research engineer and held with another offshore wind developer/operator (Statkraft), and the concept designer MWP to bring together findings from various works undertaken by the different parties. The design review and workshop allowed

for a better understanding of the methodology used for design of the MM and therefore the additional requirements for scaling the design to a WTG foundation.

In parallel with the engagement with the designer over the MM design review, a literature review and desk study of available information on the foundation concept for an offshore WTG foundation was undertaken by the research engineer and is presented in Section 3 of Appendix G. The design review, along with background research into oil and gas jack-up structural monitoring campaigns and previous E.ON offshore WTG foundation SCM specifications, facilitated the identification of key structural areas which experienced significant loading, complex loading or areas of potential concern. This enabled the condition monitoring specifications for the MWP MM foundation to be produced by the research engineer, identifying the required instrumentation and the locations to be monitored.

### 3.3.4 RP4 - Optimising parabolic solar trough foundations' design

An extensive and broad literature review was undertaken given the large amount of factors affecting the design, such as the structure, wind loading, soil conditions, and the immature nature of the technology. A review and comparison was undertaken for the current state of the various technologies for CSP, with a particular focus on the civil works required. Information was obtained from peer-reviewed journals, conferences, industry reports and plant operators. A literature review was then carried out on various structures used for parabolic troughs and the extent to which the operational behaviour of these structures have been assessed under various operational conditions, particularly wind loading at various pitches and yaw angles of the trough. This highlighted a significant lack of research in the field of wind modelling across an array of collectors for all the angles of operation experienced. However, low resolution data from small scale wind tunnel testing, performed by the NREL (Hosoya et al., 2008), was available and was used by the research engineer to

establish an estimate of the variance in loading across the array and the associated foundations for the collector specifications and wind conditions associated with Helioenergy. Reference foundations were designed by the research engineer based on a numerical model developed as part of the research project which utilised BS EN 1997-1 foundation design formulae and the Solar Collector Assembly (SCA) specifications (Kearney, 2007) to determine loads and therefore factors of safety.

CFD modelling was then undertaken by ETG's software modelling department for the sitespecific parabolic trough specifications and array spacing of Helioenergy. The software package ANSYS CFX was used for the numerical simulations to solve the steady-state Navier-Stokes equations for isothermal incompressible flows. A shear-stress transport turbulence model was used for the construction of the atmospheric boundary layer to improve the accuracy where flow separation occurred. The inlet boundary condition was a steady neutral atmospheric boundary layer consistent with the wind speed at a specified reference height and the estimated aerodynamic roughness of the ground. The outlet condition was specified as a constant static pressure boundary. Both left and right sides of the domain were specified as symmetry boundaries, essentially simulating an infinite number of troughs in the z-direction. At the top of the domain an entrainment boundary condition is applied. The geometrical domain and boundary specifications are shown in Figure 3.7. The domain mesh was mainly constructed from tetrahedral elements with refinement at the receiver and trough surfaces and at the ground using prism elements. The features of the meshing and geometries are shown in Figure 3.8.



Figure 17 Geometrical domain and boundary specifications for CFD model of CSP trough array



Figure 18 Irregular tetrahedral mesh used for CFD computations.

The load outputs of this modelling work were then compared to the wind loading derived from the wind tunnel testing load coefficients used in the foundation optimisation analysis by the research engineer. This was undertaken in order to validate and correct the foundation optimisation to account for the inaccuracies of using the small-scale wind tunnel test data.

# **4 THE RESEARCH UNDERTAKEN**

This chapter provides a review of research undertaken and results presented in relevant sections based on each research package.

# 4.1 RP1 - EFFECT OF GROUTED CONNECTION WEAR ON OFFSHORE WIND TURBINE FOUNDATIONS

## 4.1.1 EXPERIMENTAL TESTING

Engagement with the grout manufacture, Densit, in 2011, through an initial meeting and review of design and material classification documentation, indicated that limited testing of the grouts fundamental behaviour under conditions relevant to offshore WTG foundations had An extensive literature review of experimental testing of grouted been undertaken. connections, details of which can be found in the published journal paper (J1), Appendix C, showed that historic type-testing undertaken indicated that the conditions required for wear to occur could exist within grouted connections used for offshore WTG. It also highlighted that wear had been experienced under significantly higher dynamic displacements than would be experienced in operational WTG GCs. Another issue revealed was that the design principles in the existing standards up to 2011 were based on small-scale experimental testing from the oil and gas industry (Billington and Lewis, 1978; Karsan and Krahl, 1984; Sele and Kjeøy, 1989 and Lamport et al., 1991). The limits of this experimentation in terms of compressive strength of the grout and ratio of diameter to thickness were well below those currently used in offshore wind grouted connections. High-strength grout had also only been tested for compressive strength and single axis fatigue by manufacturers and limited testing had been undertaken for some of the conditions relevant to offshore wind turbine foundations (Andersen and Petersen, 2004, Schaumann and Wilke, 2007, Anders and Lohaus, 2008 and Schaumann et al., 2010). Installation conditions such as the pumping of the grout through the water filled annulus and curing of the grout during wave action were also shown to have significant effects on the strength of the grout and therefore GC (Lamport et al., 1991 and DNV 2010a).

Overall, the behaviour of the grout-steel interface over long-term operational loads was not fully understood within the industry and scientific community. Testing recently carried out (DNV, 2010; Lotsberg, 2013 and Lotsberg et al., 2013) did not address some areas of concern, such as grout wear and the effects of water ingress. It also indicated a lack of knowledge and wear data applicable to offshore WTG GC conditions. In addition, the standard model wear tests such as pin-on-disc (ASTM G99 and BS EN 1071-13:2010), abrasive wheel (BS ISO 9352:2012, BS EN 660-2:1999 and BS EN 13892-4:2002), abrasive slurry (BS EN 1071-6:2007 and BS EN 12808-2:2008), non-sliding reciprocation (BS EN 13892-7:2003 and BS EN 660-1:1999) and non-sliding rotational (BS EN 13863-4:2012), would not accurately replicate the wear conditions due to the wear mechanisms they employ. However, a sliding reciprocating wear test similar to BS ISO 14242-3:2014 or BS EN 1071-12:2010 would more accurately replicate the conditions found in offshore WTGS GCs with its relative movement between the two parts of the sample, but would not accommodate the scale and loads required to replicate the operational conditions of the GC. This lead to the development of the experimentation outlined in Section 3.3, which was used to determine the influence of the independent variables displacement, material properties, compressive stress, surface finish and presence of sea water on the dependent variable of wear rate. The development of the experimentation by the research engineer included the design, procurement and assembly of the test equipment, casting, instrumentation and measurement of the samples and running, debugging and development of the testing and logging equipment throughout the testing campaign. In tribology, wear rate is typically defined as volume lost per unit normal load per distance of relative displacement (Archard, 1953). However, within this research wear rate is quantified as the average change in the measured thickness of the

sample per 100m of cumulative relative displacement ("walked distance") of the interaction surfaces. This definition has been used in order to present the results of the experimental campaign directly into the context of the real-world applications in offshore grouted connections. Cumulative relative displacement is defined as the sum of the relative axial displacements at the grout-steel interface of the sample.

The compressive stress was varied by changing the torque which the compression bolts were tightened to, with the applied compressive stress derived from the surface area and compression bolt strain gauge response, which had been calibrated with a load cell. The compression bolts were tightened after each phase of testing and any variation in compression experienced during the test phase was accounted for by post processing of the compression bolt strain data. The surface finish was varied through testing of different samples with different surface conditions, which consisted of corroded (rust grade C to ISO 8501-1:2007), shot-blasted (Sa 2<sup>1</sup>/<sub>2</sub> to ISO 8501-1:2007) and mill-scale finishes. The presence of seawater was simulated through drip feeding of a sea water equivalent solution over the grout-steel interaction zone at 0.064 l/s. This flow rate was chosen to ensure the sample remained sufficiently wet while within the limit of the filtration system. The influence of the grout material properties was based on the variation of 28 day tensile splitting strength (BS EN 12390-6:2009), elastic modulus (BS 1881-121:1983) and compressive strength (BS EN 12390-3:2009) of five 100mm cubes and one 150 x 300mm cylinder cast from the same batch of grout as each sample tested. Confined and non-confined samples to represent different locations within the grouted connection were replicated through the inclusion of steel brackets at the top and bottom of the outer steel plate of the samples, as shown in Figure 7 in Section 4.3 of Appendix D. The influence of these variables was determined through adopting the test matrix shown in Table 4.1.

#### Table 4.1 Test matrix

SAMPLE	CHARACTERISTICS	REASONING
<b>S1</b>	Mill scale, Dry, Unconfined	Test of logging equipment & Rig
S2	Mill scale, Dry, Unconfined	Test influence of controller amplitude and
		frequency
<b>S3</b>	Sa 2 <sup>1</sup> / <sub>2</sub> , Dry, Non corroded,	Influence of surface finish and higher loads
	Confined	
<b>S4</b>	Sa 2 <sup>1</sup> / <sub>2</sub> , Wet, Corroded, Confined	Influence of water presence and corrosion
<b>S</b> 5	Sa 2 <sup>1</sup> / <sub>2</sub> , Wet, Non corroded,	Influence of corrosion
	Confined	
<b>S6</b>	Sa 2 <sup>1</sup> / <sub>2</sub> , Dry, Corroded, Confined	Influence water presence
<b>S7</b>	Repeat S4	Improve significance of results/determine the
<b>S8</b>	Repeat S4	influence of grout material properties

To quantify the dependent variable of the loss in thickness of the samples, four alternative methods were used, the methodology for which can be found in (J2), Appendix D. These methods included:

- Vernier caliper measurements of the sample component thicknesses pre- and post-test;
- Continuous LVDT measurement of each test phase;
- Weight of evacuated material at the end of each test phase;
- Geometric white light scan of the sample components pre- and post-test.

The detailed results and analysis from the experimentation can be found in Sections 3 and 4 of J2, Appendix D, which are also summarised into the main quantitative and qualitative findings below. Qualitatively it was found when de-moulding the samples 48 hours after casting that the adhesive strength between the steel and the grout was greatly reduced with corrosion, and lower for mill-scale surface finishes when compared to shot-blasted finishes.

Visual inspection of the samples' interaction surfaces at the end of testing indicated mirrorlike finishes on both the grout and steel under wet test conditions and compacted powder on both steel and grout surfaces with scoring of the steel during the dry tests, Figure 4.1.



Figure 19 Example of post-test samples, grout (left), steel (right) surfaces, wet testing (top) and dry testing (bottom)

During testing it was evident from the outset, by the evacuation of wear debris, that wear was occurring even at the lowest load increments, Figure 4.2.





Figure 20 Visual evidence of wear debris evacuation at the top (left) and directly below (right) of the wear interfaces

Upon ultimate failure of the samples, fracture plane orientations were aligned with historic axial capacity testing of plain-pipe GCs (Sele and Skjolde, 1993; Krahl and Karasan, 1985;
Billington and Lewis, 1978; Smith and Tebbett, 1989; Lamport et al., 1991 and Aritenang et al., 1990), Figure 4.3.



Figure 21 Failure of unconfined samples, S1 (left) and S2 (right) at higher load increments Quantitatively, the results from the different loss of thickness techniques are summarised in Table 4.2 and Figure 4.4 in terms of wear rates. Table 4.3 summarises the total loss in thickness.



Figure 22 Wear rates based on weight of evacuated material (top) and LVDT (bottom) methods

Table 4.2, 4.3 and Figure 4.4 indicate that there is a significant difference between the wear rates measured for the wet (S4, S5, S7 and S8) and dry (S3 and S6) samples. However, the difference in wear rate between the corroded and non-corroded samples for either wet or dry conditions is marginal.

The differences between the magnitudes of wear indicated by the different measurement techniques are explained in detail within Section 4.4.4 of Appendix D. The LVDT results were generally higher due to the variable lateral motion and cyclic distortion of the samples possibly contributing to the increased measured horizontal displacements. In addition, there is also the influence of the signal quality of the monitoring data, which can be affected by temperature variations, electrical interference and other sources of noise. This would have also provided sources of measurement error which could have accounted for the difference in measurements.

SAMPLE	TEST	COMPRESSIVE	LOSS IN THICKNESS PER 100m WALKED DISTANCE (mm)			
ID		STRESS (MDa) -				
	CONDITION	STRESS(IVII a)	LVDT	<b>BY WEIGHT OF EVACUATED MATERIAL</b>		
		0.7	0.26	0.019		
	D.m. C. 21/	1.1	0.21	0.027		
<b>S3</b>	Dry, Sa 272,	1.5	0.24	0.035		
	Un-corroded	1.9	0.35	0.047		
		2.2	0.26	0.050		
	Wet, Sa 2½, Corroded	0.6	0.45	0.13		
		0.7	0.50	0.22		
		1.1	0.55	0.42		
54		1.5	0.45	0.44		
		1.9	0.72	0.55		
		2.4	0.60	0.41		
	Wet, Sa 2½,	0.6	0.33	0.16		
		0.7	0.47	0.25		
S5		1.1	0.38	0.073		
	Un-corroded	1.5	0.43	0.26		
		1.9	0.32	0.35		
		2.4	0.64	0.69		

**Table 4.2 Comparison of wear rates** 

		0.6	0.23	0.004
C(		0.8	0.12	0.017
	Dry, Sa 21/2,	1.2	0.20	0.014
50	Corroded	1.5	0.38	0.020
		2.0	0.16	0.034
		2.4	0.24	0.015
	Wat So 21/	0.5	0.38	0.15
<b>S7</b>	Corroded	1.5	0.42	0.32
		2.3	0.53	0.34
S8	Wat So 21/	0.6	0.53	0.21
	Wel, Sa 272,	1.4	0.51	0.34
	Corroded	2.1	0.34	0.41

Table 4.3 Comparison of total loss in thickness

SAMDLE	Трот	TOTAL	TOTAL LOSS IN THICKNESS (mm)			
ID	CONDITION	WALKED DISTANCE (m)	LVDT	BY WEIGHT OF Evacuated Material	VERNIER CALIPER	
<b>S</b> 3	Dry, Sa 2 <sup>1</sup> / <sub>2</sub> , Un-corroded	1236	2.71	0.20	0.23	
<b>S4</b>	Wet, Sa 2½, Corroded	402	1.85	1.31	1.21	
S5	Wet, Sa 2½, Non-corroded	490	2.13	1.64	1.09	
<b>S6</b>	Dry, Sa 2 <sup>1</sup> / <sub>2</sub> , Corroded	625	1.38	0.11	-0.02	
<b>S7</b>	Wet, Sa 2½, Corroded	248	1.33	0.77	1.21	
<b>S</b> 8	Wet, Sa 2 <sup>1</sup> / <sub>2</sub> , Corroded	234	1.27	0.80	1.02	

The variation of coefficient of friction recorded is shown in Figure 4.5, indicating a distinct

difference between the wet and dry samples.



Figure 23 Variation of coefficient of friction

The geometric white light scanning was undertaken pre- and post-test (without cleaning of the samples) for each of the sample parts, with a comparison of the difference, relative to an unchanged reference location, providing the information shown in Figure 4.6 from the GOM Inspect software. Unfortunately due to technical issues with data management by a third party, sample S6 was the only sample where useful information was extracted for just the outer grout surfaces of the sample. The results of this align well with the visual findings, which showed areas of debris build-up causing slight rotational movement around this point of the sample rather than plain/uniform vertical movement. The S6R scan also shows the fracture of the unconfined side of the grout as a result of this lateral movement with a clear crack visible, indicated by the dark blue line. The distribution of loss of thickness of the whole area scanned is shown by the plot to the side of the sample legend. Although this shows an overall loss of thickness, the magnitude and location of the peak of the distribution is skewed, due to the interaction surface data being diluted by the data from the rest of the samples surfaces.



**Figure 24 Comparison of pre- and post-testing geometric white light scans of S6L (left) S6R (Right)** In addition to the results presented in (J2), the influence of the variation of the grout properties on the rate of wear was investigated in order to determine if particular foundations

are likely to be more affected by wear due to the quality of the grout. Through the repetition of the wet corroded conditions, samples S7 and S8, and comparison with similar test condition samples (S4), the recorded 28 day properties of the grout have been compared with the measured wear, Table 4.4.

	Choi	COMPRESSIVE STRESS				
	GROU	0.6	1.5	2.4		
SAMPLE ID	28 DAY COMPRESSIVE STRENGTH (f <sub>c</sub> ) (MPa) TENSILE STRENGTH (MPa)		ELASTICWEAR RATE (mm/10)MODULUSWEIGHT OF EVACUA(GPa)MATERIAL [LVD]		/100m) CUATED DT]	
<b>S</b> 4	110.1	7.7	54.50	0.13[0.45]	0.44[0.45]	0.41[0.60]
<b>S</b> 7	142.1	8.3	51.77	0.15[0.38]	0.32[0.42]	0.34[0.53]
<b>S</b> 8	138.5	7.7	47.67	0.21[0.53]	0.34[0.51]	0.41[0.34]
		Mea	an Wear Rate	0.16[0.45]	0.37[0.46]	0.39[0.49]
		Standard Deviation		0.04[0.08]	0.06[0.15]	0.04[0.13]
		Coefficier	nt of variation	0.25[0.17]	0.18[0.09]	0.10[0.27]

Table 4.4 Comparison of grout properties and wear rates for wet corroded samples

The analysis of Table 4.4 does not reveal any strong correlation between the grout properties and the indicated wear rates measured by either the LVDT or weight of evacuated material methods. Therefore as the 22% variation of the test samples' 28 day compressive strength (Table 4.4) was similar to the variation across Robin Rigg (Densit, 2009), it can be assumed that there will be no noticeable difference in wear rates across the wind farm based on the grout properties. The variability of the results under similar conditions indicates that the variation between wear rates for similar samples is approximately in line with the variation of material properties. In terms of determining a relationship between the grout properties and wear, literature on wear of comparatively lower characteristic compressive strength concrete suggests that tensile and compressive strengths are key factors (Horszczaruk, 2008 and Yazici and Inan, 2006). However, increased testing of samples with a greater variation in properties would be required to produce any results with significance.

From the different measurement techniques undertaken as part of the investigations, it is clear that the most significant factor on the wear rate is the presence of water, which at best doubles the wear rate (LVDT), but at worst could be up to 18 times higher (Vernier caliper) than for dry conditions. In comparison, the surface conditions of the steel appear to have only a marginal influence, although they are likely to affect initial adhesive strength of the grout/steel joint.

A possible explanation of such a significant impact of the water presence on the wear rate is the ability of the wear debris to be evacuated from the interaction surfaces, which is therefore deemed to be critical to the loss in thickness. The wear rate is therefore likely to be influenced by the rate at which water is being flushed or washed through the gaps and cracks between the grout and the MP surface. However, this is not reflected in the experimental testing where a constant low rate of sea water equivalent solution application of 0.0641/s was used to ensure continued water flow over the sample. For dry connections, i.e. when the transition piece is not submerged, evacuation and therefore wear will be limited by the rate at which the debris is evacuated from the very top or bottom of the connection. However, for wet connections there is likely to be quicker and complete transportation of the wear debris over the whole length of the connection, and so more significant loss in thickness will occur over the entire length. Depending on the tidal range of the wind farm, the connection may only ever be partially submerged and so the significant loss will be restricted to this portion. The relatively small size of the sample (150x150mm) may also provide a faster evacuation rate of wear debris for both dry and wet conditions, due to the smaller transportation distance when compared to a full size foundation. The experimentation also showed that the presence of water also effected wear through the change of the wear debris from a dry powder to a wet paste, which from Figure 4.1 can be seen to have resulted in the polishing of the interaction surfaces, similar to the effects of a grinding paste.

Finally, due to the high alkalinity of the grout of a pH of approximately 13 (Densit, 2011) and its mineralogical and chemical nature (20-85% Portland cement (PC)), it is in natural disequilibrium with its environment. It is therefore easily attacked by pure water and acid solutions (Chandra 1998; Revertegat et al., 1992; Pavlik, 1994; Faucon et al., 1996 and Glasser et al., 2008) and degradation of the material mechanical properties of grout may well be experienced due to Magnesium Sulphate and Sodium Chloride attack and leaching from the approximately neutral pH of the salt water equivalent solution (Allahaverdi and Skvara, 2000; Reardon, 1990; Berton et al., 2007 and Faucon et al., 1996). Usually this is not an issue for marine structures as a protective layer of Brucite (Magnesium Hydroxide) forms (Skalny et al., 1999), but due to the abrasive nature of the GC, this weak protective layer will continually be removed and may lead to acceleration of the degradation process (Monteny et al., 2001). Figure 4.7 suggests significant reductions in mechanical properties of the grout could be experienced within a year for any exposed surfaces. However, the rate of degradation is highly dependent on permeability and so would need to be investigated to understand how the loss of strength changes with thickness of the grout over time. The duration of each sample tested as part of RP1 is limited in comparison to design life of the structure and therefore if the grout is affected by degradation due to the exposure to the marine conditions, wear rates may be higher than indicated by the testing. Further research investigating the degradation of the grout mechanical properties would be required to substantiate this.



Figure 25 Compressive strength for PC mortars in different storage solutions (reproduction of Santhanam, 2001)

## 4.1.2 STRUCTURAL CONDITION MONITORING

To establish relationships between SCM and SCADA data, existing data was analysed rather than incurring the expense of installing SCM on every location; the case study of Robin Rigg was used, and specifically foundation locations H4 and K1 were studied. Their locations within the windfarm are shown in Figure 4.11. The January to March 2012 period of data was used to determine relationships, as minimal settlements between the TP and MP were experienced during this period. Initial analysis of the strain data revealed that there was a significant offset in strain values between the various gauges for the same magnitude of wind speed when the wind direction was aligned with the gauge location, Figure 4.8. This offset was found to be the result of the datum setting of the SCM being impractical at 0m/s, given the scarcity of the duration of these events, and so occurred at significant wind speeds. Details of the datum setting for H4 and K1 can be found in Table 4.5.

Table 4.5 H4 and K1 conditions at datum setting

WTG FOUNDATION	DATE	TIME	WIND SPEED (m/s)	WIND DIRECTION (°)
H4	25/10/11	2150	3.5	109
K1	01/09/11	0600	2.6	174



**Figure 26 TP vertical strain responses (SGA-V) of K1 at low wind speeds to derive correction factors** SCM data in the form of the horizontal displacement gauges was analysed, as described in Section 3.5 of journal paper (J3), Appendix E, to determine any measured change in horizontal displacement for constant loading conditions indicated by the SCADA data. This provided values for the numerical model calibration (case K1) and validation (case H4) of the predicted wear. An example of the result is shown in Figure 4.9 for one of the five horizontal displacement gauge locations analysed.



Figure 27 Example of measured loss in thickness between MP and TP based on change in horizontal displacement response over time for constant strain and wind direction for K1-S2-HD

The SCADA and SCM data outputs were used to determine relationships between key environmental inputs (wind speed and direction) and structural responses (strain in the TP wall and relative vertical displacement between the MP and TP). This was achieved through correlation of the strain indicated by the SGA-V gauges and the nacelle measured wind speed and correlation of the wind speed and the relative vertical displacement (VD) between the TP and MP at the top of the GC. These correlations were both made while the wind direction aligned with the SCM gauge orientations within the TP. The details and results can be found in Section 3 of J3, Appendix E, and examples of these relationships are shown in Section 4.2.

# 4.1.3 NUMERICAL MODEL

To establish how significant these wear rates derived from experimental testing were over the design life of the plant, a numerical model was developed to apply them to the loading regime experienced by typical offshore wind turbine structures. The model provided an indication of the distribution of wear around the circumference and depth of the grouted connection (GC), which will help to determine if further remediation of the existing grouted connection is going to be required within the remaining operational life of the wind turbine foundation.

As the relationships were derived from a limited period of data, the limits of validity of the model are defined by the events experienced in this period. The model is therefore valid for power and non-power production of the WTG up to wind speeds of 32m/s for the specifications of the Robin Rigg structure and WTG. The influence of wind speeds greater than 32m/s on the magnitude of wear are likely to be limited given the low probability of occurrence over the design life of the foundation, as shown in Figure 13, Appendix C, which demonstrates negligible occurrences of wind speeds greater than 30m/s have been measured over the four years of wind speed data available. Assumptions include:

- Structural response and load transfer mechanisms of the GC remains constant over the design life of the foundation;
- Short and long term variations in the metrological conditions are accounted for by the three year period of inputted SCADA data;

• Availability of the WTG during the period of inputted SCADA data is representative over its lifetime.

The relationships between the SCADA and SCM data, a transfer function derived from bending theory and DNV-OS-J101 equations for normal compressive stress within a GC, the turbine tower and foundation specifications and the experimental wear rate were then used to derive the wear for an inputted period of SCADA data based on the architecture shown in Figure 3.5. The details of the methodology can be found in Section 3 of J3, an example of the model output is shown in Figure 4.10 and an example of the wear calculations within the model can be found in Appendix A.



#### Figure 28 Distribution of wear around the circumference and depth of the GC

The initial output was then calibrated against the SCM measured wear at K1, as described in Section 4.1.2 and Section 3.5 of J3, for the three months period from January to March 2012. This calibration was required as the input SCADA and SCM data were provided as 10-minute averages and so did not provide information on the higher frequency load inputs and response

of the structure. Analysis by the research engineer of a limited period of high frequency SCM data on vertical displacement between the MP and TP at RR, indicated a significant increases in the accumulated measured displacement with frequency of data. The calibration was found to align well with the first mode of the substructure's natural frequency response at 0.29-0.33Hz, and the blade passing (1P and 3P) driving frequencies from the WTG at 0.14-0.31Hz and 0.43-0.92Hz. To improve the confidence within the model, longer periods of high frequency data should be analysed to improve the accuracy of the influence of the higher frequency responses of the structure on wear. This along with other recommendations for further research can be found in Section 5.5.1.

Input SCADA data and SCM measured wear for periods of May to July 2012, July to September 2012 and January to March 2013 for H4 and K1 were then used to validate the calibrated model as described in Section 3.5, Appendix E. With the model validated, three years of SCADA data for 11 locations evenly distributed around the wind farm were then inputted into the model to determine an accumulated wear over this period of time and provide an indication of the spatial variation of wear. The three year period of input data was chosen to ensure sufficient data was included to account for the temporal variation of wind speed and directions that occur (Carta, Bueno and Ramirez, 2008; Fruh, 2013; Carta, Ramirez and Bueno, 2008; Akpinar and Akpinar, 2005; Coelingh, Wijk and Holtslag., 1996; Palutikof and Barthelmie, 1996; Kou, Liang, Gau and Gau, 2014; Carta, Veazquez, Eazquez and Cabrera, 2013), considerably longer than the six months minimum recommendation of IEC61400-1. To account for the availability of data during this period, the output was scaled by the proportion of available data to non-available data. This value of wear was then scaled up to an equivalent for the 20-year design life of the foundation. As the model is based on a period of data early in the design life of the structure it does not take account of any change in structural response over the lifetime of the structure, a comparison with the response once

degradation and wear of the GC have occurred would enable the applicability of the model over the lifetime of the structure to be determined.

The effects of the relatively high wear at the very top and bottom of the connection indicated in Figure 4.10, are likely to lead to a redistribution of the pressure and therefore change in wear distribution over the operation life of the GC. The extent of which would be dependent on the stiffness of the connection and so investigation of this would be a recommendation for further research to improve the refinement of the model.

The distribution of wear at the top of the GC, shown in Figure 4.11, indicates limited variation across the wind farm, with the outlier of C3. It can be seen that the maximum wear at the very top of the GC is to be found on the predominant wind direction side and is in the order of 3.5 to 4.8mm, with around 1mm on the opposite side indicating a possible gap of 5 to 6mm after 20 years of operation.

Upon investigation of the input data for C3, it was found that C3 had slightly lower durations of wind speeds, but not enough to have a significant impact on the wear results. However, it did show that the power data provided for C3 by the WTG OEM was in the wrong format compared to the other locations.



Figure 29 Spatial variation of 20 years of accumulated wear for 11 foundations at Robin Rigg offshore wind farm Analysis of the SCM data also allowed assessment of the fracture of the grout that was evident from the experimental testing at compressive stress levels greater than 2.0MPa. This was not related to the ultimate shear failure of the grout, but accumulated fatigue damage over the testing period. As there was only a limited number of samples tested until ultimate failure of the grout, further testing would be required in order to determine fatigue failure curves for the stress conditions experienced in the test arrangement.

Based on a review of the compressive stresses experienced for 2012 derived from the numerical model developed, the probability distribution of compressive stress at the top of the GC was produced (Figure 4.12), for the strain gauge orientations (S1-6) indicated in the SCM layout, Figure 3.6.





It can be seen that compressive stresses greater than 2MPa are experienced at the very top of the GC at all locations around its circumference, reaching a maximum of 2.8MPa for the prominent wind directions. This maximum is a result of the peak thrust produced by the WTG at 14m/s wind speed (Figure 4.15), measured during this monitoring period, and the geometry of the structure and GC. Based on the measured wind speed distribution (Figure 13, Section 4, Appendix E), it is apparent that the 14m/s wind speed probability of occurrence is relatively low compared to the mode of 4m/s and as shown in Figure 4.15 the load on the structure reduces as the wind speed increases beyond 14m/s, resulting in the truncation at 2.8MPa. It is evident that the strain gauge orientation, S3, as aligned with the prominent wind direction, experiences the highest duration of these higher compressive stresses and so is likely to experience greater fatigue damage accumulation which may result in fracture of the grout. Given the indicated stress shown in Figure 4.12 includes the assumed SCF, the magnitude of the compressive stress will reduce very quickly from the end discontinuity of the GC, more details can be found in Section 3 of J3, Appendix E. Based on this reduction, the maximum magnitude of compressive stress experienced by the grout is below 2MPa by 250mm from the top or bottom of the GC. This therefore indicates that if fracture of the grout was to occur due to fatigue it would initially only be in the very top or bottom of the GC.

# 4.2 RP2 - EXAMINING LOADS IN OFFSHORE WIND TURBINE FOUNDATIONS

# 4.2.1 PREDICTED STRAIN RESPONSE

The review of the OEM WTG specifications (Vestas, 2009), provided the thrust coefficient

for the trust experienced by the WTG during operational wind speeds, Table 4.6.

WIND SPEED (m/s)	THRUST COEFFICIENT, C <sub>t</sub>	WIND SPEED (m/s)	THRUST COEFFICIENT, C <sub>t</sub>	WIND SPEED (m/s)	THRUST COEFFICIENT, Ct
4	0.874	12	0.480	20	0.122
5	0.836	13	0.420	21	0.105
6	0.805	14	0.370	22	0.092
7	0.805	15	0.296	23	0.081
8	0.808	16	0.240	24	0.072
9	0.739	17	0.199	25	0.064
10	0.650	18	0.167		
11	0.554	19	0.142		

Table 4.6 OEM WTG thrust coefficients

These coefficients were then used to derive the thrust for each 1m/s wind speed increment using Equation (4.1) (Burton et al., 2011) and the specifications for the Robin Rigg WTG being used as a case study.

$$T = 0.5C_t \rho_a A_{Swept} V_{\infty}^2 \tag{4.1}$$

where *T* is the thrust at hub height;  $C_t$  is the coefficient of thrust;  $\rho_a$  is the density of air (kg/m<sup>3</sup>);  $A_{swept}$  is the swept area of the rotor blades and  $V_{\infty}$  is the undisturbed air velocity.

Based on the Robin Rigg WTG structure specifications, this thrust is applied 77.9m above the SGA-V location of the SCM. Therefore the moment at this point is given by Equation (4.2)

$$M_{tp1.5} = 77.9T \tag{4.2}$$

where  $M_{tp1.5}$  is the moment at 1.5m above the top of the MP, i.e. the SGA-V gauge level.

This moment is converted to a vertical strain value using the transfer function (4.8), derived below. From simple beam bending theory, the following formula holds:

$$\sigma = \frac{zM}{l},\tag{4.3}$$

where  $\sigma$  is the normal stress; *z* is the distance to the neutral axis; *M* is the applied bending moment and *I* is the second moment of area, which for a hollow section is:

$$I = \frac{\pi}{4} (R_{tpo}^4 - R_{tpi}^4), \tag{4.4}$$

where  $R_{tpo}$  and  $R_{tpi}$  are the outer and inner radius of the TP.

Substituting (4.4) and (4.2) into (4.3), one obtains:

$$\sigma_{tpi1.5} = \frac{R_{tpi1.5}77.9T}{\frac{\pi}{4}(R_{tpo1.5}^4 - R_{tpi1.5}^4)}$$
(4.5)

Considering now the relationship:

$$\epsilon = \frac{\sigma}{E_s},\tag{4.6}$$

where  $\varepsilon$  is the strain and  $E_s$  is the Young's modulus of the steel, Equation (4.5) gives:

$$\epsilon_{tpi1.5} = \frac{R_{tpi1.5}77.9T}{\frac{\pi}{4}(R_{tpo1.5}^4 - R_{tpi1.5}^4)E_s}.$$
(4.7)

Substituting (4.1) into (4.7) the strain at the inner face of the TP, 1.5m above the top of the MP take the expression:

$$\epsilon_{tpi1.5} = \frac{155.8R_{tpi1.5}C_t\rho_a A_{Swept} V_{\infty}^2}{\pi (R_{tpo1.5}^4 - R_{tpi1.5}^4) E_s},$$
(4.8)

where

$$A_{swept} = \pi R_{rotor}^2 = 6361.7 \text{m}^2, R_{tpi} = 2.22 \text{m}, R_{tpo} = 2.27 \text{m}, \rho_a = 1.225 \text{kg/m}^3, E_s = 210 \times 10^9 \text{N/m}^2$$

# 4.2.2 MEASURED STRAIN

Once initial offsets in the SCM data were corrected, as described in Section 4.1.2, the 10minute average data from both the SCM and SCADA systems was processed to ensure the respective time series of data were aligned correctly, due to the two data acquisition systems not sharing a common clock. The SCADA data could then be compared with the SCM measured vertical strain responses. This was achieved by correlating the strain response with wind speed for events when the wind direction was within 1° of the orientation of each vertical strain gauge (SGA-V) and the turbine was generating power. This resulted in the relationship between wind speed and vertical strain provided by the trend lines shown in Figure 4.13, with two trend lines incorporated to offer the conservative best fit to the data.



Figure 31 Measured vertical strain (SGA-V) response for locations S2 and S3 with wind speed for an aligned wind direction during power generation

The scatter shown in the strain response can be explained by: i) the 10-minute averaging; ii) the influence of wave loading; iii) nacelle wind direction misalignment. Based on the analysis of the SCADA data, nacelle misalignment was shown to be +/- 5°. Information on wave height and direction was not incorporated as: i) it is not a major source of loading during WTG operation at this water depth and site location; ii) no wave data was available for the exact location. However, it could be incorporated in the future through the use of limited data available from a MM nearby using similar analysis techniques if further refinement is required. Although statistically wave loading can be related to wind speed as wind drives the waves, depending on site bathometry and temporal variation, the wave direction and magnitudes can vary and be different to the wind, with some sites having significant durations of 180° misalignment between wind and wave directions.

A similar analysis of the strain response for non-power generation events was undertaken and resulted in the strain response shown in Figure 4.14.



Figure 32 Measured vertical strain (SGA-V) response for locations S2 and S3 with wind speed for an aligned wind direction during non-power generation

From Figure 4.14, a reasonable scatter can be seen in the data and below the wind turbine cutin speed of 4m/s, unexpectedly higher strains were measured when compared to the power generation strains and estimates of strain based on aerodynamic drag on the structure. One explanation of this could be the lack of aerodynamic damping (4% in fore-aft direction, Ramboll, 2007b) from the turbine, resulting in the wave and current loading having a more significant impact on the structure during non-power production. However, further work is required to substantiate this.

## 4.2.3 COMPARISON

A comparison of the 10-minute average measured strain with the predicted strain based on the WTG's thrust coefficients for power generation is shown in Figure 4.15. This shows that the predicted response lies within the scatter of the measured strain response, indicating that from a static perspective, the loads provided by the WTG OEM to foundation designers appear to be reasonable as the coefficients themselves will have been derived from data with reasonable scatter in the first place. The graph also demonstrates the likely factor of safety applied to the

design, with curves representing the partial load (PLF) and material (PMF) factors applied as recommended by DNV-OS-J101 to account for load and material variability.



Figure 33 Measured strain compared to thrust derived strain for power generation

A correlation of the SCM measured vs. the model predicted strain for three of the vertical strain gauges installed on the two foundations at Robin Rigg can be seen in Figure 4.16. This shows a very strong positive correlation between the SCM measured strain and model predicted strain.





This demonstrates that the SCADA data has the potential to be used to provide useful information, and if calibrated and validated against a larger population of turbines could be used to estimate loads experienced by offshore wind turbine foundations across a wind farm without the use of expensive SCM systems on every location.

In order to assess the dynamic response of the structure, work needs to be undertaken to investigate the strain response at higher frequencies of the strain and wind speed data.

It should be noted that it was assumed that the upstream undisturbed wind velocity  $V_{\infty}$  is equal to the measured wind speed as this provides the best fit with the measured data. However, as the wind speed is measured using an anemometer located on top of the nacelle, it will be downstream of the WTG's rotor. Given the rotor blade is extracting energy from the airstream, the average downstream velocity is likely to be lower than the upstream velocity. IEC1400-12 recommends for performance testing of WTGs that the MM measurements are only to be used if outside a 90° sector downwind of the WTG, suggesting the thrust coefficient derived strain in this report will be unrepresentatively low. However. approximations for this loss in wind speed based on blade element momentum theory (Burton et al., 2011) demonstrated that the resultant strain trends constructed based on this correction did not align with the measured strains or WTG thrust curve, suggesting that the OEM coefficients are corrected for the difference in free stream and measured wind velocity. Confirmation was sought from the OEM on the methodology used to derive the thrust coefficients, but unfortunately a response was never provided, probably due to intellectual property concerns and commercial sensitivities.

It is also assumed that that the TP's as-built dimensions and material properties are exactly the same as those specified. Variation of thicknesses, diameters and Young's modulus will all affect the calculated strain. It would therefore be useful to determine the influence of these factors on the results based on acceptable tolerances for manufacture of the TP and typical material properties variations to demonstrate the confidence in the results and that they align with the PMF applied in design.

# 4.3 RP3 - ASSESSMENT OF THE SS MWP MK. 2 FOUNDATION CONCEPT

# 4.3.1 MM DETAILED DESIGN REVIEW

As part of assessing the suitability of the MWP Mk. 2 concept as an offshore MM and WTG foundation, the design documentation provided by SS on the MM installed at E.ON's Södra Midsjöbanken development site in the Swedish Baltic Sea was reviewed. The design of the foundation utilises the American Bureau of Shipping (ABS) Rules for Building and Classing Mobile Offshore Units (MOU). The design review compared the design approach of these standards with Det Norske Veritas (DNV) standards, which are typically used for offshore wind turbine foundation design. This included the review of twenty design reports and their respective appendices, along with the offshore standards shown below:

- Det Norske Veritas, 2012. DNV-OS-J101 Design of Offshore Wind Turbine Structures;
- Det Norske Veritas, 2010. DNV-RP-C203 Fatigue Design of Offshore Steel Structures;
- Det Norske Veritas, 2011. DNV-OS-C101 Design of Offshore Steel Structures, General;
- Det Norske Veritas, 2010. CLASSIFICATION NOTES No. 30.7 Fatigue Assessment of Ship Structures;
- American Bureau of Shipping, 2008. Guide for Building and Classing Mobile Offshore Units;
- American Bureau of Shipping, 2008. Rules for Building and Classing Mobile Offshore Drilling Units;
- American Bureau of Shipping, 1997. Rules for Building and Classing Offshore Installations.

The review process was aimed at identifying the key differences in design approach and highlighting any residual design risk not addressed by the ABS MOU approach. The design reports were summarised, with the key information included in an ETG published technical report (T5), whose highlights can be found in Chapter 5. The review demonstrated that there was no residual design risk due to the use of the ABS MOU approach, with no significant design challenges and a fundamentally sound MM concept, including transportation, installation and operational loads, all of which had been certified by ABS.

### **4.3.2** FOUNDATION CONCEPT REVIEW

The satisfactory results of the MM design review, along with the operational experience from the successful deployment of the MM at Södra Midsjöbanken, justified further investigation of the foundation to support an offshore WTG. This included a review of design and cost documentation submitted for the following development sites:

- RES St Brieuc Design Basis (France);
- E.ON Rampion Design Basis (UK);
- E.ON Arkona + Kriegers Flak Design Basis (Germany + Denmark);
- Carbon Trust Steel Foundation Benchmarking Study (UK);
- Statkraft Dogger Bank Design Basis (UK).

For the design basis studies, site specific details on met-ocean conditions, water depths, turbine size, and soil conditions were provided to the foundation concept designers by E.ON. This enabled a high level feasibility and cost indicator to be produced and benchmarked with other novel and traditional foundation concepts. This information was then used to undertake an overall review of the concept, with an ETG report produced (T1), a redacted version of which is included in Appendix G. The report provides a summary of the development of the MWP concept from its introduction to the market in 2009, through the Carbon Trust's Offshore Wind Accelerator (OWA) programme competition, up until the end of 2013. Information from the design basis were also summarised and evaluated, with data provided used to estimate the financial breakeven point, in terms of the distance from shore at which

the installation cost savings equal the relatively high fabrication costs of the foundation. The data also allowed for more representative comparisons to be made for benchmarking at certain sites, where representative information was not provided on cost, etc. The technical assessment of the foundation concepts was based on:

- Concept maturity;
- Design variance;
- Fabrication complexity;
- Degree of standardisation;
- Flexibility on fabricators;
- Flexibility on installation vessel;
- Extent of pre-installation activities;
- Offshore installation time;
- Environmental constraints.

# 4.3.3 STRUCTURAL CONDITION MONITORING

The MWP foundation review indicated that there was potential for the self-installing concept to offer cost savings over traditional foundations, such as monopiles and jackets, at far shore locations and be competitive for some of E.ON's mid-term development sites. The analysis of cost data by the research engineer of various foundation concept designs for the MWP and Jacket foundations at different wind farms development sites, indicated a cost saving over jacket foundations could be achieved at wind farms around 180km from shore. At this point the relatively lower installation cost of the MWP concept outweighed the higher fabrication costs. The details of this analysis can be found in as shown in Section 3 of Appendix G. As such, further research and development was undertaken to help reduce the risk and cost of this concept as a future offshore WTG foundation. This included the research engineer reviewing independent design studies undertaken by other offshore wind developers and a detailed design review of the Met Mast foundation. These reviews are described in detail in Sections 3.5 and 3.6 of Appendix G respectively. As part of this advancement, it was necessary to undertake SCM of the met mast foundation to enhance the understanding of the operational behaviour of the structure and compare it to the predicted responses of design that had been highlighted by the design review, allowing validation of the structural design and assumptions. However, during the project, the research engineer was made aware that the future of the E.ON asset could not be guaranteed beyond April 2014, when the current environmental monitoring campaign came to an end. As such, the SCM had to be installed before November 2013 to ensure a significant storm event was captured, as this event would result in higher utilisation and responses in the structure and therefore provide better quality and more useful data. Ensuring instrumentation could occur before weather windows became too short was also important to make sure the risk of weather downtown was minimised to keep the cost of installation to a minimum. The FEM outputs and summaries of the highest stresses reported in the MM design documentation were used to aid in identifying potential locations for monitoring equipment e.g. strain gauges. Based on this and information gained from reviewing publications on historic structural condition monitoring campaigns on oil and gas jack up platforms, a technical specification was produced by the research engineer for competitive tender for the installation of the structural condition monitoring. After scope optimisation, the costs returned were in the order of €350k with an additional €350k vessel costs including 50% weather delay. Through technical and financial evaluation of the tenders, the preferred contractor (Fugro) was recommended to the procurement manager based on their superior technical and HSE methods. A detailed design meeting was held in Malmo (Sweden) along with an offshore site visit in early August 2013, where final details were clarified and any access and installation challenges discussed and resolved with MWP, Statkraft, ETG, Bassoe Technology and Fugro. Bassoe Technology were the subcontractors employed by MWP for detailed design aspects. A project proposal including costs, expected outcomes and benefits were presented to Statkraft, MWP and renewable energy company RES, who all expressed interest in participating in a collaborative research project. Unfortunately, one of SS's jack-up vessels was delayed at a previous contract, and so its planned use for installing the SCM while already mobilising to Södra Midsjöbanken could not be exploited. This required an alternative less suitable vessel to be sourced which resulted in an increased cost estimate due to a longer programme. Despite MWP formally agreeing to cover all vessel costs and Statkraft all data analysis and modelling costs, Stratkraft's and RES's initial indications of financial contributions were withdrawn and so the remaining cost was beyond what was internally agreed within E.ON to be cost-effective. A decision was therefore made in October 2013 by the head of T&I renewables not to install the SCM until the future of the structure was confirmed in mid-2014 and outcomes of the various cost benchmarking evaluation studies were finalised. This has resulted in no instrumentation being installed to date.

# 4.4 RP4 - OPTIMISING PARABOLIC SOLAR TROUGH FOUNDATIONS' DESIGN

A literature review was undertaken to provide background information to the project and justify further research and development in parabolic trough technology through comparison with the other commercially developed technologies. This demonstrated the parabolic troughs advantages over other technologies due to its relatively low capital cost, low site area requirement and high efficiency.

A literature and commercial case study review highlighted four main commercially used foundations, details of which can be found in Section 3 of the published conference paper (C1), Appendix F. The four main foundation types considered per trough end frame are shown in Figure 4.17 which include:

- Reinforced Slab;
- Pair of Mini Piles;
- Pair of Augured Caissons;
- Single Augured Caisson.

The complex parabolic shape of the solar collector, along with their rotation around the horizontal axis as it tracks the sun during the day and the surrounding array of collectors in the solar field, results in very complex air flow and therefore significant variations in wind load around the array. To address this variability of the wind loading on parabolic troughs, pressure and drag coefficients derived from small-scale wind tunnel testing (Hosoya et al., 2008) were used as inputs to a numerical model developed by the research engineer. Wind tunnel test results were used as the literature review had highlighted this was the best available data, with no information available on full scale measurements. The inputted coefficients allowed for the spatial variation of horizontal and vertical forces and overturning moments to be derived across the solar array based on Equations (4.9) to (4.12) (Hosoya et al., 2008) and the SCA specifications shown in Table 4.7. The extent of this variation in load across the solar array is shown in Figure 2, Appendix F.





$$F_{z} = qLWC_{fz}$$
(4.10)

 $M_y = qLW^2C_{my}$ Pitching Moment,  $M_y$ 

where

$$q = \frac{\rho_a V^2}{2} \tag{4.12}$$

(4.11)

is the mean dynamic pressure measured at the pivot point height of the solar collector; L is the length of the collector; W is the collector surface height; V is the mean wind speed at pivot height;  $C_{fx}$  is the horizontal force coefficient;  $C_{fz}$  is the vertical force coefficient;  $C_{my}$  is the pitching moment coefficient and  $\rho_a$  is the density of air.

Table 4.7 Dimensions of typical Eurotrough variant (Kearney, 2007)

Collector Structure	Torque tube + stamped steel cantilever
WIND SPEED DESIGN BASIS (m/s)	33
<b>APERTURE WIDTH (m)</b>	5.76
FOCAL LENGTH (m)	1.71
LENGTH PER COLLECTOR MODULE (m)	12
LENGTH PER SCA (m)	148.5

The loads were then used to review the current designs being used at Helioenergy in order to provide information for due diligence on E.ON's investment. The checks were performed using BS EN 1997-1 foundation calculations looking at safety factors for bearing capacity, horizontal sliding and uplift for four load cases (inner field, corner outer-field, corner outer-field with barrier and 2<sup>nd</sup> row). The designs were then optimised using the numerical model developed, based on achieving a minimum overall safety factor of 2 across the various conditions while minimising material volumes. The outputted material quantities were then used to determine costs based on Langdon (2010). Details and results of which can be found in Section 3 of the conference paper (C1), Appendix F, and examples of the foundation design equations can be found in Appendix B.

As well as demonstrating significant optimisation of the foundations could be achieved, the analysis undertaken by the research engineer also demonstrated that the loads experienced when wind speeds were greater than the operational limit of 14m/s and the collectors were rotated in their stowed positions (-90 degrees) could be reduced if the stow angle was adjusted to -100 degrees. In the case of pilled foundations, this increased the survival wind speed of the foundation from 23m/s to 26m/s, the details of which can be found in Section 3 of Appendix F.

As the available information was considered to have missing data and limitations due to being derived from small scale wind tunnel tests, an improved understanding of wind flow through the field of parabolic trough collectors was required. It was therefore recommended that further work was carried out in the form of onsite monitoring and CFD modelling. This would allow further optimisation of the SCA and foundation design through exploiting lower factors of safety due to higher certainty in loads. CFD modelling of the collector geometry and spacing found at Helioenergy was therefore undertaken by the ETG software modelling team.

A comparison of the results of CFD modelling and the original wind loading that was used for the optimisation of the foundations was undertaken by the research engineer and resulted in the production of an ETG technical report (T8). The report highlighted discrepancies between the two modelling techniques and where possible, these discrepancies (air density and wind speed reference height) were compensated for, allowing for a more accurate comparison, Figure 4.18. This showed good agreement in the change in vertical and horizontal load with trough angle for the 1<sup>st</sup> row collectors and vertical load for the 2<sup>nd</sup> and 3<sup>rd</sup> row of collectors apart from at 90 degrees. However, the horizontal load between -20 and 25 degree for the 2<sup>nd</sup> and third row collectors shown significant variation in load between the CFD and wind tunnel results. The increasing dissimilarity between the two modelling techniques' forces with increasing row number is likely to be due to the CFD model not accurately capturing the vortex shedding effects on the collector and the turbulence caused by the upstream collectors.

Based on the percentage change of the CFD forces in comparison to the wind tunnel coefficient derived forces, the horizontal and vertical coefficients of the wind tunnel testing were changed accordingly in the foundation optimisation numerical model. This provided a basic estimate of the effect of this change in relation to factors of safety originally used from the foundation design equations. Given the largest difference was seen in the vertical force at a pitch of around 90°, with the CFD forces being 2.74 and 2.16 times greater than those derived from the wind tunnel for rows 2 and 3 respectively, this position was investigated. The effect was assessed on the pile type foundation as this was shown to be most susceptible to change in vertical force. This was shown to have marginal effect in reducing the factor of safety on the pile design used at Helioenergy, see Table 4.8.

#### Table 4.8 Effect of increased load due to CFD estimate on pile foundation factors of safety

FAILURE MODE

	Row	SLIDING	BEARING	UPLIFT
ORIGINAL WIND TUNNEL DERIVED FACTORS OF	$2^{nd}$	>10	5.8	8
SAFETY	$3^{rd}$	>10	7.9	8
UPDATED FACTORS OF SAFETY BASED ON CFD		>10	3.6	>10
		>10	6	>10



Figure 36 Comparison of CFD and wind tunnel derived forces for 5m/s wind speed horizontally (left), vertically (right), row 1 (top), row 2 (middle) and row 3 (bottom)

# **5 FINDINGS & IMPLICATIONS**

This chapter provides the key findings, contributions to existing theory and practice, impact on the sponsoring company and wider industry, recommendations for further research and critical evaluation of the research undertaken as part of the Engineering Doctorate.

# 5.1 KEY RESEARCH FINDINGS

# 5.1.1 **RP1 - Effect of Grouted Connection Wear on Offshore Wind Turbine Foundations**

## **Experimental Testing Literature Review**

The most extensive critical review of grouted connection experimentation to date (presented in Appendix C and summarised in Section 4.1.1) has highlighted that:

- Limited testing has historically been undertaken on configurations that are directly relevant and applicable to offshore wind turbines;
- Understanding the limits of design equations and the reasoning behind those limits, i.e. the scale and material properties of the testing undertaken to derive the relationships, is critical to ensuring satisfactory lifetime operation. Extrapolation beyond these limits should be supported by additional testing;
- Previous test results should be considered in new applications of existing concepts, as the review of historical grouted connection testing highlighted the potential risk of reduced axial capacities of the connection when experiencing combined axial loads and bending moments;
- Implications of installation procedures can be critical to operational response, with historical testing showing reduced grout properties for the first few metres of injected grout due to the mixing of the grout with sea water as it is injected into the grouted connection;

- Knowledge sharing between designers, standard bodies and operators is required to avoid such wide spread occurrences of insufficient designs and aid in the optimisation of future designs;
- Need for more research in this area, which has partly been addressed by the research presented here and is discussed further in Section 5.5.

# **Experimental Testing**

For the first time in the literature, it has been proved through an innovative experimental setup (presented in Appendix D and summarised in Section 4.1.1) that:

- Wear occurs under compressive stresses and environmental conditions experienced by offshore WTG GCs where relative movement occurs between steel and grout;
- Not only does wear occur, but it was found that the influence of surface finish on wear rate is small compared to the presence of water. Wear rates for different relevant conditions experienced by GCs in offshore structures have therefore been derived. This has been done for the first time in the literature;
- Wet connections show wear rates up to 18 times larger than in dry connections for the same conditions;
- Wet coefficient of friction can be lower than the DNV recommended design evaluation values.

# **Numerical Model**

The work undertaken by the research engineer presented in Appendix E and summarised in Section 4.1.3 has shown that:

• A numerical model can be developed that uses low frequency SCADA data to predict wear, the accuracy of which has shown to be good based on the SCM indications of

wear, thus demonstrating the creation of a tool that can accurately predict the development of wear in offshore structural grouted connections, the first within the industry;

- Under conditions experienced by the WTG structure over a representative period, grout wear appears to be in the order of 3 to 5mm at the very top and bottom of the GCs over their 20-year lifetime across the wind farm, based on both SCM and numerical model predictions, (assuming that no significant changes in the structural response or environmental conditions occur);
- Due to the significant stress concentration factor at the top and bottom 0.7m of the GC, wear over the majority of the GC is insignificant, being less than 0.4mm. This provides to the offshore wind industry the first scientific evidence that there is limited risk of the wear failure mode over the life of the connection.

# **5.1.2 RP2 - EXAMINING LOADS IN OFFSHORE WIND TURBINE FOUNDATIONS** Key findings of Section 4.2 included:

- A significant relationship between measured wind speed and strain within the structure has been derived from an operational wind farm. Therefore allowing prediction of loads and measurement of load histories for each WTG location from the readily available SCADA data, avoiding the need for expensive SCM on every WTG structure;
- Analysis of the 10-minute average strain, wind speed and direction during power generation indicate that the measured strain is similar to the strain derived from the WTG design thrust coefficients. Demonstrating that this can be a useful tool for monitoring structural utilisation over the design life of the foundations.

It was also found that:

• During non-power production, structural strain responses are higher than predicted. However, as long as WTG availability is high, non-power production durations will be low and therefore the influence over the design life of the foundation will be relatively small.

# 5.1.3 RP3 - Assessment of the SS MWP Mk. 2 Foundation concept

The key findings from this work package (Section 4.3 and Appendix G) were:

- The detailed design reviews of the MWP Mk. 2 concept highlighted that fundamentally it is feasible and can meet the key ULS and frequency requirements. However, they indicate that FLS requirements are challenging to achieve, so further work is required to ensure these are satisfactory and the implications are included in the structural weight and therefore cost estimates;
- For a WTG foundation, technically the concept provides a feasible alternative to traditional foundations offering minimal noise impacts and ease of installation;
- Available information indicates that under favourable conditions the concept could become competitive at distances greater than 200km from the shoreline. The number of potential sites is therefore likely to be limited and would be unlikely to be high on the development priority;
- Significant fabrication cost reductions need to be made to enable any serious consideration to be provided. Utilisation of a temporary jacking system may help to achieve this, but additional technical challenges will need to be addressed.

# 5.1.4 RP4 - Optimising parabolic solar trough foundations' design

For the first time within the literature it has been shown in Section 4.4 and Appendix F that:

- The slab foundations typically used on commercial CSP plants have considerable factors of safety and so optimisation can be achieved on this foundation type, with a significant saving in terms of material volume;
- The results of the analysis of initial typical site designs indicate that the mini-pile design could be susceptible to uplift when site wind speeds are greater than the maximum operating wind speed of 14m/s and the collector is in the stow position, but stow position optimisation could improve the situation, thus reducing the risk to the collectors;
- The caisson design achieves high factors of safety for all failure modes at operational wind speeds and therefore can be optimised. A financial comparison indicates that the single caisson design could represent the most cost-effective solution for CSP foundations, based on the assumptions made;
- Overall, the calculations suggest that foundation design optimisation is possible and significant financial savings can be made in the construction of foundations in the solar field of parabolic trough CSP projects, with this work indicating a possible saving of around £1.8m on a small 50MW example case study.

It was also found that:

• The comparison of results of CFD and wind tunnel testing derived forces were generally in good agreement for the 1<sup>st</sup> row of troughs over the majority of orientations, but this reduced for the 2<sup>nd</sup> and 3<sup>rd</sup> rows over a wider range of orientations;
- The NREL wind tunnel data generally produced higher horizontal loads than the CFD modelling, with a maximum difference of 35% for the 1<sup>st</sup> row, but up to 100% for the 3<sup>rd</sup> row;
- Vertically, there was good agreement for the first collector with the CFD force 50% of the wind tunnel derived forces at 45°. However, for the 2<sup>nd</sup> and 3<sup>rd</sup> row from pitch angles between 50 and 100° significant difference between forces were found, with the CFD forces around 250% higher at a pitch of 90°;
- The implications of this increased vertical loading was assessed on the pile designs and showed a marginal effect in reducing the factor of safety on the design used at Helioenergy;
- The CFD analysis has shown that horizontal forces are reduced if the convex surface is directed towards the oncoming wind;
- Forces appear to scale well with size and velocity. The CFD analysis has provided a fair degree of confidence that forces scale proportionally with parabola aperture dimension and are proportional to the square of the head-on wind speed. Initial designs of foundation for troughs of different size can therefore be undertaken by scaling the forces;
- Wind barriers significantly reduce the forces on the first row of collectors and therefore factors of safety on the first row foundations can be reduced, saving materials, installation time and cost;
- The presence of a wind barrier results in the 2nd and 3rd rows experiencing higher forces than the first row, so that these need to be optimised for the expected maximum wind speed likely to be experienced during the lifetime of the plant.

# 5.2 CONTRIBUTION TO EXISTING THEORY AND PRACTICE

# 5.2.1 RP1 - EFFECT OF GROUTED CONNECTION WEAR ON OFFSHORE WIND TURBINE FOUNDATIONS

The experimental testing undertaken as part of RP1 has formed part of the second Joint Industry Project (JIP) on grouted connections managed by DNV and supported by operators, grout manufactures, international standard bodies and academics. A review of the research undertaken as part of RP1 by DNV has resulted in them expressing interest in the results forming part of the new international design code for offshore WTG structures, DNV-OS-J101, as the research represents the most comprehensive and only information of grout wear in grouted connections specific to operational conditions experienced. The research also provides both input values and the methodology to be able to estimate wear based on the GCs specific details.

## 5.2.2 RP2 - EXAMINING LOADS IN OFFSHORE WIND TURBINE FOUNDATIONS

The research undertaken as part of RP2 has provided an efficient and robust way of using readily available SCADA data to indirectly determine structural responses in WTG substructures. In turn, allowing the loads experienced by the foundation to be monitored or even predicted, based on environmental condition predictions, over its design life. Prior to this research only limited information about the operational response of offshore windfarms had been analysed and this has all been based on dedicated expensive SCM systems. This lack of information on the loads of offshore wind turbines results in the partial safety factors applied to the loads being higher to account for this uncertainty. If these factors are excessively high it leads to an over conservative and therefore unnecessarily expensive structure. If the factors are too low this can results in a structure with insufficient capacity that is likely to fail within it design life. The research methodology presented therefore demonstrates how the loads of the structures throughout an offshore wind farm could be

measured with only a small cost effective sample of foundations having SCM, which would help to reduce the uncertainty of loading on offshore wind turbine structures. A literature review of published research indicates that the development of such a tool has not been undertaken previously, and therefore represents a novel contribution to theory and practice.

The research has also highlighted the need to understand the actual loads and environmental conditions experienced so that this information can be fed back into the design methodology and models to improve their accuracy. This will therefore help to ensure that future designs are fit for purpose and optimised for the loads and environmental conditions they will experience. This will not only help to reduce CapEx through reduction in steel quantities, but also OpEx through targeting the structures which monitoring indicates are experiencing the highest loads or showing significant changes is structural response. Therefore allowing a targeted inspection strategy and reduced inspection cost.

# 5.2.3 RP3 - Assessment of the SS MWP Mk. 2 Foundation concept

The research undertaken as part of RP3 represented the most comprehensive external review of this novel foundation concept reported across the offshore wind industry. However, the decision not to proceed with SCM of the MWP structure resulted in limited novel contributions to the theory or practice compared to what had originally been envisaged when the project was first undertaken.

# 5.2.4 RP4 - Optimising parabolic solar trough foundations' design

The research undertaken as part of RP4 demonstrated for the first time in the literature how existing data on small-scale wind tunnel tests could be used along with site specific conditions to more accurately determine the complex wind loading experienced by CSP parabolic troughs during operation. The use of these load predictions was to assess the conservativeness of existing foundation design at a case study site and therefore assist in identifying potential optimisation. This along with recommendations for optimum foundation choices represents a significant contribution to theory and practice in this area, with no other research in these areas found when undertaking the literature review.

# 5.3 IMPLICATIONS/IMPACT ON THE SPONSOR

# 5.3.1 RP1 - EFFECT OF GROUTED CONNECTION WEAR ON OFFSHORE WIND TURBINE FOUNDATIONS

The research was able to determine, to the best of the available information and the theory obtained and developed as part of this research, the magnitude of the wear for the conditions actually experienced and statistically likely to occur over the design life of the OWF. It was therefore possible to demonstrate that there was unlikely to be a risk to the foundation integrity due to wear at the 60 WTGs owned and operated by E.ON with a capacity of 180MW and investment of  $\pounds$ 20m. It also demonstrate that the worst affected areas would be on the predominantly windward side at the very top of the GC, which could be inspected to monitor for degradation. This therefore helps to focus and minimise inspection work, reducing operational costs to the company. The findings of this research have been disseminated throughout the company via the production of an ETG issued technical report (T2).

E.ON also has a 30% stake in the joint venture of London Array, representing an investment of  $\notin$ 16m. Although not a plain-pipe GC, as it utilises a conical section to ensure there is negligible risk of the long term settlement seen in previously constructed GCs, there is still potential for the conditions required to promote wear to occur in terms of relative displacements, normal compressive stresses and the presence of water. The methodology presented here could therefore be used to determine the risk of wear for this type of connection with appropriate input factors entered.

Although the current development portfolio up to 2020 is focusing on the use of XL monopiles, these foundations are likely to utilise bolted connections after the recent positive experiences at E.ON's recently commissioned Humber Gateway in the UK and Amrumbank in Germany. However, post-2020 development sites within E.ON are likely to either utilise jacket structures due to deeper waters with larger WTGs or MPs with submerged grouted connections to minimise MP weight. A modified methodology of that developed here could therefore be used to undertake due diligence on the GCs of these structures.

# 5.3.2 **RP2 -** Examining loads in offshore wind turbine foundations

The research has led to the production of a numerical tool which can be used to determine with an acceptable level of confidence the loads experienced by the structure using readily available SCADA data present on all WTGs. With further development in terms of increased population of monitored WTGs across the wind farm over the two currently instrumented foundations, sufficient representation and significance of the results would enable the SCADA data to be used to reduce the frequency of expensive offshore inspection and prioritisation to be made, significantly reducing the O&M costs. As BSH requirements are for 10% of the foundations installed to have SCM (Federal Maritime and Hydrographic Agency, 2007), there will be limited additional CapEx to acquire the required information for the current Amrumbank and future projects located in German waters. Through presentation of the findings to the foundation package and engineering managers of E.ON's current and future development offshore wind sites, they are now aware of the potential benefits of this research.

# 5.3.3 RP3 - Assessment of the SS MWP Mk. 2 Foundation concept

The work highlighted the suitability of the MWP concept as a MM foundation and provided information on the limitations of design. Through production of the ETG technical report (T1), future project development managers have a resource that they can utilise to ascertain the suitability of the foundation for future sites. The work was also used to form the basis of work undertaken, screening potential development sites for redeployment internally and externally, enabling the project development manager to determine the potential market for the asset.

The benchmarking and evaluation of the concept with other foundations allowed for an internal assessment of the concept to be presented to the head of T&I, enabling an informed decision on whether the benefits of supporting the development of the concept would be likely to be realised for the future development sites of E.ON. The concept review also allowed for E.ON's project foundation managers to be informed of the suitability and maturity of this foundation concept, and therefore whether it should be included in the future foundation tendering processes.

# 5.3.4 RP4 - Optimising parabolic solar trough foundations' design

Through the production of the ETG technical reports (T6-8) as part of this research, it was demonstrated to E.ON's CSP management that the foundation design used at Helioenergy was conservative and represented minimal risk from an integrity perspective to the 275m project investment made. Through a more detailed understanding of the loads applied and improved certainty, a reduction of the overall factors of safety of the foundation design can offer the company potential savings in the order of £1.8m/50MW if further plants were constructed, Appendix F.

The CFD analysis also highlighted that adjustment for the currently used stow position to a position with the curved back surface of the mirror towards the wind, not only reduced horizontal forces, but also reduces potential damage to the mirror surface, thus reducing O&M costs on current plant and both OpEx and CapEx for future plants. However, the realisation

of the research impact has been limited so far due to E.ON's decision to focus development on Photo Voltaic (PV) technology over CSP and so no further projects have or are likely to be invested in, in the near future. It should be noted though, that the principles of this work could be applied to optimise PV foundations across the PV array.

# 5.4 IMPLICATIONS/IMPACT ON WIDER INDUSTRY

# 5.4.1 **RP1 - EFFECT OF GROUTED CONNECTION WEAR ON OFFSHORE WIND TURBINE FOUNDATIONS**

Information has been widely disseminated through various presentations given to industry representatives at offshore foundation forums set up to share knowledge across the industry as a result of the unexpected settlement. Through the publication of three journal papers (J1-3) and two conference papers (C2 and C3) related to this research topic, along with DNV proposing that the research will form part of the new edition of DNV-OS-J101, it will ensure that the whole international industry has access to the methodology and representative wear rates to assess existing GCs for the risk of wear. With over 600 foundations having been reported to be affected by unexpected settlement, this represents a significant impact with an approximate asset value of  $\pounds$ 2.2bn. The research also allows designers and developers to check new designs of both monopile and jacket GCs for the potential of wear to occur.

# 5.4.2 RP2 - EXAMINING LOADS IN OFFSHORE WIND TURBINE FOUNDATIONS

Through presentation of the findings of this research to foundation experts across multiple OWF developers at a foundation forum in Copenhagen, the research has been disseminated to key people in the industry. The research can then be utilised to achieve the impacts described in Section 5.3.2.

# 5.4.3 RP3 - Assessment of the SS MWP Mk. 2 Foundation concept

The confidential nature of the intellectual property of the foundation concept and specific cost figures, originally presented in the redacted ETG technical report (T1) produced as part of this

research, mean that, unlike the rest of the research projects, the information has not been widely disseminated across the industry. However, the feedback provided to the concept designer through the detailed design review will enable development and improvement of the concept, helping to strengthen its market position. Circulation of a redacted version of the report to the parties immediately involved in the potential collaboration project also helped to demonstrate the competiveness of the foundation concept under certain site conditions. This has all helped to support the development of the concept to bring it closer to achieving deployment as a commercial WTG foundation, and therefore realisation of potential cost savings, which for projects such as St Brieuc could help achieve a reduction in CapEx of £50m when compared to a traditional jacket foundation.

# 5.4.4 RP4 - Optimising parabolic solar trough foundations' design

Through the publication of the conference paper (C1), the research can have impact throughout industry. Other developers are continuing with projects, with over 7,500MW of CSP in development across the world (USDoE, 2011) and 95% of installed capacity being parabolic trough technology (CSP Today, 2014). Therefore, in addition to the impacts discussed in Section 5.3.4, a potential saving could be made of £270m across the industry through optimisation of the foundations. For new development sites the analysis undertaken has also shown that the benefits of a wind barrier are very significant. If soil from site levelling activities is insufficient to create earth banks, at least to the height of the top of the trough, then the use of a porous barrier fence is recommended, as the reduction in loading can represent significant reduction in foundation sizes on the first row and therefore CapEx.

# 5.5 RECOMMENDATIONS FOR INDUSTRY/FURTHER RESEARCH

# 5.5.1 **RP1 - EFFECT OF GROUTED CONNECTION WEAR ON OFFSHORE WIND TURBINE FOUNDATIONS**

To improve the accuracy and significance of the model predictions, it is recommended to:

- Perform FE analyses to demonstrate the accuracy of the transfer functions used within the numerical model for the compressive stress within the grout, as the SCF makes a significant difference to compressive stress magnitude and therefore wear.
- Upon installation of the remedial solutions and proposed SCM of additional foundations, relationships between vertical displacement and wind speed need to be evaluated to confirm there is no detrimental effect on wear through installation of the elastomeric bearings and reduction in axial stiffness through removal of the hard contact currently provided by the jacking brackets.
- Undertake a review of the implications on dynamic response of the Robin Rigg structure due to the findings of loss of thickness of the upper and lower 0.7m of the grouted connection by numerical modelling and the representative coefficient of friction indicated for wet connections by experimentation.
- Develop an understanding of the influence of the flexure of the TP and MP walls on localised wear rate and the resultant development of the wear profile.
- Execute a comparison of higher frequency SCADA and SCM data to improve relationships derived in the model, as significant scatter can be caused by the 10-minute averaging period.

# 5.5.2 RP2 - Examining loads in offshore wind turbine foundations

To provide a more accurate prediction of strain responses, it is recommended to:

• Incorporate wave loading, especially during non-power generation events, as this will help develop a better understanding of non-aerodynamically damped loads and will provide improved accuracy in the prediction of strains;

- Undertake analysis on an increased population of the wind farm to improve the significance of the findings and the accuracy of the model. As part of the remedial works, an increased number of WTG structures will be instrumented, and so additional data will be available.
- Perform higher frequency data analysis to capture the dynamic response of the structures, and therefore consider peaks rather than average strains. Capturing of dynamic responses will also allow for fatigue monitoring to be undertaken.

# 5.5.3 RP3 - Assessment of the SS MWP MK. 2 Foundation concept

Given the potentially favourable CapEx possibility of the MWP Mk. 2 concept for far shore sites, it is recommended that:

- MWP be included in future foundation screening for both MM and WTG foundations for far shore sites.
- Continued technology tracking undertaken through supplier engagement to ensure that the latest developments are available for foundation option decisions, improve certainty in cost estimates and ensure that any opportunity to realise potential cost saving is exploited.
- Consider the feasibility of structural condition monitoring of the MWP Mk. 2 MM foundation at Södra Midsjöbanken if circumstances change and MWP demonstrate that fatigue requirements can be met and cost benefits of the revised jacking system. This will aid in de-risking the structure and will improve knowledge on structural performance.

# **5.5.4 RP4 - OPTIMISING PARABOLIC SOLAR TROUGH FOUNDATIONS' DESIGN** To realise the potential of this research, it is recommended to:

- Consider further laboratory modelling supported by on-site measurements to enhance current understanding of key parameters influencing solar collector foundation design.
- Calibrate the CFD modelling results with a proposed on-site monitoring campaign of the loads experienced by the foundations and the environmental conditions.
- As part of validating the CFD model forces, investigate evidence of regular timedependent fluctuations that could be due to the presence of vortex shedding. If present, consider running an alternative CFD formulation that can analyse timedependent vortex shedding effects although it should be recognised that this can be computationally very demanding.

It is also recommended to:

• Specify that the stow position and the control algorithms can be flexible to take advantage of an optimised stow position. This not only minimises loading on the entire structure, but also reduces potential damage to the mirror surface.

# 5.6 CRITICAL EVALUATION OF THE RESEARCH

# 5.6.1 **RP1 - EFFECT OF GROUTED CONNECTION WEAR ON OFFSHORE WIND TURBINE FOUNDATIONS**

The production and acceptance of multiple journal and conference papers along with the potential inclusion within DNV-OS-J101 clearly demonstrate the merit and value of the research undertaken as part of this project. Reasonably accurate and significant results have been achieved given the available data, equipment and finite time for design, production and execution of testing. The numerical model developed offers a simple and robust method for the prediction of wear that is easy for practitioners to utilise and has demonstrated the limited risk to foundation integrity. In completing this, the aim and objectives of this project outlined in Section 2.1 have been achieved.

However, the constraints of available testing frames and ensuring sufficient degrees of freedom to reproduce the correct wear action resulted in sample stability and constant compressive stress application during testing not being ideal. Improvements made as part of the development of the test equipment helped to improve the stability, but due to time constraints a full redesign could not be undertaken. If the experimental testing was to be undertaken again, more time would be invested in the initial design and development of the testing apparatus. This would not only have resulted in less variation in the results, due to more constant conditions, but also a quicker, less user-intense testing programme. The lack of experience with the testing equipment available onsite when technical challenges arose also proved detrimental to the testing programme. With more time available, more repetitions of samples for the different conditions would have been undertaken.

The wear numerical model developed and SCM data used are only based on a three-month period from one foundation, with the other foundation's data used for validation, due to the limited availability of data. However, this only represents 1/60<sup>th</sup> of the foundations affected by wear and a 1/240<sup>th</sup> of the design life of the structures; a longer duration and number of SCM locations would have helped to improve the significance of the results.

The assumption used in terms of stress concentration and distribution are based on limited available information. Detailed FE modelling of the connection would have helped provide a better understanding of the stress distribution, but time and resource constraints prevented this part of the research from being carried out.

The model assumes constant structural behaviour on the grouted connection. However, this has the potential to change as the wear occurs and so FEM and structural condition monitoring of a connection after a prolonged period of wear would have been useful to

understand this change. The limited period of available data precluded the latter, but FE analysis could have been undertaken if more time and resources were available.

# 5.6.2 RP2 - EXAMINING LOADS IN OFFSHORE WIND TURBINE FOUNDATIONS

A simple and robust procedure has been used, which has enabled a comparison of the measured strains and strains derived from OEM WTG specifications during power production. The tool developed as part of this research strand offers a better understanding of the loads experienced across a wind farm by the WTG structures.

Improvement to the strain response prediction accuracy of the structure could be achieved through incorporation of wave and current loading information, but due to time constraints and availability of data, information on wave magnitudes available from a site MM was not incorporated. In addition, a lack of high resolution data was available from the site on current velocity and sea bed depths, thus loads and structural response associated with these would be unknown. As there is also no information available on the wave direction, the implication of possible misalignment between the wind and waves on the structure would remain unknown.

# 5.6.3 RP3 - Assessment of the SS MWP Mk. 2 Foundation concept

A detailed assessment of the MWP foundation concept was undertaken for use as both a MM and offshore WTG foundation. Through inclusion of a detailed design review based on available design reports, a thorough understanding of the technical maturity and commercial competiveness of the concept was determined. As such, the aim and objectives of this research project outlined in Section 2.3 were achieved.

Although from the sponsoring company's perspective the objectives of the work have been achieved, without the installation of SCM to the offshore structure the novel impact of this research package was limited. If SCM had been installed to the foundation supporting a MM, it would have offered an opportunity to improve industry and academic knowledge, not only on the structures response, but on environmental loading in water depths that represent boundaries between different wave theories.

# 5.6.4 RP4 - Optimising parabolic solar trough foundations' design

Through the review of the site design documentation and wider information on site specific and general foundations, the foundation types and designs used for CSP parabolic troughs have been established. A literature review allowed data to be acquired to establish an estimation of the variation of wind loading across the solar array for the parabolic trough dimensions on site. This, along with site specific foundation designs has enabled their optimisation. The uncertainty of the applicability of the data used to determine the wind loading has been reduced through comparison of forces with CFD modelling of the sitespecific array layout.

Unfortunately, the management decision not to actively support the development of CSP technologies meant that the recommendation for a full-scale field monitoring campaign was not undertaken to validate site assumptions on characteristic design wind speeds, soil parameters and loads experienced by the foundations across the solar array. This would have represented novel research with international impact, and allowed for further optimisation through higher certainty in the design assumptions and loads.

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# APPENDIX A - WEAR NUMERICAL MODEL CALCULATIONS

# **Example of Input Data**

Input SCADA Data	from PI for foundation	being investigated										Weather Data				
		(1)				(2)	)			(3)						
		Data from PI				Foundation Sp	pecifications		Locati	on of Wea	r			Design Data		
Date/Time	wind direction [°]	wind speed [m/s]	Power [kW]	Nacelle direction [°]	MP OD		4.3	m	Radial Positio	on	330 °			Wind Speed	Probability	@75m MSL
01/01/2012 00:00	200.4919739	14.75942802	20851664	208	Length o	of grouted conne	ction 6.45	m	Depth from to	op of MP	0 m	Wind Direction	°) Duration (%)	(m/s)		(%)
01/01/2012 00:10	212.2630157	13.16439915	20852148	212.8999939	Grout th	nickness	0.07	m					0 5.7	<b>,</b>	0 0	0
01/01/2012 00:20	222.991394	16.7655468	20852640	227.1999969	MP Thic	kness	0.05	m				3	0 7.1		1 0.0115	1.15
01/01/2012 00:30	219.809845	13.67540932	20853140	227.1999969	TP Thick	kness	0.05	m				e	0 11.3	<mark>;</mark>	2 0.0318	3.18
01/01/2012 00:40	213.3963165	15.99081421	20853640	220.5	internal	conditions	Dry Co	rroded				g	0 <mark>4.8</mark>	<mark>;</mark>	3 0.0494	4.94
01/01/2012 00:50	210.4188843	14.72488976	20854140	219.4238586	Coeffie	cnt of friction	0.7					12	0 3	;	4 0.0636	6.36
01/01/2012 01:00	215.7969055	17.21240425	20854632	220.5	Hub Hei	ight (CD)	84.35	m				15	0 6.8	; ;	5 0.0739	7.39
01/01/2012 01:10	218.6080322	17.92765236	20855132	220.5								18	0 10.4	•	6 0.0801	8.01
01/01/2012 01:20	215.7728271	13.36795616	20855632	220.5								21	0 13.6	5	7 0.0824	8.24
01/01/2012 01:30	212.5932617	14.36155796	20856132	213.1999969								24	0 15.6	5	8 0.0813	8.13
01/01/2012 01:40	212.052536	14.60543156	20856602	218.3999939								27	0 11.7	,	9 0.0776	7.76
01/01/2012 01:50	210.5244751	15.49094677	20857104	211.1000061								30	0 <mark>6.8</mark>	1	0 0.072	7.2
01/01/2012 02:00	208.2284088	16.08522224	20857604	211.1000061								33	0 3.2	. 1	1 0.0652	6.52
01/01/2012 02:10	208.2660675	17.23757553	20858104	211.1000061										1	2 0.0577	5.77
01/01/2012 02:20	221.3820801	16.89999962	20858604	223										1	3 0.0501	5.01
01/01/2012 02:30	217.4697571	15.33952141	20859104	223.6999969										1	4 0.0427	4.27
01/01/2012 02:40	221.3432312	16.66191101	20859604	223.6999969										1	5 0.0357	3.57
01/01/2012 02:50	221.9326935	12.82355976	20860102	229.6000061										1	.6 0.0293	2.93
01/01/2012 03:00	224.3771515	16.31443977	20860602	229.6000061										1	7 0.0236	2.36
01/01/2012 03:10	227.1000061	16.6807251	20861102	229.6000061										1	.8 0.0186	1.86
01/01/2012 03:20	222.252655	17.63134575	20861602	. 224										1	9 0.0145	1.45
01/01/2012 03:30	224.6079559	15.39095402	20862102	230.3000031										2	0 0.011	1.1
01/01/2012 03:40	220.1210327	12.63543224	20862602	. 223.3000031										2	1 0.0082	0.82
01/01/2012 03:50	224.128418	17.85368347	20863102	. 229.3000031										2	2 0.006	0.6
01/01/2012 04:00	217.5358276	16.29624748	20863602	. 223.3000031										2	3 0.0043	0.43
01/01/2012 04:10	223.300705	17.78581619	20864102	223.3000031										2	4 0.0031	0.31
01/01/2012 04:20	227.537674	20.15330696	20864602	. 229.3000031										2	5 0.0021	0.21
01/01/2012 04:30	274.852417	13.81878948	20865100	267.3999939										2	.6 0.0014	0.14
01/01/2012 04:40	262.9586182	17.11402702	20865542	265.2999878										2	.7 0.001	0.1
01/01/2012 04:50	247.7474365	13.35256767	20865962	. 252										2	8 0.0006	0.06
01/01/2012 05:00	252.0394287	14.37319946	20866444	252										2	9 0.0004	0.04
01/01/2012 05:10	253.2938385	12.66610527	20866914	252										3	0.0003	0.03
01/01/2012 05:20	244.7806244	11.88062286	20867394	254.2806244										3	1 0.0002	0.02
01/01/2012 05:30	239.0031586	13.12085629	20867840	240.8000031										3	2 0.0001	0.01
01/01/2012 05:40	235.5801697	13.22478676	20868272	240.8000031										3	3 0.0001	0.01
01/01/2012 05:50	234.3956757	13.76971626	20868744	240.8000031										3	4 0	0
01/01/2012 06:00	230.1123657	14.93251801	20869234	235.1999969										3	5 0	0

	Data Quali	ty		
No. inputs		No	o. non bad	
	wind	wind		
	direction	speed		Nacelle
	[°]	[m/s]	Power [kW]	direction [°]
14485	11449	11449	11434	11449
Quality (%)		0.7	789368312	
Equivalent wear duration (months)		9.4	172419745	

														Transfer fur	nction from	n wind spe	eed and po	ower produc	tion to disp	placemen	nt																			
		Raw (	ata			Cleaned Data																Ver	cal Dinslar	comont h	sed on t	ia														
	Wind	Wine	Active	Nacelle	:	cicalica bata	Wind	Active	Nacelle	Power												Vei	car Dipsia	cement b	ised on t	'5														Chosen
Date	e / Direct	ion Spee	d Power	Directi	0		Speed	Power	Direction	Generatio	on		VD (Non																											Radial
Tim	e [°]	[m/s	Count [k	:W] n [°]	Date / Time	Wind Direction [°]	(V)	Count [kW]	[°]	[kW]	VD (power)		Power)	wind ward \	VD	S1					S2	_		_	53	_		S4				S	55			S6				position
						IF(OR(C5=C3,C5="comm fail",C5="error",C5 = "no data",C5="pt created", C5="failed",C5="bad",C5="i/o timeout"),0,C5)				differecn in active power count	IF(V<12.5,((0.00004789*(V^5)) (0.00146449*(IV^4))+(0.01333 (0.02408712*(V^2)) + (0.00329) 0.0224*I5+1.14496))	- 426*(V^3)) - 9948*15)),(-	V*0.008333	IF(Power ge 0,VD(power 3 Power))	eneration > r),VD((Non	VD*COS	S(RADIANS	S(orientation	n-wind direc	ction))																				
###	##### 200.	492 14.	6 20851	564 2	08 01/01/2012 00:0	200.491973	9 14.75	9 20851664	208	3		0.814348812	0.1229947	42 0.	.122994742	2 0.0	02 0.04	4 0.06 0	0.08 0.09	0.11	0.12 0.	.12 0.12	0.12 0.1	12 0.11	0.09 0.	0.06	0.04 0.0	2 0.00 -	0.02 -0.04	-0.06 -	0.08 -0.09	-0.11	### ###	* ### ##	*** **** **	## -0.09	-0.08 -0	.06 -0.04	-0.02 0.1	JO -0.08
40	909.01 212.	263 13.	20852	148 212	.9 40909.0069	212.263015	7 13.16	4 20852148	212.9	9 4	34	0.850077459	0.1097028	87 0.	.850077459	9 -Q.(	03 0.11	1 0.26 0	0.40 0.52	2 0.63	0.72 0.	.79 0.83	0.85 0.8	84 0.81	0.75 0.	67 0.57	0.45 0.3	2 0.18	0.03 -0.11	-0.26 -	0.40 -0.52	-0.63	### ###	* ### ##	*** **** **	## -0.75	-0.67 -0	.57 -0.45	-0.32 -0.	18 -0.40
40	909.01 222.	991 16.	20852	540 227	.2 40909.0138	39 222.99139	4 16.76	6 20852640	227.2	2 4	92	0.769411752	0.1397123	31 0.	.769411752	2 -0.1	17 -0.04	4 0.09 0	0.23 0.35	5 0.46	0.56 0.	.65 0.71	0.75 0.7	77 0.76	0.74 0.	59 0.61	0.52 0.4	2 0.30	0.17 0.04	-0.09 -	0.23 -0.35	-0.46	### ###	* ### ##	*** ****	## -0.74	-0.69 -0	.61 -0.52	-0.42 -0.	30 -0.23
40	909.02 219	9.81 13.	20853	140 227	.2 40909.0208	33 219.80984	5 13.67	5 20853140	227.2	2 5	0 p15470: Displacement based on	0.p15470: Displacment	hased on	p15470:	nlacomont b	nased on w	0.00	0 0.15 0	0.29 0.42	2 0.54	0.64 0.	.73 0.79	0.83 0.8	84 0.83	0.79 0.	72 0.64	0.54 0.4	2 0.28	0.14 0.00	-0.15 -	0.29 -0.42	-0.54 4	### ###	* ### ##	*** **** **	## -0.79	-0.72 -0	.64 -0.54	-0.42 -0.	28 -0.29
40	909.03 213.	396 15.	20853	540 220	.5 40909.0277	213.396316	5 15.99	1 20853640	220.5	5 5	derived dispacement to wind	0. derived dispa	acement to w	nd speed at c	circumferentia	ial point on	0.09	9 0.22 0	0.35 0.47	7 0.57	0.66 0.	.72 0.77	0.79 0.7	78 0.75	0.70 0.	53 0.54	0.43 0.3	1 0.18	0.05 -0.09	-0.22 -	0.35 -0.47	-0.57 #	### ###	* ### ##	# ### #!	## -0.70	-0.63 -0	.54 -0.43	-0.31 -0.	18 -0.35
40	909.03 210.	419 14.	2 20854	140 219.4	40909.0347	2 210.418884	3 14.72	5 20854140	219.424	1 5	speed relationships for power	0 speed relation	onships for po	ver windward v	wind side top	op of GC	0.14	4 0.27 0	0.40 0.52	2 0.62	0.70 0.	.76 0.80	0.82 0.8	80 0.77	0.71 0.	53 0.53	0.41 0.2	8 0.15	0.01 -0.14	-0.27 -	0.40 -0.52	-0.62 #	### ###	* ### ##	# ### #!	## -0.71	-0.63 -0	.53 -0.41	-0.28 -0.	15 -0.40
40	909.04 215.	797 17.	20854	532 220	.5 40909.0416	215.796905	5 17.21	2 20854632	220.5	5 4	generation events	0. generation e	events	-			0.06	5 0.19 0	0.31 0.43	3 0.53	0.62 0.	.68 0.73	0.76 0.7	76 0.74	0.69 0.	53 0.54	0.44 0.3	3 0.21	0.08 -0.06	-0.19 -	0.31 -0.43	-0.53 #	### ###	* ### ##	# ### #!	## -0.69	-0.63 -0	.54 -0.44	-0.33 -0.	21 -0.31
40	909.05 218.	608 17.	20855	132 220	.5 40909.0486	218.608032	2 17.92	8 20855132	220.5	5 5	00 L	0															0.:	5 0.24	0.11 -0.02	-0.15 -	0.27 -0.39	-0.49 #	### ###	* ### ##	# ### #!	## -0.69	-0.63 -0	.56 -0.46	-0.36 -0.	24 -0.27
40	909.06 215.	773 13.	20855	532 220	.5 40909.0555	215.772827	1 13.36	8 20855632	220.5	5 5	00	0.845517782	0.1113991	89	V	/ariati	on of \	Vertica	l Displa	aceme	ent Ar	round	Circur	mfere	nce of		0.3	7 0.23	0.09 -0.06	-0.21 -	0.35 -0.48	-0.59 4	### ###	* ### ##	# ### #!	## -0.77	-0.70 -0	.61 -0.49	-0.37 -0.	23 -0.35
40	909.06 212.	593 14.	20856	132 213	.2 40909.062	212.593261	7 14.36	2 20856132	213.2	2 5	00	0.823261102	0.1196791	71					Groute	ed Co	nnect	tion					0.3	2 0.18	0.04 -0.11	-0.25 -	0.38 -0.50	-0.61	### ###	* ### ##	*** **** **	## -0.73	-0.65 -0	.56 -0.44	-0.32 -0.	18 -0.38
40	909.07 212.	053 14.	20856	502 218	.4 40909.0694	14 212.05253	6 14.60	5 20856602	218.4	4 4	70	0.817798333	0.1217114	43					Gioute	cu cu	meet	lion					0.3	1 0.17	0.03 -0.11	-0.25 -	0.38 -0.50	-0.61	### ###	* ### ##	*** **** **	## -0.72	-0.64 -0	.55 -0.43	-0.31 -0.	17 -0.38
40	909.08 210.	524 15.	19 20857	104 211	.1 40909.0763	210.524475	1 15.49	1 20857104	211.1	L 51	02	0.797962792	0.1290907	07	1								1				0.2	8 0.15	0.01 -0.13	-0.27 -	0.39 -0.51	-0.61	### ###	* ### ##	*** **** **	## -0.69	-0.62 -0	.52 -0.41	-0.28 -0.	15 -0.39
40	909.08 208.	228 16.	20857	504 211	.1 40909.0833	3 208.228408	8 16.08	5 20857604	211.1	L 51	00	0.784651022	0.1340429	82	0.8									-	VD as S3		0.2	5 0.11 -	0.02 -0.16	-0.29 -	0.41 -0.52	-0.62 4	### ###	* ### ##	# ### #!	## -0.67	-0.59 -0	.49 -0.37	-0.25 -0.	1 -0.41
40	909.09 208.	266 17.	20858	104 211	.1 40909.0902	208.266067	5 17.23	8 20858104	211.1	L 5	00	0.758838308	0.1436458	88							•	• ·	Ť (	• • •	Cosine Re	lationship	0.2	4 0.11 -	0.02 -0.15	-0.28 -	0.40 -0.51	-0.60 #	### ###	* ### ##	# ### #!	## -0.65	-0.57 -0	.47 -0.36	-0.24 -0.	-0.40
4	0909.1 221.	.382 16	.9 20858	504 2	40909.0972	221.382080	1 16.	9 20858604	223	3 5	00	0.766400009	0.1408327	67 0	0.6						•			•			0.4	0 0.28	0.15 0.02	-0.11 -	0.24 -0.37	-0.48 #	### ###	* ### ##	# ### #!	## -0.73	-0.67 -0	.60 -0.51	-0.40 -0.	28 -0.24
4	0909.1 21	7.47 15.	20859	104 223	.7 40909.1041	.7 217.469757	1 15.3	4 20859104	223.7	5	00	0.80135472	0.1278288	34 E (	0.4					•	•						0.:	7 0.24	0.10 -0.04	-0.17 -	0.31 -0.43	-0.54 4	### ###	* ### ##	# ### #!	## -0.74	-0.68 -0	.59 -0.49	-0.37 -0.	24 -0.31
40	909.11 221.	343 16.	20859	504 223	.7 40909.1111	221.343231	2 16.66	2 20859604	223.7	5	00	0.771733193	0.1388487	03 <u>E</u>	0.2					•					٠		0.4	0.28	0.15 0.02	-0.12 -	0.25 -0.37	-0.48 4	### ###	* ### ##	# ### #!	## -0.73	-0.68 -0	.60 -0.51	-0.40 -0.	28 -0.25
40	909.12 221.	.933 12.	20860	102 229	.6 40909.1180	221.932693	5 12.82	4 20860102	229.6	5 4	98	0.857712261	0.1068625	71 5	0.2					•					۲		0.4	5 0.32	0.18 0.03	-0.12 -	0.27 -0.40	-0.53 #	### ###	* ### ##	# ### #!	## -0.82	-0.76 -0	.68 -0.57	-0.45 -0.	32 -0.27
40	909.13 224.	.377 16.	20860	602 229	.6 40909.12	224.377151	5 16.31	4 20860602	229.6	5 5	00	0.779516549	0.1359531	21 Ê	0					•							0.4	4 0.32	0.19 0.06	-0.08 -	0.21 -0.34	-0.45	### ###	* ### ##	# ### #!	## -0.75	-0.70 -0	.63 -0.55	-0.44 -0.	32 -0.21
40	909.13 2	27.1 16.	20861	102 229	.6 40909.1319	227.100006	1 16.68	1 20861102	229.6	5 5	00	0.771311758	0.1390054	86 26-0	0.2				•							•	0.4	7 0.35	0.23 0.10	-0.04 -	0.17 -0.30	-0.42	### ###	* ### ##	# ### #!	## -0.75	-0.71 -0	.65 -0.57	-0.47 -0.	35 -0.17
40	909.14 222.	253 17.	20861	502 2	40909.1388	222.25265	5 17.63	1 20861602	224	1 5	00	0.750017855	0.1469272	94 14					•							•	0.4	0 0.28	0.16 0.03	-0.10 -	0.23 -0.35	-0.46	### ###	* ### ##	# ### #!	## -0.71	-0.66 -0	.59 -0.50	-0.40 -0.1	28 -0.23
40	909.15 224.	608 15.	20862	102 230	.3 40909.1458	33 224.607955	9 15.39	1 20862102	230.3	3 5	00	0.80020263	0.1282574	37 <b>č</b> -0	0.4				•								0.4	5 0.33	0.20 0.06	-0.08 -	0.21 -0.34	-0.46	### ###	* ### ##	# ### #!	## -0.77	-0.72 -0	.65 -0.56	-0.45 -0.	33 -0.21
40	909.15 220.	121 12.	20862	602 223	.3 40909.1527	220.121032	7 12.63	5 20862602	223.3	3 5	00	0.861926318	0.1052948	47 -0	0.6	<u> </u>		•									0.4	3 0.30	0.15 0.00	-0.15 -	0.29 -0.43	-0.55 4	### ###	* ### ##	# ### #!	## -0.81	-0.75 -0	.66 -0.56	-0.43 -0.	30 -0.29
40	909.16 224.	128 17.	20863	102 229	.3 40909.1597	2 224.12841	8 17.85	4 20863102	229.3	3 5	00	0.74503749	0.14878	01	<b>^</b> •	* • .		. • ·									0.4	2 0.30	0.18 0.05	-0.08 -	0.20 -0.33	-0.44	### ###	* ### ##	######	## -0.72	-0.67 -0	.60 -0.52	-0.42 -0.	30 -0.20
40	909.17 217.	.536 16	.3 20863	602 223	.3 40909.1666	217.535827	6 16.29	6 20863602	223.3	3 5	00	0.779924056	0.1358015	19 -	0.0		• • • `										0.3	6 0.23	0.10 -0.03	-0.17 -	0.30 -0.42	-0.53	### ###	* ### ##	*** ***	## -0.72	-0.66 -0	.58 -0.48	-0.36 -0.	23 -0.30
40	909.17 223.	301 17.	20864	102 223	.3 40909.1736	223.30070	5 17.78	6 20864102	223.3	3 5	00	0.746557717	0.1482145	42	-1					1			1	1		1	0.4	1 0.30	0.17 0.04	-0.09 -	0.21 -0.34	-0.45	### ###	* ### ##	*** ***	## -0.72	-0.67 -0	.60 -0.51	-0.41 -0.	30 -0.21
40	909.18 227.	538 20.	20864	502 229	.3 40909.1805	6 227.53767	4 20.15	3 20864602	229.3	3 5	00	0.693525924	0.1679435	53	0	30	60	90 1	120 15	50 1	180 2	210 2	40 27	70 3	00 3	30 3	50 0.4	2 0.32	0.21 0.09	-0.03 -	0.15 -0.26	-0.37 4	### ###	* ### ##	# ### #!	## -0.68	-0.64 -0	.59 -0.51	-0.42 -0.	32 -0.15
40	909.19 274.	852 13.	20865	100 267	.4 40909.187	274.85241	7 13.81	9 20865100	267.4	1 4	98	0.835419116	0.1151561	18						Direct	tion (°)						0.8	3 0.81	0.76 0.68	0.59	0.48 0.35	0.21 (	0.07 ###	* ### ##	# ### #!	## -0.69	-0.76 -0	.81 -0.83	-0.83 -0.1	31 0.48
40	909.19 262.	959 17.	20865	542 265	.3 40909.1944	262.958618	2 17.11	4 20865542	265.3	3 4	12	0.761605795	0.1426163	21 0.	.761605795	5 -0.t	61 -0.52	2 -0.41 -0	0.30 -0.17	/ -0.04	0.09 0.	.22 0.35	0.46 0.5	56 0.64	0.70 0.	/4 0.76	0.76 0.7	3 0.68	0.61 0.52	0.41	0.30 0.17	0.04	### ###	* ### ##	# ### #!	## -0.70	-0.74 -0	.76 -0.76	-0.73 -0.7	0.30 الأذ
4	0909.2 247.	747 13.	20865	962 2	40909.2013	247.747436	5 13.35	3 20865962	252	2 4	20	0.845862484	0.1112709	52 0.	.845862484	4 -0.5	52 -0.39	9 -0.26 -0	0.11 0.03	3 0.18	0.32 0.	.45 0.57	0.67 0.7	75 0.81	0.84 0.	85 0.83	0.78 0.7	2 0.63	0.52 0.39	0.26	0.11 -0.03	-0.18	###   ###	* ### ##	# ### #	##   -0.84	-0.85 -0	1.83 -0.78	-0.72 -0.4	0.11 3

				Con	npressive stre	ess assun	ning cosin	e distirbution	(MPa)																-
																									Chosen
	MicroStrain																								Radial
MicroStrain (power)	(non-power)	Windward Compressive Stresss (MPa)	S	1			S2			S3			S4				S5				S6				position
IF(V>14,(-0.01463008* (V^3) + 1.35768828 * (V^2)-																									
41.37958554 * (V) + 493.05198902), (-0.00031752*		IF(Power>0,((3*PI()*DY5*210000*0.981*1.64)/(((Inputs!J\$4)/2+Inputs!J\$6)*(((Inputs!\$J\$4/2)*(Inputs!\$J\$5/	(^2)																						
(V^6) + 0.0172742* (V^5) - 0.3359363* (V^4) +		*(PI()+3*Inputs!\$J\$10))+(3*PI()*Inputs!\$J\$10*((Inputs!\$J\$4/2)^2)*Inputs!\$J\$5))))/1000000,((3*PI()*DZ5*21	100																						
2.68056512* (V^3) - 6.6326729* (V^2) +	IF(V<4,	00*0.981*1.64)/(((Inputs!J\$4)/2+Inputs!J\$6)*(((Inputs!\$J\$4/2)*(Inputs!\$J\$5^2)*(PI()+3*Inputs!\$J\$10))+(3*I	PI()																						
6.74351759* (V)))	12.5*V, 21)	*Inputs!\$J\$10*((Inputs!\$J\$4/2)^2)*Inputs!\$J\$5))))/1000000)	v	/indward stress*C	COS(RADIANS	(orientati	ion-wind o	direction))																	
131.0341	21.0000		453 (	0.007 0.015 0.02	22 0.029 0.0	34 0.03	9 0.042	0.045 0.045	0.045 0.043	0.039 0.0	35 0.029 0.02	3 0.016 0.00	8 4E-04 -0.0	1 -0.02 -0.	02 -0.03 -0.0	03 -0.04	-0.04 -0.	04 -0.05	-0.04 -0.0	04 -0.04	-0.03 -0	0.03 -0.02	-0.02 -(	).01 -(	J -0.0288
142.5738	3 21.0000	0.30	073	0.01 0.041 0.09	0.143 0.1	.88 0.22	7 0.26	0.284 0.3	0.307 0.304	0.293 0.2	72 0.243 0.20	07 0.164 0.11	6 0.065 0.01	2 -0.04 -0.	09 -0.14 -0.1	-0.23	-0.26 -0.	28 -0.3	-0.31 -0	.3 -0.29	-0.27 -0	).24 -0.21	-0.16 -0	).12 -0.07	/ -0.143
111.9799	21.0000	0.24	413	0.05 -0.01 0.02	29 0.071 0.	11 0.14	5 0.177	0.202 0.222	0.235 0.241	0.24 0.2	31 0.215 0.19	3 0.165 0.13	1 0.094 0.05	4 0.013 -0.	03 -0.07 -0.	11 -0.15	-0.18 -0	0.2 -0.22	-0.24 -0.2	24 -0.24	-0.23 -0	J.22 -0.19	-0.16 -0	J.13 -0.09	-0.0706
p15470: p15470:		p15470: 0.30	089	0.05 0.001 0.05	55 0.107 0.1	55 0.19	9 0.237	0.268 0.291	0.304 0.309	0.304 0	29 0.267 0.23	6 0.198 0.15	4 0.105 0.05	3 -0 -0.	05 -0.11 -0.	16 -0.2	-0.24 -0.	27 -0.29	-0.3 -0.3	31 -0.3	-0.29 -0	J.27 -0.24	-0.2 -0	J.15 -0.1	-0.1066
Strain 1.5m above top of MP on strain 1.5m Strain 1.5m	above top of M	P on Normal compressive stress at the top of the GC 0.25	558	0.02 0.029 0.07	73 0.115 0.1	.53 0.18	6 0.214	0.235 0.249	0.255 0.254	0.245 0.2	29 0.205 0.17	6 0.141 0.10	2 0.059 0.01	5 -0.03 -0.	07 -0.11 -0.	15 -0.19	-0.21 -0.	23 -0.25	-0.26 -0.2	25 -0.25	-0.23 -0	).21 -0.18	-0.14	-0.1 -0.0€	0.1146- ز
micorstrain realtionship derived for	de based on wind	d speed to based on strain to stress transfer junction 0.28	832	-0 0.047 0.09	95 0.14 0.	18 0.21	6 0.244	0.265 0.279	0.283 0.279	0.267 0.2	46 0.218 0.18	4 0.143 0.09	9 0.051 0.00	2 -0.05 -0.	09 -0.14 -0.1	18 -0.22	-0.24 -0.	27 -0.28	-0.28 -0.2	28 -0.27	-0.25 -0	0.22 -0.18	-0.14	-0.1 -0.05	0.1398 ز
power production events power production	uction events	0.2	337	0.02 0.017 0.05	57 0.096 0.1	.31 0.16	3 0.19	0.21 0.225	0.233 0.233	0.227 0.2	13 0.193 0.16	8 0.137 0.10	2 0.064 0.02	4 -0.02 -0.	06 -0.1 -0.	13 -0.16	-0.19 -0.	21 -0.22	-0.23 -0.2	23 -0.23	-0.21 -0	).19 -0.17	-0.14	-0.1 -0.0€	i -0.0958 د
		0.22	226	0.03 0.005 0.04	4 0.081 0.1	.16 0.14	7 0.174	0.195 0.211	0.22 0.223	0.218 0.2	07 0.19 0.16	67 0.139 0.10	7 0.071 0.03	3 -0.01 -0.	04 -0.08 -0.	12 -0.15	-0.17 -0	0.2 -0.21	-0.22 -0.2	22 -0.22	-0.21 -0	).19 -0.17	-0.14 -(	).11 -0.07	/ -0.0812
142.8013	3 21.0000	0.30	078	0.03 0.023 0.07	76 0.126 0.1	73 0.21	5 0.25	0.277 0.296	0.306 0.307	0.298 0.2	81 0.254 0.22	1 0.18 0.13	4 0.084 0.03	1 -0.02 -0.	08 -0.13 -0.	17 -0.21	-0.25 -0.	28 -0.3	-0.31 -0.3	31 -0.3	-0.28 -0	).25 -0.22	-0.18 -0	J.13 -0.08	3 -0.1263
135.4695	5 21.0000	0.25	920	0.01 0.038 0.08	37 0.134 0.1	.77 0.21	5 0.246	0.27 0.285	0.292 0.29	0.279 0.2	59 0.232 0.19	8 0.157 0.11	2 0.064 0.01	3 -0.04 -0.	09 -0.13 -0.	18 -0.21	-0.25 -0.	27 -0.28	-0.29 -0.2	29 -0.28	-0.26 -0	J.23 -0.2	-0.16 -0	J.11 -0.0€	i -0.1344
132.7239	21.0000	0.28	860	0.01 0.04 0.08	38 0.134 0.1	76 0.21	2 0.242	0.265 0.28	0.286 0.283	0.272 0.2	53 0.226 0.19	0.152 0.10	0.06 0.0	1 -0.04 -0.	09 -0.13 -0.1	18 -0.21	-0.24 -0.	27 -0.28	-0.29 -0.2	28 -0.27	-0.25 -0	J.23 -0.19	-0.15 -0	J.11 -0.0E	i -0.1341
123.4615	5 21.0000	0.26	661	-0 0.044 0.08	39 0.131 0.1	69 0.20	2 0.229	0.249 0.262	0.266 0.262	0.251 0.2	32 0.205 0.17	3 0.135 0.09	3 0.049 0.00	2 -0.04 -0.	09 -0.13 -0.3	L7 -0.2	-0.23 -0.	25 -0.26	-0.27 -0.2	26 -0.25	-0.23 -0	).21 -0.17	-0.14 -(	0.09 -0.05	0.1309- ز
117.8453	3 21.0000	0.25	540 0	0.008 0.052 0.09	0.134 0.1	.69 0.	2 0.224	0.241 0.251	0.254 0.249	0.236 0.2	16 0.189 0.15	0.12 0.07	9 0.036 -0.0	1 -0.05 -0.	09 -0.13 -0.	17 -0.2	-0.22 -0.	24 -0.25	-0.25 -0.2	25 -0.24	-0.22 -0	).19 -0.16	-0.12 -0	).08 -0.04	4 -0.1337
108.2503	3 21.0000	0.23	333 (	0.007 0.047 0.08	36 0.123 0.1	.55 0.18	3 0.205	0.222 0.231	0.233 0.228	0.217 0.1	.98 0.174 0.14	4 0.11 0.07	3 0.034 -0.0	1 -0.05 -0.	09 -0.12 -0.	16 -0.18	-0.21 -0.	22 -0.23	-0.23 -0.2	23 -0.22	-0.2 -0	).17 -0.14	-0.11 -0	J.07 -0.03	3 -0.1227
110.8897	7 21.0000	0.23	390	0.05 -0.01 0.03	36 0.076 0.1	.14 0.14	9 0.179	0.204 0.223	0.234 0.239	0.236 0.2	26 0.21 0.18	37 0.158 0.12	4 0.087 0.04	7 0.006 -0.	04 -0.08 -0.	11 -0.15	-0.18 -0	0.2 -0.22	-0.23 -0.2	24 -0.24	-0.23 -0	0.21 -0.19	-0.16 -(	J.12 -0.09	-0.0763
124.9684	4 21.0000	0.26	693	0.04 0.012 0.05	58 0.103 0.1	45 0.18	2 0.214	0.239 0.257	0.267 0.269	0.263 0.2	49 0.227 0.19	8 0.164 0.12	4 0.081 0.03	5 -0.01 -0.	06 -0.1 -0.1	14 -0.18	-0.21 -0.	24 -0.26	-0.27 -0.2	27 -0.26	-0.25 -0	J.23 -0.2	-0.16 -(	0.12 -0.08	3 -0.1032
112.8356	5 21.0000	0.24	432	0.05 -0.01 0.03	37 0.078 0.1	.17 0.15	2 0.183	0.208 0.227	0.238 0.243	0.24 0	.23 0.213 0.1	9 0.161 0.12	6 0.089 0.04	8 0.006 -0.	04 -0.08 -0.	12 -0.15	-0.18 -0.	21 -0.23	-0.24 -0.2	24 -0.24	-0.23 -0	).21 -0.19	-0.16 -0	J.13 -0.09	J -0.0778
142.3622	2 21.0000	0.30	068	0.06 -0.01 0.04	13 0.095 0.1	.44 0.18	9 0.228	0.26 0.285	0.3 0.307	0.304 0.2	92 0.271 0.24	2 0.205 0.16	2 0.115 0.06	3 0.01 -0.	04 -0.1 -0.1	14 -0.19	-0.23 -0.	26 -0.28	-0.3 -0.3	31 -0.3	-0.29 -0	J.27 -0.24	-0.21 -0	).16 -0.11	-0.0952
115.803	1 21.0000	0.24	496	0.06 -0.02 0.02	24 0.067 0.1	.08 0.14	5 0.178	0.206 0.227	0.242 0.249	0.248 0	.24 0.225 0.20	03 0.175 0.14	1 0.103 0.06	2 0.019 -0.	02 -0.07 -0.3	11 -0.15	-0.18 -0.	21 -0.23	-0.24 -0.2	25 -0.25	-0.24 -0	).23 -0.2	-0.17 -(	).14 -0.1	1 -0.0672
112.6792	2 21.0000	0.24	428	0.07 -0.03 0.01	L2 0.054 0.0	94 0.13	2 0.165	0.194 0.216	0.232 0.241	0.243 0.2	37 0.224 0.20	04 0.178 0.14	6 0.111 0.07	1 0.03 -0.	01 -0.05 -0.	09 -0.13	-0.17 -0.	19 -0.22	-0.23 -0.2	24 -0.24	-0.24 -0	J.22 -0.2	-0.18 -0	).15 -0.11	-0.0542
105.3443	3 21.0000	0.22	270 .	0.05 -0.01 0.03	31 0.069 0.1	.06 0.13	9 0.168	0.192 0.21	0.222 0.227	0.225 0.2	16 0.201 0.1	8 0.153 0.12	1 0.086 0.04	8 0.009 -0.	03 -0.07 -0.3	11 -0.14	-0.17 -0.	19 -0.21	-0.22 -0.2	23 -0.22	-0.22 -	0.2 -0.18	-0.15 -0	J.12 -0.09	-0.0692
124.4531	1 21.0000	0.26	682	0.07 -0.02 0.02	25 0.071 0.1	15 0.15	5 0.191	0.221 0.244	0.26 0.267	0.267 0.2	59 0.242 0.21	9 0.188 0.15	2 0.112 0.06	8 0.022 -0.	03 -0.07 -0.3	12 -0.16	-0.19 -0.	22 -0.24	-0.26 -0.2	27 -0.27	-0.26 -0	).24 -0.22	-0.19 -(	).15 -0.11	-0.0712
142.3032	2 21.0000	0.30	067	0.05 -0 0.05	3 0.104 0.1	53 0.19	7 0.235	0.265 0.288	0.302 0.307	0.302 0.2	88 0.266 0.23	5 0.198 0.15	4 0.106 0.05	4 6E-04 -0.	05 -0.1 -0.1	15 -0.2	-0.23 -0.	27 -0.29	-0.3 -0.3	31 -0.3	-0.29 -0	).27 -0.24	-0.2 -(	).15 -0.11	-0.1043
103.7837	7 21.0000	0.22	237	0.05 -0.02 0.02	23 0.061 0.0	98 0.13	1 0.161	0.185 0.204	0.217 0.223	0.222 0.2	15 0.201 0.18	31 0.156 0.12	5 0.091 0.05	5 0.016 -0.	02 -0.06 -0	.1 -0.13	-0.16 -0.	19 -0.2	-0.22 -0.2	22 -0.22	-0.22 -	0.2 -0.18	-0.16 -0	J.13 -0.09	-0.0612
115.9627	7 21.0000	0.24	499	0.03 0.011 0.05	54 0.095 0.1	.34 0.16	9 0.198	0.222 0.238	0.248 0.25	0.244 0.2	31 0.211 0.18	4 0.152 0.11	6 0.075 0.03	3 -0.01 -0.	05 -0.1 -0.1	13 -0.17	-0.2 -0.	22 -0.24	-0.25 -0.2	25 -0.24	-0.23 -0	0.21 -0.18	-0.15 -(	0.02 -0.08	3 -0.0955
104.2540	21.0000	0.22	247	0.05 -0.01 0.02	26 0.065 0.1	01 0.13	4 0.164	0.188 0.206	0.219 0.224	0.223 0.2	15 0.201 0.1	8 0.154 0.12	3 0.089 0.05	2 0.013 -0.	03 -0.06 -0	.1 -0.13	-0.16 -0.	19 -0.21	-0.22 -0.2	22 -0.22	-0.22 -	-0.2 -0.18	-0.15 -(	).12 -0.09	-0.0646
90.7966	5 21.0000	0.19	957	0.06 -0.03 0.00	08 0.042 0.0	075 0.10	5 0.132	0.155 0.174	0.187 0.194	0.196 0.1	.91 0.181 0.16	5 0.144 0.11	9 0.09 0.05	9 0.026 -0.	01 -0.04 -0.0	07 -0.11	-0.13 -0.	16 -0.17	-0.19 -0.1	19 -0.2	-0.19 -0	).18 -0.17	-0.14 -(	).12 -0.09	<i>)</i> -0.0422
143.6984	4 21.0000	0.30	097	0.28 -0.25 -0.2	22 -0.18 -0.	13 -0.0	8 -0.03	0.028 0.081	0.132 0.178	0.22 0.2	54 0.281 0.29	9 0.309 0.30	8 0.299 0.2	8 0.253 0.2	18 0.177 0.	13 0.079	0.026 -0.	03 -0.08	-0.13 -0.1	18 -0.22	-0.25 -0	J.28 -0.3	-0.31 -(	J.31 -0.3	3 0.17698
109.2002	2 21.0000	0.23	353	0.19 -0.16 -0.1	L3 -0.09 -0.	.05 -0.0	1 0.029	0.069 0.107	0.142 0.172	0.197 0.2	17 0.229 0.23	35 0.234 0.22	5 0.21 0.18	8 0.16 0.1	28 0.092 0.0	53 0.012	-0.03 -0.	07 -0.11	-0.14 -0.3	17 -0.2	-0.22 -0	).23 -0.24	-0.23 -f	).23 -0.21	0.0918
142.7808	3 21.0000	0.30	077	0.19 -0.14 -0.0	09 -0.04 0.0	0.06	5 0.117	0.164 0.207	0.243 0.272	0.293 0.3	05 0.307 0.30	01 0.285 0.2	6 0.228 0.18	8 0.143 0.0	94 0.041 -0.	01 -0.07	-0.12 -0.	16 -0.21	-0.24 -0.2	27 -0.29	-0.3 -0	).31 -0.3	-0.28 -f	J.26 -0.2?	3 0.04148
135.3365	5 21.0000	0.25	917	-0.2 -0.15 -0.1	L1 -0.06 -0.	01 0.0	4 0.09	0.137 0.179	0.217 0.247	0.27 0.2	85 0.291 0.28	9 0.277 0.25	8 0.23 0.19	5 0.155 0.1	09 0.061 0.0	01 -0.04	-0.09 -0.	14 -0.18	-0.22 -0.2	25 -0.27	-0.29 -0	.29 -0.29	-0.28 -1	).26 -0.23	3 0.06084

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Wet corroded Wear rate for given stress (Linear), wind direction (mm/mm),																																	
wet corro	ded wear rate for g	iven stress (Linear),	wind direction (mm/	'mm),																													Chose
																																с г	Dodial
Marcine al	64			62					<b>C</b> 2						C 4						<b>CF</b>						66					n.	(duidi
wind	51			52					55						54						35						50					p	JOSILIO
IF(σ>0,0.0	00001*σ,0.00000016	5*σ)																															
4.53E-08	7:48E-09 1.51E-0	8 2.23E-08 2.88E-	08 3.44E-08 3.9E	-08 4.24E-08	8 4.45E-08	4.53E-08	4.46E-08	4.27E-08 3.94E-08	3.49E-08	2.94E-08	2.3E-08	1.58E-08	8.24E-09	3.89E-10	-1.2E-09	-2.4E-09	-3.6E-09	-4.6E-09	-5.5E-09	-6.2E-09	-6.8E-09	-7.1E-09	-7.2E-09	-7.1E-09	-6.8E-09	-6.3E-09	-5.6E-09	-4.7E-09	-3.7E-09	-2.5E-09	-1.3E-09	-6.2E-11	-5E-0
4.53E-08	-1.9E-p15470;	E-	07 1.88E-07 2.27E	-07 2.6E-0	7 2.84E-07	3E-07	3.07E-07	3.04E-07 2.93E-07	7 2.72E-07	2.43E-07	2.07E-07	1.64E-07	1.16E-07	6.53E-08	1.21E-08	-6.6E-09	-1.5E-08	-2.3E-08	-3E-08	-3.6E-08	-4.2E-08	-4.5E-08	-4.8E-08	-4.9E-08	-4.9E-08	-4.7E-08	-4.4E-08	-3.9E-08	-3.3E-08	-2.6E-08	-1.9E-08	-1E-08	-2E-0
4.83E-07	-8.7E- Wear based	on experimental	08 1.1E-07 1.45E	-07 1.77E-0	7 2.02E-07	2.22E-07	2.35E-07	2.41E-07 2.4E-07	7 2.31E-07	2.15E-07	1.93E-07	1.65E-07	1.31E-07	9.43E-08	5.43E-08	1.26E-08	-4.7E-09	-1.1E-08	-1.8E-08	-2.3E-08	-2.8E-08	-3.2E-08	-3.6E-08	-3.8E-08	-3.9E-08	-3.8E-08	-3.7E-08	-3.4E-08	-3.1E-08	-2.6E-08	-2.1E-08	-1.5E-08	-1E-(
6.18E-07	-8.4E- testing wear	rates graphs for a E-	07 1.55E-07 1.99E	-07 2.37E-0	7 2.68E-07	2.91E-07	3.04E-07	3.09E-07 3.04E-07	7 2.9E-07	2.67E-07	2.36E-07	1.98E-07	1.54E-07	1.05E-07	5.26E-08	-1.6E-10	-8.7E-09	-1.7E-08	-2.5E-08	-3.2E-08	-3.8E-08	-4.3E-08	-4.7E-08	-4.9E-08	-4.9E-08	-4.9E-08	-4.6E-08	-4.3E-08	-3.8E-08	-3.2E-08	-2.5E-08	-1.7E-08	-2E-(
5.12E-07	-2.4E- given stress	E-	07 1.53E-07 1.86E	-07 2.14E-0	7 2.35E-07	2.49E-07	2.55E-07	2.54E-07 2.45E-07	7 2.29E-07	2.05E-07	1.76E-07	1.41E-07	1.02E-07	5.93E-08	1.52E-08	-4.7E-09	-1.2E-08	-1.8E-08	-2.4E-08	-3E-08	-3.4E-08	-3.8E-08	-4E-08	-4.1E-08	-4.1E-08	-3.9E-08	-3.7E-08	-3.3E-08	-2.8E-08	-2.3E-08	-1.6E-08	-9.5E-09	-2E-0
5.66E-07	-3.3E-10 4.71E-0	8 9.49E-08 1.4E-	07 1.8E-07 2.16E	-07 2.44E-0	7 2.65E-07	2.79E-07	2.83E-07	2.79E-07 2.67E-07	7 2.46E-07	2.18E-07	1.84E-07	1.43E-07	9.88E-08	5.12E-08	2.07E-09	-7.5E-09	-1.5E-08	-2.2E-08	-2.9E-08	-3.4E-08	-3.9E-08	-4.2E-08	-4.5E-08	-4.5E-08	-4.5E-08	-4.3E-08	-3.9E-08	-3.5E-08	-2.9E-08	-2.3E-08	-1.6E-08	-8.2E-09	-2E-0
4.67E-07	-3.8E-09 1.71E-0	8 5.73E-08 9.58E-	08 1.31E-07 1.63E	-07 1.9E-0	7 2.1E-07	2.25E-07	2.33E-07	2.33E-07 2.27E-07	7 2.13E-07	1.93E-07	1.68E-07	1.37E-07	1.02E-07	6.36E-08	2.36E-08	-2.7E-09	-9.2E-09	-1.5E-08	-2.1E-08	-2.6E-08	-3E-08	-3.4E-08	-3.6E-08	-3.7E-08	-3.7E-08	-3.6E-08	-3.4E-08	-3.1E-08	-2.7E-08	-2.2E-08	-1.6E-08	-1E-08	-2E-0
4.45E-07									E-07	1.9E-07	1.67E-07	1.39E-07	1.07E-07	7.1E-08	3.33E-08	-8.7E-10	-7E-09	-1.3E-08	-1.9E-08	-2.4E-08	-2.8E-08	-3.1E-08	-3.4E-08	-3.5E-08	-3.6E-08	-3.5E-08	-3.3E-08	-3E-08	-2.7E-08	-2.2E-08	-1.7E-08	-1.1E-08	-1E-(
6.16E-07		Distruibut	ion of Wear I	Rate at t	the Top	of the (	Groute	ed	E-07	2.54E-07	2.21E-07	1.8E-07	1.34E-07	8.37E-08	3.1E-08	-3.6E-09	-1.2E-08	-2E-08	-2.8E-08	-3.4E-08	-4E-08	-4.4E-08	-4.7E-08	-4.9E-08	-4.9E-08	-4.8E-08	-4.5E-08	-4.1E-08	-3.5E-08	-2.9E-08	-2.1E-08	-1.3E-08	-2E-0
5.84E-07		0	onnoction fo	r 240° M	Vind Di	raction			E-07	2.32E-07	1.98E-07	1.57E-07	1.12E-07	6.37E-08	1.32E-08	-6E-09	-1.4E-08	-2.2E-08	-2.8E-08	-3.4E-08	-3.9E-08	-4.3E-08	-4.6E-08	-4.7E-08	-4.6E-08	-4.5E-08	-4.1E-08	-3.7E-08	-3.2E-08	-2.5E-08	-1.8E-08	-1E-08	-2E-0
5.72E-07			onnection to	1240 0		rection			E-07	2.26E-07	1.92E-07	1.52E-07	1.07E-07	5.97E-08	1.02E-08	-6.3E-09	-1.4E-08	-2.1E-08	-2.8E-08	-3.4E-08	-3.9E-08	-4.2E-08	-4.5E-08	-4.6E-08	-4.5E-08	-4.4E-08	-4E-08	-3.6E-08	-3.1E-08	-2.4E-08	-1.7E-08	-9.6E-09	-2E-0
5.32E-07	3.5E-07								E-07	2.05E-07	1.73E-07	1.35E-07	9.33E-08	4.86E-08	2.44E-09	-7E-09	-1.4E-08	-2.1E-08	-2.7E-08	-3.2E-08	-3.7E-08	-4E-08	-4.2E-08	-4.3E-08	-4.2E-08	-4E-08	-3.7E-08	-3.3E-08	-2.8E-08	-2.2E-08	-1.5E-08	-7.8E-09	-2E-0
5.08E-07									E-07	1.89E-07	1.57E-07	1.2E-07	7.94E-08	3.63E-08	-1.3E-09	-8.3E-09	-1.5E-08	-2.1E-08	-2.7E-08	-3.2E-08	-3.6E-08	-3.9E-08	-4E-08	-4.1E-08	-4E-08	-3.8E-08	-3.5E-08	-3E-08	-2.5E-08	-1.9E-08	-1.3E-08	-5.8E-09	-2E-0
4.67E-07	0.0000003					**			E-07	1.74E-07	1.44E-07	1.1E-07	7.31E-08	3.35E-08	-1.1E-09	-7.6E-09	-1.4E-08	-2E-08	-2.5E-08	-2.9E-08	-3.3E-08	-3.5E-08	-3.7E-08	-3.7E-08	-3.7E-08	-3.5E-08	-3.2E-08	-2.8E-08	-2.3E-08	-1.8E-08	-1.2E-08	-5.4E-09	-2E-0
4.78E-07	2 5E-07				•	•			E-07	2.1E-07	1.87E-07	1.58E-07	1.24E-07	8.71E-08	4.72E-08	5.76E-09	-5.7E-09	-1.2E-08	-1.8E-08	-2.4E-08	-2.9E-08	-3.3E-08	-3.6E-08	-3.7E-08	-3.8E-08	-3.8E-08	-3.6E-08	-3.4E-08	-3E-08	-2.5E-08	-2E-08	-1.4E-08	-1E-0
5.39E-07	2.52 07			•		•			E-07	2.27E-07	1.98E-07	1.64E-07	1.24E-07	8.09E-08	3.5E-08	-1.9E-09	-9.3E-09	-1.7E-08	-2.3E-08	-2.9E-08	-3.4E-08	-3.8E-08	-4.1E-08	-4.3E-08	-4.3E-08	-4.2E-08	-4E-08	-3.6E-08	-3.2E-08	-2.6E-08	-2E-08	-1.3E-08	-2E-0
4.86F-07	0.0000002						•		F-07	2.13E-07	1.9F-07	1.61F-07	1.26F-07	8.85F-08	4.78F-08	5.7F-09	-5.9F-09	-1.2E-08	-1.9E-08	-2.4F-08	-2.9E-08	-3.3E-08	-3.6F-08	-3.8F-08	-3.9E-08	-3.8F-08	-3.7E-08	-3.4E-08	-3F-08	-2.6F-08	-2F-08	-1.4E-08	-1F-0
6 14F-07	e e						•		E-07	2 71E-07	2 42F-07	2 05E-07	1 62E-07	1 15E-07	6 34F-08	1 03E-08	-6 9E-09	-1 5E-08	-2 3E-08	-3E-08	-3 7E-08	-4 2E-08	-4 6F-08	-4 8F-08	-4 9F-08	-4 9F-08	-4 7F-08	-4 3E-08	-3 9F-08	-3 3E-08	-2 6E-08	-1 8E-08	-2F-0
4 99E-07	2 1.5E-07			+					E-07	2.71E 07	2.03E-07	1 75E-07	1.02E 07	1.03E-07	6 2E-08	1.05E-08	-3.9E-09	-1 1E-08	-1 7E-08	-2 3E-08	-2 9E-08	-3 3E-08	-3 6E-08	-3.9E-08	-4F-08	-4F-08	-3.8E-08	-3 6E-08	-3 2E-08	-2.8E-08	-2 3E-08	-1 6E-08	-1F-0
4 86E-07							•		E-07	2.23E-07	2.03E-07	1 78E-07	1.46E-07	1 11E-07	7 14E-08	3E-08	-2E-09	-8 7E-09	-1 5E-08	-2 1E-08	-2 6E-08	-3 1E-08	-3 5E-08	-3 7E-08	-3 9E-08	-3 9E-08	-3.8E-08	-3 6E-08	-3 3E-08	-2.8E-08	-2 3E-08	-1.8E-08	-9F-0
4 54E-07	3			•					E-07	2.01E-07	1.8E-07	1 53E-07	1.70E 07	8 6F-08	4.82E-08	8 92E-09	-4 9E-09	-1 1E-08	-1 7E-08	-2 2E-08	-2 7E-08	-3 1E-08	-3 4F-08	-3 5E-08	-3 6E-08	-3 6E-08	-3 5E-08	-3 2E-08	-2 9E-08	-2 4E-08	-1 9E-08	-1 4E-08	-1F-0
5 36E-07	5E-08		•					·	E-07	2 /2F-07	2 19E-07	1 88F-07	1.52E-07	1 12E-07	6 76E-08	2 15E-08	-4E-09	-1 1E-08	-1 8E-08	-2 5E-08	-3 1E-08	-3 5E-08	-3 9F-08	-4 2E-08	-4 3E-08	-4 3E-08	-/ 1E-08	-3 9E-08	-3 5E-08	-3E-08	-2 /F-08	-1 8F-08	-1E-0
6 125 07								•	E 07	2.420 07	2.150.07	1 095 07	1.520.07	1.065.07	5 205 09	6 49E 10	9 45 00	1 75 09	2 45 09	2.50 00	2 95 09	4 25 09	1 6E 00	4.22 00	4.50 00	4.5E 00	4.10 00	4 25 09	2 95 00	2 25 08	2.40 00	1.75.09	25.0
4 475 07	0	20 00		450 4			270	200 200	200 5.07	2.001-07	1.915.07	1.565.07	1.34L-07	0.14E.09	5.465.09	1 615 09	2 75 00	0.95.00	1 6E 09	2 15 09	-3.8L-08	2E 09	2 25 00	2 55 09	2 65 08	2 65 09	-4.0L-08	2 25 09	2 OF 09	-3.21-08	-2.JL-08	1 55 09	15.0
4.47L-07	-5E-08	30 60	• 90 · 120	150 1	80 210	240	270	300 330 🔷 🖕	360 5.07	2.011-07	1.010-07	1.500-07	1.251-07	7 525 09	2 295 09	1.010-00	9 6E 00	1 55.00	2 15 09	2.11-08	2.00-08	2 55 09	-3.3L-08	-3.3L-08	4E 09	2 OF 08	-3.4L-08	-3.2L-08	2.5L-08	-2.31-08	1 95 09	1 25 09	-10-0
4 405 07									E-07	2.110-07	1.841-07	1.521-07	1.100-07	0.00E 00	5.201-00	1 205 09	4 35 00	15.00	1 65 00	2.76-00	-3.2L-08	-3.JL-08	3.86-08	3 55 08	2 65 08	2 CE 00	2 45 00	2.25.00	2.92-08	2.41-08	25.00	1 45 00	10 10
4.43E-07	-1E-07			0	- 41 (9)				E-07	1 915 07	1.655.07	1 445 07	1.236-07	0.03E-00	5.1/E-00	2 575 00	-4.2E-09	-1E-00	-1.0E-08	-2.1E-00	-2.0E-08	- JE-00	-3.3E-08	-3.32-08	2 15 08	2 15 00	-3.4E-08	-3.22-08	-2.3E-00	-2.JE-00	1 05 09	1 45 00	-10-0
5.91E-07				Orient	ation (*)				E-07	1.01E-U/	1.03E-07	1.44E-U/	1.195-07	3.03E-08	3.95-08	2.3/E-08	-1.5E-09	-0.6E-09	1.2E-08	-1./E-U8	-2.1E-08	-2.3E-08	-2.0E-U8	- 3E-U8	-3.1E-06	-3.1E-08	-3.1E-08	-2.9E-U8	-2.0E-08	-2.3E-00	-1.9E-00	-1.4E-00	-/=-U
0.19E-07	25.00 2.05.0	0 25 00 4 55	00 0 45 00 1 05	00 2.005 0	0 05 00	1.075.07	1 425 07	1 735 07 1 075 0	E-07	2.81E-07	2.991-07	5.09E-07	5.U8E-07	2.991-07	2.8E-07	2.53E-07	2.18E-U/	1.//E-0/	1.3E-07	1.94E-08	2.02E-U8	-4.4E-09	-1.3E-08	-2.1E-U8	-2.9E-U8	-3.5E-U8	-4.1E-08	-4.5E-U8	-4.8E-U8	-4.9E-U8	-4.9E-U8	-4.8E-U8	2E-U
4./1E-0/	-3E-U8 -2.6E-U	8 -2E-U8 -1.5E-	U8 -8.4E-U9 -1.9E	-U9 2.88E-U	8 0.9E-08	1.U/E-U/	1.42E-07	1.72E-07 1.97E-0.	/ 2.1/E-0/	2.29E-07	2.35E-07	2.34E-07	2.25E-07	2.1E-07	1.88E-07	1.0E-07	1.28E-07	9.18E-08	5.28E-08	1.21E-08	-4.0E-09	-1.1E-08	-1./E-08	-2.3E-08	-2.8E-08	-3.2E-08	-3.5E-08	-3./E-08	-3.8E-08	-3./E-08	-3.0E-08	-3.4E-08	9E-0
0.15E-07	-3E-U8 -2.3E-0	8 -1.5E-U8 -6.6E-	U9 1.21E-U8 6.53E	-U8 1.1/E-0	/ 1.64E-07	2.0/E-07	2.43E-07	2.72E-07 2.93E-0	/ 3.05E-07	3.0/E-07	3.01E-07	2.85E-07	2.6E-07	2.28E-07	1.88E-07	1.43E-07	9.38E-08	4.15E-08	-1.9E-09	-1E-08	-1.9E-08	-2.6E-08	-3.3E-08	-3.9E-08	-4.4E-08	-4./E-08	-4.9E-08	-4.9E-08	-4.8E-08	-4.6E-08	-4.2E-08	-3.6E-08	4E-0
5.83E-07	-3.1E-08 -2.5E-0	8 -1.8E-08 -9.7E-	09 -1.7E-09 4.04E	-U8 8.99E-0	8 1.37E-07	1.79E-07	2.17E-07	2.4/E-07 2.7E-07	/ 2.85E-07	2.91E-07	2.89E-07	2.77E-07	2.58E-07	2.3E-07	1.95E-07	1.55E-07	1.09E-07	ь.08E-08	1.04E-08	-6.5E-09	-1.4E-08	-2.2E-08	-2.9E-08	-3.5E-08	-4E-08	-4.3E-08	-4.6E-08	-4.7E-08	-4.6E-08	-4.4E-08	-4.1E-08	-3.7E-08	6E-0

المعاملة والمعالم																																				
Vertical dis	blacemen	difference																																		
S1						S2					S	53						S4					S	5					S	5						Chosen Direction
0.05	0.07	0.20	0.32	0.43	0.52	0.60	0.67	0.71	0.73	0.73	0.70	0.66	0.59	0.51	0.41	0.30	0.18	0.05	0.07	0.20	0.32	0.43	0.52	0.60	0.67	0.71	0.73	0.73	0.70	0.66	0.59	0.51	0.41	0.30	0.18	0.32
0.14	0.15	0.17	0.17	0.17	0.17	0.16	0.14	0.12	0.10	0.07	0.05	0.02	0.01	0.04	0.07	0.10	0.12	0.14	0.15	0.17	0.17	0.17	0.17	0.16	0.14	0.12	0.10	0.07	0.05	0.02	0.01	0.04	0.07	0.10	0.12	0.17
0.03	0.04	0.05	0.06	0.07	0.08	0.08	0.08	0.08	0.08	0.07	0.06	0.05	0.04	0.03	0.01	0.00	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.08	0.08	0.08	0.08	0.07	0.06	0.05	0.04	0.03	0.01	0.00	0.02	0.06
0.10	0.09	0.08	0.06	0.05	0.03	0.01	0.01	0.02	0.04	0.06	0.07	0.08	0.09	0.10	0.10	0.10	0.10	0.10	0.09	0.08	0.06	0.05	0.03	0.01	0.01	0.02	0.04	0.06	0.07	0.08	0.09	0.10	0.10	0.10	0.10	0.06
0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.04	0.04	0.03	0.02	0.01	0.01	0.00	0.01	0.02	0.03	0.03	0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.04	0.04	0.03	0.02	0.01	0.01	0.00	0.01	0.02	0.03	0.03	0.05
0.07	0.08	0.09	0.09	0.09	0.09	0.09	0.08	0.07	0.06	0.05	0.03	0.02	0.00	0.02	0.03	0.05	0.06	0.07	0.08	0.09	0.09	0.09	0.09	0.09	0.08	0.07	0.06	0.05	0.03	0.02	0.00	0.02	0.03	0.05	0.06	0.09
0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.03	0.03	0.02	0.01	0.01	0.00	0.01	0.01	0.02	0.03	0.03	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.03	0.03	0.02	0.01	0.01	0.00	0.01	0.01	0.02	0.03	0.03	0.04
0.03	0.04	0.06	0.08	0.09	0.10	0.11	0.11	0.11	0.11	0.10	0.09	0.08	0.06	0.05	0.03	0.01	0.01	0.03	0.04	0.06	0.08	0.09	0.10	0.11	0.11	0.11	0.11	0.10	0.09	0.08	0.06	0.05	0.03	0.01	0.01	0.08
0.05	0.04	0.04	0.03	0.02	0.02	0.01	0.00	0.01	0.02	0.03	0.03	0.04	0.05	0.05	0.05	0.05	0.05	0.05	0.04	0.04	0.03	0.02	0.02	0.01	0.00	0.01	0.02	0.03	0.03	0.04	0.05	0.05	0.05	0.05	0.05	0.03
0.01	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.00
0.02	0.02	0.01	0.01	0.00	0.00	0.01	0.01	0.02	0.02	0.02	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.02	0.02	0.01	0.01	0.00	0.00	0.01	0.01	0.02	0.02	0.02	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.01
0.03	0.03	0.02	0.02	0.02	0.01	0.00	0.00	0.01	0.01	0.02	0.02	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.03	0.02	0.02	0.02	0.01	0.00	0.00	0.01	0.01	0.02	0.02	0.03	0.03	0.03	0.03	0.03	0.03	0.02
0.00	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.03	0.03	0.03	0.02	0.02	0.02	0.02	0.01	0.01	0.00	0.00	0.01	0.01	0.01	0.02	0.02	0.02	0.02	0.03	0.03	0.03	0.02	0.02	0.02	0.02	0.01	0.01	0.00	0.01
0.17	0.17	0.17	0.15	0.14	0.12	0.09	0.07	0.04	0.01	0.02	0.05	0.08	0.11	0.13	0.15	0.16	0.17	0.17	0.17	0.17	0.15	0.14	0.12	0.09	0.07	0.04	0.01	0.02	0.05	0.08	0.11	0.13	0.15	0.16	0.17	0.15
0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.05	0.04	0.03	0.02	0.01	0.00	0.01	0.02	0.03	0.04	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.06	0.05	0.04	0.03	0.02	0.01	0.00	0.01	0.02	0.03	0.04	0.06
0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.05	0.05	0.04	0.03	0.02	0.01	0.00	0.01	0.02	0.03	0.04	0.05	0.05	0.06	0.06	0.06	0.06	0.06	0.05	0.05	0.04	0.03	0.02	0.01	0.00	0.01	0.02	0.03	0.04	0.06
0.03	0.01	0.00	0.02	0.03	0.05	0.06	0.07	0.08	0.08	0.09	0.09	0.08	0.08	0.07	0.06	0.05	0.04	0.03	0.01	0.00	0.02	0.03	0.05	0.06	0.07	0.08	0.08	0.09	0.09	0.08	0.08	0.07	0.06	0.05	0.04	0.02
0.02	0.03	0.04	0.06	0.07	0.07	0.08	0.08	0.09	0.08	0.08	0.07	0.06	0.05	0.04	0.03	0.01	0.00	0.02	0.03	0.04	0.06	0.07	0.07	0.08	0.08	0.09	0.08	0.08	0.07	0.06	0.05	0.04	0.03	0.01	0.00	0.06
0.03	0.04	0.04	0.04	0.04	0.04	0.03	0.03	0.02	0.02	0.01	0.01	0.00	0.01	0.01	0.02	0.03	0.03	0.03	0.04	0.04	0.04	0.04	0.04	0.03	0.03	0.02	0.02	0.01	0.01	0.00	0.01	0.01	0.02	0.03	0.03	0.04
0.07	0.07	0.06	0.06	0.05	0.04	0.03	0.02	0.01	0.00	0.02	0.03	0.04	0.05	0.05	0.06	0.07	0.07	0.07	0.07	0.06	0.06	0.05	0.04	0.03	0.02	0.01	0.00	0.02	0.03	0.04	0.05	0.05	0.06	0.07	0.07	0.06
0.04	0.03	0.03	0.02	0.01	0.00	0.01	0.02	0.03	0.04	0.05	0.05	0.06	0.06	0.06	0.06	0.05	0.05	0.04	0.03	0.03	0.02	0.01	0.00	0.01	0.02	0.03	0.04	0.05	0.05	0.06	0.06	0.06	0.06	0.05	0.05	0.02
0.05	0.06	0.07	0.08	0.09	0.09	0.09	0.09	0.08	0.07	0.06	0.05	0.04	0.02	0.01	0.01	0.02	0.04	0.05	0.06	0.07	0.08	0.09	0.09	0.09	0.09	0.08	0.07	0.06	0.05	0.04	0.02	0.01	0.01	0.02	0.04	0.08
0.03	0.05	0.07	0.09	0.10	0.12	0.12	0.13	0.13	0.13	0.12	0.11	0.09	0.08	0.06	0.04	0.01	0.01	0.03	0.05	0.07	0.09	0.10	0.12	0.12	0.13	0.13	0.13	0.12	0.11	0.09	0.08	0.06	0.04	0.01	0.01	0.09
0.08	0.09	0.09	0.09	0.09	0.09	0.08	0.07	0.06	0.05	0.04	0.02	0.00	0.01	0.03	0.04	0.06	0.07	0.08	0.09	0.09	0.09	0.09	0.09	0.08	0.07	0.06	0.05	0.04	0.02	0.00	0.01	0.03	0.04	0.06	0.07	0.09
0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.07	0.06	0.05	0.03	0.02	0.01	0.01	0.02	0.04	0.05	0.06	0.07	0.08	0.08	0.08	0.08	0.08	0.08	0.07	0.06	0.05	0.03	0.02	0.01	0.01	0.02	0.04	0.05	0.06	0.08
0.04	0.05	0.06	0.06	0.07	0.07	0.08	0.07	0.07	0.07	0.06	0.05	0.04	0.03	0.01	0.00	0.01	0.03	0.04	0.05	0.06	0.06	0.07	0.07	0.08	0.07	0.07	0.07	0.06	0.05	0.04	0.03	0.01	0.00	0.01	0.03	0.06
0.55	0.59	0.62	0.63	0.62	0.59	0.54	0.47	0.40	0.31	0.21	0.10	0.01	0.12	0.22	0.32	0.41	0.49	0.55	0.59	0.62	0.63	0.62	0.59	0.54	0.47	0.40	0.31	0.21	0.10	0.01	0.12	0.22	0.32	0.41	0.49	0.63
0.15	0.16	0.17	0.18	0.18	0.17	0.16	0.15	0.13	0.10	0.08	0.05	0.02	0.02	0.05	0.08	0.10	0.13	0.15	0.16	0.17	0.18	0.18	0.17	0.16	0.15	0.13	0.10	0.08	0.05	0.02	0.02	0.05	0.08	0.10	0.13	0.18
0.09	0.13	0.16	0.18	0.20	0.22	0.23	0.23	0.22	0.21	0.19	0.17	0.14	0.10	0.07	0.03	0.01	0.05	0.09	0.13	0.16	0.18	0.20	0.22	0.23	0.23	0.22	0.21	0.19	0.17	0.14	0.10	0.07	0.03	0.01	0.05	0.18
0.03	0.04	0.05	0.06	0.06	0.07	0.07	0.07	0.06	0.06	0.05	0.04	0.03	0.02	0.01	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.06	0.07	0.07	0.07	0.06	0.06	0.05	0.04	0.03	0.02	0.01	0.00	0.01	0.02	0.06

References



80	290	300	310	320	330	340	350
80	290	<b>S4</b>	310	320	330	340	350
641	1.4612	1.2350	0.9959	0.7626	0.5503	0.3654	0.2093
462	1.2699	1.0733	0.8655	0.6627	0.4782	0.3176	0.1819
283	1.0785	0.9116	0.7351	0.5629	0.4062	0.2697	0.1545
103	0.8872	0.7498	0.6047	0.4630	0.3341	0.2219	0.1271
924	0.6958	0.5881	0.4742	0.3631	0.2620	0.1740	0.0997
745	0.5045	0.4264	0.3438	0.2633	0.1900	0.1262	0.0723
566	0.3131	0.2647	0.2134	0.1634	0.11/9	0.0783	0.0449
521	0.1409	0.1191	0.0960	0.0735	0.0531	0.0352	0.0202
464	0.1285	0.1086	0.0876	0.0671	0.0484	0.0321	0.0133
397	0.1226	0.1037	0.0836	0.0640	0.0462	0.0307	0.0176
330	0.1168	0.0987	0.0796	0.0609	0.0440	0.0292	0.0167
263	0.1109	0.0937	0.0756	0.0579	0.0417	0.0277	0.0159
195	0.1050	0.0887	0.0715	0.0548	0.0395	0.0263	0.0150
128	0.0991	0.0837	0.0675	0.0517	0.0373	0.0248	0.0142
061	0.0932	0.0788	0.0635	0.0486	0.0351	0.0233	0.0133
994	0.0873	0.0738	0.0595	0.0456	0.0329	0.0218	0.0125
927	0.0814	0.0688	0.0555	0.0425	0.0307	0.0204	0.0117
000 793	0.0755	0.0058	0.0313	0.0394	0.0284	0.0189	0.0108
726	0.0637	0.0538	0.0474	0.0303	0.0202	0.0174	0.0091
658	0.0578	0.0489	0.0394	0.0302	0.0218	0.0145	0.0083
591	0.0519	0.0439	0.0354	0.0271	0.0196	0.0130	0.0074
524	0.0460	0.0389	0.0314	0.0240	0.0173	0.0115	0.0066
457	0.0401	0.0339	0.0274	0.0209	0.0151	0.0100	0.0057
390	0.0342	0.0289	0.0233	0.0179	0.0129	0.0086	0.0049
323	0.0283	0.0240	0.0193	0.0148	0.0107	0.0071	0.0041
256	0.0225	0.0190	0.0153	0.0117	0.0085	0.0056	0.0032
189	0.0166	0.0140	0.0113	0.0086	0.0062	0.0041	0.0024
121	0.0107	0.0090	0.0073	0.0056	0.0040	0.0027	0.0015
0004	0.0048	0.0040	0.0033	0.0023	0.0018	0.0012	0.0007
001	0.0001	0.0013	0.0020	0.0028	0.0036	0.0045	0.0054
002	0.0012	0.0024	0.0037	0.0051	0.0066	0.0082	0.0098
003	0.0018	0.0035	0.0054	0.0075	0.0096	0.0120	0.0143
004	0.0024	0.0046	0.0071	0.0098	0.0127	0.0157	0.0188
005	0.0029	0.0058	0.0088	0.0121	0.0157	0.0194	0.0233
006	0.0035	0.0069	0.0105	0.0145	0.0187	0.0232	0.0278
007	0.0041	0.0080	0.0122	0.0168	0.0217	0.0269	0.0323
800	0.0046	0.0091	0.0139	0.0191	0.0248	0.0307	0.0368
009	0.0052	0.0102	0.0156	0.0215	0.0278	0.0344	0.0413
010	0.0058	0.0113	0.0175	0.0258	0.0308	0.0382	0.0458
011	0.0003	0.0124	0.0150	0.0201	0.0358	0.0413	0.0503
013	0.0074	0.0135	0.0224	0.0308	0.0399	0.0494	0.0593
014	0.0080	0.0157	0.0241	0.0331	0.0429	0.0532	0.0638
015	0.0086	0.0168	0.0258	0.0355	0.0459	0.0569	0.0683
016	0.0091	0.0179	0.0275	0.0378	0.0490	0.0607	0.0728
017	0.0097	0.0191	0.0292	0.0401	0.0520	0.0644	0.0772
018	0.0103	0.0202	0.0309	0.0425	0.0550	0.0682	0.0817
019	0.0108	0.0213	0.0326	0.0448	0.0580	0.0719	0.0862
020	0.0114	0.0224	0.0343	0.0472	0.0611	0.0756	0.0907
021	0.0120	0.0235	0.0359	0.0495	0.0641	0.0794	0.0952
022	0.0125	0.0240	0.0370	0.0518	0.0071	0.0851	0.0997
023	0.0131	0.0268	0.0333	0.0565	0.0731	0.0906	0.1042
025	0.0142	0.0279	0.0427	0.0588	0.0762	0.0944	0.1132
026	0.0148	0.0290	0.0444	0.0612	0.0792	0.0981	0.1177
026	0.0151	0.0297	0.0454	0.0626	0.0810	0.1004	0.1204
079	0.0455	0.0893	0.1367	0.1881	0.2436	0.3018	0.3620
117	0.0672	0.1318	0.2017	0.2777	0.3596	0.4455	0.5344
155	0.0888	0.1743	0.2668	0.3673	0.4756	0.5892	0.7067
193	0.1105	0.2168	0.3318	0.4568	0.5915	0.7329	0.8790
230	0.1321	0.2593	0.3969	0.5464	0.7075	0.8766	1.0514
268	0.1538	0.3018	0.4620	0.6360	0.8235	1.0203	1.2237
201	0.1046	0.3230	0.4945	0.0807	0.8815	1.0921	1.2023

# **APPENDIX B** - CSP FOUNDATION CALCULATIONS

It should be note	d that live load forces and m	oments follow the	sign convention a	as shown in	the picture,	but force	s within the	soil follow s	soil sign conventi	on of positive being (	lownwards			F.S<1 1 <f.s< F.S&gt;2</f.s< 
SCA Specs.       Trough     Abengoa       Height, W (m)     5.76       Length, L (m)     12       Design Wind Speed, U (m/s)     33	Air Density, ρ (k Dynamic Pressu Μιτ	g/m3) ire, q 11 Pile Design	1.29 S.T.P 702.405	1	Soil Prop Υ δ conc/soil Φ	18 20 30		Concrete γ⊧	25		_ 4	Wind		
Inner Field           Cfz         0.417           Cfz         0.247           Cmy         0.095           Horizontal force (kN)         20.2454           Vertical force (kN)         11.9919           Moment (kNm)         28.5687	Pile depth, Dp (m) Pile outer diameter, Do (r Wall thickness, t (m) End pile seperation, S (m	e Specs. m) 0. 0.0 1.	3.5 .13 .038 .75 1.7	_					based on AP de δ pile/soil Nq q limit f limit	I codes nse sand/silt 30 18.4 9.6 95.7	fa bio	my Wind Zopk w Ator War Defense of Continue Sprees Existing 5 (AN)	Topus San An resource for anyo	
Corner Outer Field           Cfx         2.714           Cfz         -0.014           Cmy         -0.108           Horizontal force (kN)         131.785           Vertical force (kN)         -0.6707           Moment(kNm)         -20.8428			Θv average Θv = Dp , γ / 31.5	Θvtip 2 Θv = Dp.γ 63	Shaft friction f = Ka . ⊝v . T 6.06217	, f (kN/m <sup>-</sup> ) an(δ) 7826	End bearing (kN/r q = Ov 1 115	capacity, q m2) tip . Nq 9.2	plugged Q = (π, Do <sup>2</sup> / 4).q 15.38632701	unplugged Q = (π. (Do <sup>2</sup> - (Do-2t) 1.85713473	)/4).q 2	ріцддей Unplugg F= π. Do. H. f F= π. Do. 8.865428483 17.33085	Total capacity (KN) ed Plugged Unplugg H.2f Fp+Qp Fup+Q 283 24.05175 18.9875	ged Low lup 388
Corner Fenced Outer Field	Load C	Case 1 outer field	ces and moment		Break down of	moments	1	Ho	rizontal resistance					
Cfx 1.159		H/ pile V/	/pile moment force	arm	V front	V back	Oh average		Hbear			F.S Sliding		F.S Eura-Our
Cfz         0.133           Cmy         0.029           Horizontal force (kN)         56.2897           Vertical force (kN)         0.45718	: Load C <u>ase</u>	32.9413335 -0.169	925818 -8.469380179	9	-8.639306	e 8.299454	94.5 Phav = 0.5 Kp . 6	θv	42.9975			1.305275		2.197
Moment (kNm) 8.10983		H/ pile V/	pile moment force	arm	k down of mon V front	V back	Oh average	Ho	rizontal resistance Hbear			F.S Sliding		F.S
2nd row Cfx 0.512	1	4.06743019 1.6142	295267 2.317094577		3.931389844	∈ -0.7028	94.5 Bhav = 0.5 Kp	θv	42.9975			3.056528		27.01
Cfz 0.303 Cmy 0.06096	Load C	Case 3 inner field												-
Horizontal force (kN) 24.8577 Vertical force (kN) 14.7107		For H/ pile V/	ces and moment /pile moment force	arm	Break down of V front	moments V back	Oh average	Ho	rizontal resistance Hbear			F.S Sliding		F.S
Moment (kNm) 17.0474	5	061361853 2 9979	976925 7 813731714		10 81170864	e-4 81575	Dhav = 0.5 Kp . 6	θv	42 9975			8 495243		3.942
	Load	Case 4 2nd row			10.01110004		01.0		12.0010					0.01
		For H/ pile V/	ces and moment	arm	Break down of V front	moments V back	Ah average	Ho	rizontal resistance Hbear			F.S.Sliding		F.S
Cfx is chosen to be a maximum and corresponding Cfz and Cmy for that	6	214429901 3.6776	680195 4.870692601		8 548372796	e	Dhav = 0.5 Kp . 6 94.5	θv	42,9975			8 485243		3.942
pitch and yaw are used					0.040072700		01.0		12.0010					0.01
	s	lab Design		<b>.</b>										
		Slab Specifications												
	Width Length	-	2.5	5	shape factor		0.18	0.22						
$\frac{Ka}{(a=1-\sin(\Phi)/(1+\sin(\Phi))}$	Depth		1 87	5										
0.333333333 KD	Weight Footing Depth.	Df	250 218.7	5										
$Kp = 1 + sin(\Phi) / 1 - Sin(\Phi)$	No	,	Ny Ba	-	. ault		84	passive Pres	sure					
	Nα = e <sup>man(D)</sup> tan <sup>2</sup> (π/4+Φ/2) 18.40112222	Ny = 1.5(N 15.08	lq - 1)tan(Φ)) q = γ 398139	. Df . Nq + β . 493.402968 506.965801	γ.Β.Νγ		Θv = Df. y 16.2	en average iv = 0.5 Kp . ( 24.3	Θv					
	Load Case 1 outer field								Sliding ca	nacity		E S Sliding	E S overturning	FS
	overturning moment	Returning net m	noment total V	×	-	1/8	7		Base	e Passive	Total	Hmax / H/cassion	returning / overturning	
	overtaining moment	moment abo	ut toe	~ MA/		20			Frictio	on soil		frieties with exercise	icianing overlaning	
	131.785334 5	31.0022372 399.23	369032 250.6797033	1.59261758	0.407382417	0.666667	2 = L /6 2.6666666667	84.953506	2 Hmax = V . 104.934043 91.239	195 54.675	145.915	0.892443 1.107385	4.029908	5.807
	Load Case 2 outer field fence	d							Cliding on					
	overturning moment	Returning net m moment about	ut toe total V	x	e	L/6	z	q	Hma	х		F.S Sliding	F.S overturning	F.S
	77.2939139	1 500 422.70	M 060861 243.5428189	x = M/V 1.73565407	e = L/2 - x 0.264345925	0.666667	Z = L <sup>2</sup> /8 2.666666667	q = V/L + M/2 87.760195	Z Hmax = V. 93.5450685 88.642	tan(δcs) 234 54.675	143.3173	with passive 1.575311 2.548971	6.468815	5.622
	Load Case 3 inner field													
	overturning moment	Returning momer moment to	nt about total V toe M	x x = M/V	e e = L/2 - x	L/6	Z Z = L <sup>2</sup> /8	q q = V/L + M/2	Sliding ca Hma Z Hmax = V	pacity ix tan(δcs)		F.S Sliding with passive	F.S overturning	F.S
	67.79797371	382.8125 315.01	145263 206.7580923	1.52358983	0.228410172	0.583333	2.041666667	82.392941	90.4697089 75.253	54.875	129.9288	3.717072 6.417879	5.648371	6.1
	Load Case 4 2nd row								Sliding on	pacity				
	overturning moment	Returning moment	nt about total V	x	e	L/6	z	q	Hma	ix		F.S Sliding	F.S overturning	F.S
	151 8252814	500 249 43	M 747188 204 0302702	x = M/V	e = L/2 - x 0 043589744	0 583333	Z = L <sup>2</sup> /8 2 041888887	q = 2V/3x 88.61775	Hmax = V . 79 7148100 74 264	tan(õcs)	128,9392	with passive	5.648371	5.700
	101.0202014	000 040.11			0.040008144	0.000000	2.011000007			04.070				0.120

# Total Uplift (kN) wer of 2 lower of 2 friction only 8.665426463 S Bearing F.S uplift up/ downward Fplug/ upward V V 27880299 1.044095923 S Bearing F.S uplift 31766267 2.204163618 S Bearing F.S uplift 3280223 0.801485385 S Bearing F.S uplift 42869223 0.801485385 S Bearing F.S uplift 4289223 1.013693093

Bearing quit/q 7917676 Bearing 2172662 Bearing

Bearing

15509

2nd row

Horizontal force (kN) Vertical force (kN)

45 0235

Moment (kNm)





uplift lower of 2 friction only 34.66170695	
34.00110365	
F.S uplift	
8.816654471	
F.S uplift	
2.062144055	

ŀ		Cassion desig	n														
	Cassion Specs.					Shape factor		0.3	3								
	Height Diameter footing depth Df Weight Cassion Seperation		1.6 0.85 1.5 22.69800692 1.7	1.7	5		volume of c	oncrete	0.9079203		1.815841						
	Nq Nα = e <sup>πtan(Φ)</sup> tan <sup>2</sup> (π/4+Φ 18.40112222	/2) N	Ny Ny = 1.5(Ng - 1)tan(0 15.0698139	Βe Φ)) q = γ	aring Capacity, . Df . Nq + β . γ <b>566.000746</b>	, quit γ.Β.Νγ		Θv Θv = Df . γ 27	Θh ∋h = Kp . Θv 81		force arm a = Oh . Df/2 60.75	2					
	Load Case 1 outer fiel	d															
	H/ cassion	V/ cassion	overturning moment	Returning moment	net moment about toe	total V	x	e	L/6	z	q		Sliding Resistanc e Hmax	F.S sliding	F.S over	rturning	F.S Bearing
	65.882667	-0.339851635	105.4122672	50.57569215	M -54.836575	23.03785856	x = M/V -2.38028	e = L/2 - x 2.805280915	0.1416667	Z = L <sup>2</sup> /6 0.12041667	= \//L + M/. -428.287	q = 2V/3x -6.4524201	51.6375	Hmax / H/cassion 0.78378	returning / c 0.479	verturning	qult/q -1.32154584
	Load Case 2 outer field fe	nced															
	H/ cassion	V/ cassion	overturning moment	Returning moment	net moment about toe	total ∨	x	e	L/6	z	q		Sliding Resistanc	F.S sliding	F.S over	rturning	F.S Bearing
	28.13486037	3.228590534	50.44284308	35.46540294	M -14.97744	19.46941639	x = M/V -0.76928	e = L/2 - x 1.194280385	0.1416667	Z = L <sup>2</sup> /6 0.12041667	= V/L + M/. -101.475	q = 2∨/3x - <b>16.872406</b>	51.6375	1.835357	0.703	081	-33.5459417
	Load Case 3 inner fiel	d															
	H/ cassion	V/ cassion	overturning moment	Returning moment	net moment about toe	total ∨	x	е	L/6	z	q		Sliding Resistanc e Hmax	F.S sliding	F.S over	turning	
	10.12272371	5.99595385	32.02798223	35.46540294	M 3.43742071	16.70205307	x = M/V 0.205808	e = L/2 - x 0.219191726	0.1416667	Z = L <sup>2</sup> /6 0.12041667	= V/L + M/2 48.19553	z 54.102305	51.6375	5.101147	1.107	326	10.4616752
	Load Case 4 2nd row	,															
						4-4-11/		_		-	_			E O allalian	5.0		
	H/ Cassion	V/ cassion	overturning		M	total V	x = M/V	e = L/2 - x	L/6	Z = L <sup>2</sup> /6	q = V/L + M/2 20 82075	Z	5lidina	F.S sliding	F.S over		40 3503473
	12.4200380	1.55556655	33.32040003	33.40340234	1.55655451	13.34204033	End have	0.324031032	0.1410007	0.12041007	30.03075	101.00000	51.0375	4.1040401	1.043	1000	10.0000172
							End bear (F	ing capacity, q :N/m2)	Bearin	ng cap Q			Friction F		Total capacity	Total	uplift
			Θv average Θv = Dp . γ / 2 13.5	Θv tip Θv = Dp . γ 27	t friction, f (kN Ka . Ov . Tar <b>1.63786605</b>	ν/m <sup>2</sup> ) n(δ)	q = Ov tip . N 496.8303	q Q	plugged = (π . Do <sup>2</sup> / 4 281.92644	).a			plugged F= π . Do . Df . f 6.560523		Plugged Fp+Qp 288.487	Lower of 2	lower of 2 friction only 6.56052265
	Load Case 1 outer fiel	d					-										
	Forces and moment	V/cassion	moment force arr	Bre	ak down of mor	Ments V back	Oh average	Ho	rizontal resista Hhear	ance		Net v		E S Sliding			E S Bearin
	65.882667	-0.339851635	-16.93876036		-17.278612	Oh 16.59890872	av = 0.5 Kp . 40.5	Θv	51.6375					0.78378		[	7.21639223
	Load Case 2 outer field fe	nced															
	Forces and moment			Bre	ak down of mor	ments	]	Ho	orizontal resista	ance							
	H/ cassion	V/ cassion	moment force arr	n	V front	V back Oh	Oh average av = 0.5 Kp .	Θv	Hbear					F.S Sliding			F.S Bearing
	20.1340003/	3.220380334	4.034108100		1.00211309	-1.4033300Z	40.0		31.0373					1.000007		L	11.3000227
	Load Case 5 Inner field fe	Forces and	moment		Break down (	of moments	٦	Ho	prizontal resista	ance							
	H/ cassion	V/ cassion	moment force arr	n	V front	V back	Oh average	θv	Hbear					F.S Sliding			F.S Bearing
	10.12272371	5.99595385	15.62746343		21.6234173	-9.63150958	40.5		51.6375					5.101147		E	8.92333055
	Load Case 4 2nd row	Forces and	moment		Break down	of momente	Ъ	L.,	vizontel regist	ance							
	H/ cassion	V/ cassion	moment force arr	n	V front	V back	Oh average	на	Hbear					F.S Sliding			F.S Bearing
	12.4288598	7.35536039	9.741385202		17.0967456	⊖h -2.38602481	av = 0.5 Kp . 40.5	Θv	51.6375					4.154645		[	11.5008211



# APPENDIX C - EXPERIMENTAL TESTING OF GROUTED CONNECTIONS FOR OFFSHORE SUBSTRUCTURES: A CRITICAL REVIEW (PAPER J1)

# **Full Reference**

Dallyn, P., El-Hamalawi, A., Palmeri, A. and Knight, R., 2014. Experimental Testing of Grouted Connections for Offshore Substructures: A Critical Review. *Structures, Vol. 3. pp. 90-108.* 

# Abstract

Grouted connections have been extensively used in the oil and gas industry for decades, and more recently their application has been extended to the offshore wind industry. Unfortunately plain-pipe grouted connections for large diameter monopile foundations have recently exhibited clear signs of insufficient axial capacity, resulting in slippage between the transition piece and monopile. Motivated by the emergence of such problems, this paper presents a critical review of the technical literature related to the experimental testing for grouted connections for offshore substructures, covering all the key material and design parameters that influence their capacity, including the confinement provided by pile and sleeve, surface finish, simultaneous bending action, connection length, dynamic loading, early-age cycling during grout curing, grout shrinkage, radial pre-stress and temperature. The review also focuses on the relevance of such parameters for offshore wind applications and addresses what needs to be considered to ensure that their design achieves the desired capacity, behaviour and efficiency.

**Keywords** – Experimental testing, foundation design, grouted connections, offshore structures, wind energy converters

Paper type – Review, Journal

# **1 INTRODUCTION**

Grouted connections have had extensive applications for the foundation of oil and gas platforms, where they have been used for main, skirt and cluster piles, as shown within Figure 1. A grouted joint is a structural connection formed by use of cementitious grout cast in an annulus formed between two concentric circular tubes with different diameters. The principal methods of load transfer are through shear friction mobilised by the normal stress induced by interlocking of surface imperfections, and compression of the grout.



Figure 37 Typical foundation pile arrangements where grouted pile – sleeve connections are used; a) Main and skirt Piles, b) Cluster piles [1]

With the aim of optimising the design of platform foundations and reducing material quantities, extensive work was carried out in the late 1970s and early 1980s to quantify the performance of both plain-pipe and shear-key grouted connections through experimental testing, particularly for offshore applications. This stream of work was predominantly focused on the influence of grout strength, shear-key height and spacing, ratio of diameter to thickness of the piles, outer sleeves and grout annulus on the ultimate capacity of the connection.

Since 2002, grouted connections have also been used extensively in the offshore wind industry, where large diameter-sleeved grouted connections comprise around 60% of installations in Europe [2]. Some typical details for an energy converter 80 m tall, capable of 80GWh/year are shown in Figure 2. Unfortunately it has been reported that the axial capacity of these plain-pipe grouted connections may be insufficient over the design life of the plant, with significant

unexpected early-stage settlements resulting from this insufficient design [3]. This has led to very expensive ongoing remediation works being required to existing foundations affected by these failures. In addition to the use of grouted connections for monopiles, they are starting to be more widely used between the pin piles and jackets for offshore WTG (wind turbine generator) in sites with deeper mean water level (MWL), with the market share of jackets and tri-pile substructures increasing from around 10% to 20% from 2010 to 2013 respectively [2]. The reason for the continued use of the grouted connection is that with the inclusion of shear keys it offers a cost effective way to provide an efficient structural connection, while being able to accommodate piling installation tolerances.



**Figure 38 Typical general arrangement of a plain-pipe grouted connection for an offshore WTG foundation** The unsatisfactory performance of grouted connections reported in recent offshore wind installations, such as Horns Rev 1, Kentish Flats and Belwind, as well as the predicted growth of this highly strategic energy sector in future years, motivate this review paper, which is mainly focussed on grouted pile sleeve connections, with the aim to understand whether: i) the cost of these failures could have been avoided; ii) occurrence of similar issues can be reduced in the future. References pre-dating 2007 have been reviewed to provide a detailed assessment of the

level of knowledge and understanding within the offshore wind industry at the time when grouted foundations with insufficient capacity were designed. As detailed in the following sections, the majority of these references consist of research related to the behaviour of axially loaded connections, as used in the oil and gas industry for jacket structures, while the effects of bending actions were marginally investigated before 2007, predominantly in relation to the loading conditions experienced by wind turbines. An account of research carried out in more recent years is also given, to identify current trends and challenges.

For the sake of clarity, the literature review has been broken down into sections based on the different areas investigated by the papers and the relevance of the testing methodology to offshore wind applications, with papers presented chronologically to demonstrate how industry knowledge has developed over the years. A summary of the key testing information from each paper is presented in Table 2 (which includes also a comparison with typical offshore WTG foundation grouted connections, for both monopile and jacket pile connections), which accompanies and supports the key conclusions of previous work and recommendation for future research.

# 2 STATIC AXIAL TESTING

Billington and Lewis [4] have carried out pioneering work on the experimental assessment of the axial capacity of grouted connection in the 1970s. They have used the results of over 400 tests to determine the effects of surface condition of steel, radial stiffness, grout properties and length-to-diameter ratio. Key grout properties included Young's modulus, compressive strength and expansion/shrinkage during curing. In particular, the influence of shrinkage on full-scale specimens was investigated, showing that even though compressive strength of grout increased by 61%, effects of shrinkage were to reduce the bond strength by 42% for plain-pipe connections; while for shear-key connections there were no noticeable effects of grout shrinkage or expansion on bond strength. Bond strength is defined as the ultimate axial capacity of the connection dived by the surface area of the grout annulus/pile interface. It was also shown that variations in surface roughness have a dramatic effect on bond strength of plain-pipe connections. Piles tested with typical surface rusting, as used in practice, showed greater variance in results and up to 25% less bond strength than shot-blasted surface test samples (Figure 3). Having used over 30 tests to derive each curve, these results are statistically significant and so there is confidence in the results presented.



Figure 39 The effect of surface roughness on the relationship between bond strength and compressive strength for plain pipes; where A represents the plain pipe factor, which depends upon surface roughness and pile/sleeve geometry [4]

Tests on plain pipes indicate that the reduction in radial stiffness, which accompanies an increase in pile diameter ( $D_p$ ), for a given wall thickness (t), can lead to unacceptable reductions in ultimate axial bond strength. Axial capacity testing was undertaken on eight samples of varying stiffness factors K (Eq. (1)) and showed limited scatter.

$$K = (E_g/E_s)(t/d)_g + [(d/t)_s + (d/t)_p]^{-1},$$
(1)

where subscripts g, p and s means grout, pile and sleeve respectively; t is the thickness; d is the diameter; and E is the Young's Modulus. Based on this, it was shown that the relationship between bond strength ( $f_b$ ) and grout cube strength ( $f_{cu}$ ) could be represented by the empirical expression:

$$\mathbf{f}_{b} = \mathbf{B} \cdot \mathbf{K} \cdot \mathbf{f}_{cu}^{1/2},\tag{2}$$

where B is a factor dependent on length-to-diameter ratio and surface roughness. The authors compared these findings to the bond strength predicted by the American Petroleum Institute (API) and the Department of Energy (DOE) and found that the safety factor reduced to 1.5 for low stiffness connections, well below the recommended value of 6.0. For shear-key connections the results are only based on five samples and the distribution of stiffness factors is very limited, and so the significance of the derived relationship is more limited. For shear-key connections a relationship between bond strength and stiffness factor was derived in the form:

$$\mathbf{f}_{b} = \mathbf{C} \cdot \mathbf{K} \cdot (\mathbf{h/s}) \cdot \mathbf{f}_{cu}^{1/2}, \tag{3}$$

where C is a factor dependent on length-to-diameter ratio; and (h/s) is the shear key height divided by spacing. The devised relationships between bond strength and stiffness for both plain and shear-key connections are shown in Figure 4, demonstrating the enhanced performance of the second type of connection, whose mode of failure consists of crushing of the grout ahead of the shear-key and diagonal cracking originating from the head of the shear-key.



Figure 40 The effect of stiffness factor on bond strength for both plain pipes and pipes with mechanical plains, adapted from [4]

Comparison of full-scale and small-scale (1:4) tests on shear-key connections were reported to give directly comparable results in terms of bond strength (Figure 5). However, no testing was

carried out on plain-pipe connections and so the influence of scale effects on this connection type was not assessed. In addition, the maximum full-scale grout strength was 45MPa, while agreement cannot be confirmed beyond those values.



**Figure 41 Results of 1:4 scale bond tests using three different grouts compared with full-scale results [4]** The test results were stated as being conservative due to the pile being in tension because of the loading arrangement and so the corresponding effects of the Poisson's ratio tend to separate the steel from the grout. Moreover, in this pioneering study there is no mention of the influence of the increased effective radial stiffness of the pile and sleeve provided by the loading plates due to their close proximity to the grouted connection.

The same data set as in Ref. [4] was further analysed by Billington [5], pointing out that large scatter in the experimental results, particularly for plain connections (h/s=0) needs careful consideration (Figure 6).



### Figure 42 Relationship between F<sub>bu</sub>/K<sub>CL</sub> and h/s [5]

It is confirmed that bond strength is proportional to the square root of grout compressive strength,  $f_{bu} \propto f_{cu}^{0.5}$ . Research into the benefit of composite structures was also presented, which showed significant stress reductions in principal stresses of around 60% for the grouted chords and braces compared to similar non-grouted configurations. The results were reasonably significant from a statistical point of view, as they were based on five samples for each type. Some results were also presented for the influence of bending actions on the axial capacity of annular connections, which are reviewed and discussed in Chapter 5.

Billington and Tebbett [6] continued the work presented in Refs. [4] and [5] to derive DEO formulae for the ultimate capacity of plain and shear-key grouted connections, filling some gaps and further increasing the significance of the results previously derived. They also presented results of subsequent phases of testing, looking specifically at the effects of cyclic loading. A more detailed investigation into the partial safety factors applied for assessing the ultimate bond strength, with reference to their laboratory work experience, indicated that an overall safety factor of 4.5 could be used, rather than the larger value of 6.0 commonly adopted in the offshore industry at that time. Although the paper states that for plain connections the ultimate bond strength ( $f_{bu}$ ) is proportional to the square root of grout cube strength ( $f_{cu}$ ), the results presented in Figure 7 indicate that for values of  $f_{cu}$  larger than 45MPa,  $f_{bu}$  does not increase further, at least
for the analysed conditions. The paper however fails to identify this aspect, which is quite important in practical applications, as it appears that there is little advantage in using grout with higher compressive strength if the failure is dictated by the steel-bonding strength in plain-pipe grouted connections.



Figure 43 Relationship between bond strength and compressive strength for plain pipes [6] A linear relationship for bond strength against shear connector spacing is presented, but the relationship is only based on three ratios of bond strength to spacing and for this reason it has little significance. A relationship for relative axial displacement between the pile and grout was also presented based on small-scale (1:4) samples. Results indicated that at upper bond strength (normal stress  $\sigma = f_{bu}$ ), the axial displacement is  $\delta_{um} = D_p/40$ , meaning that for typical offshore wind monopile foundations this would be equivalent to more than 100mm axial displacement. For lower loads, the following relationships were proposed:  $\delta_{um}/10 = D_p/400$  for  $\sigma = f_{bu}/2$ ;  $\delta_{um}/50$  for  $\sigma = f_{bu}/6$ . Preliminary results of fatigue tests were also reported, with five shear-key samples tested under zero-mean stress and equal tension/compression cycles which did not show signs of fatigue damage up to  $10^7$  cycles for normal stress up to 40% of the ultimate strength.

In parallel with the work conducted by Billington and his associated, another large experimental campaign was led by Karsan and Krahl [7], aimed at deriving the design equations for the new API code of recommended practice for grouted connections and justifying them through a reliability analysis. This was based on 201 tests, reduced to 147 to ensure grout strength greater

than 17MPa; of which 62 were plain-pipe connections. They also compared this to the DOE's testing, which had 117 tests, consisting of 44 plain and 73 shear-key samples. The results of the comparison therefore have high statistical significance. Similarly to Billington [4], they observed a considerably greater variance in the factor of safety for the plain-pipe connection tests than for shear-key connections (Figure 8), again demonstrating that the codes at the time did not correctly account for factors which influenced the capacity of plain-pipe grouted connections.





Interestingly, the authors stated that the effects of loading conditions other than axial, such as bending, transverse shear or torque, may be important and such loads, if significant, should be considered in design by appropriate analytical or testing procedures due to lack of published data, but no work was carried out by Krahl and Karsan to quantify their significance.

Further results by Krahl and Karsan are presented in Ref. [1]. As in the previous papers, there is a considerable scatter of data for plain-pipe connections, and it can be seen in Figure 9 that for values of the ultimate grout strength  $f_{cu} > 50$ MPa the ultimate bond strength  $f_{bu}$  does not increase further. This is in contradiction to Billington [4, 5], who had proposed  $f_{bu} \propto f_{cu}^{0.5}$ , with higher limits of equation validity on grout strength than 50MPa. As stated in both papers by Krahl and Karsan [7, 1] it highlights the importance of not using design equations beyond the limits of the experimentation that they were derived from, as the data trends may not apply beyond these limits.



Figure 45 Measured bond strength versus grout compressive strength for tests of grouted tubular connections without shear connectors [1] Like Billington [4], the authors discuss the failure modes of the grout being a combination of grout crushing and slippage at the pile-grout interface, with diagonal cracks between shear keys, but they developed it further analytically by considering the equilibrium of a free body diagram for a piece of grout between two consecutive cracks, as shown within Figure 10. They also proposed other possible failure modes if shear key spacing and heights are sufficient, including pure shear of the grout. However, none of the experimental testing undertaken was in the region that these failure modes would be expected and so the limits of occurrence were not validated.



Figure 46 Free body diagram of cracked annular grout (full ring with cross section shown) a) larger s/t<sub>s</sub>, b) smaller s/t<sub>s</sub>, c) crushing forces in grout at shear key [1]

Tebbett and Billington [8] and Tebbett [9] reviewed some previous work undertaken at Wimpey laboratories and carried out additional testing to extend the range of validity of the DOE equations. It was noted that when high sleeve stiffness ( $(D/t)_s < 40$ ) is combined with high pile stiffness ( $(D/t)_p < 32$ ), then the DOE design code overestimates the connection strength. Early-age cyclic loading during curing was also found to lead to larger displacements at ultimate load, roughly 200% greater than samples cured under static load (see Table 1), but the ultimate load capacity remains unchanged. This data provided substance to ideas earlier touched upon by Billington [4, 5], with reasonable significance as these conclusions were based on eight samples.

	DISPLACEMENT AT ULTIMATE LOAD								
SPECIMEN TYPE	STATIC CURING (MM)	CYCLIC CURING (MM)							
OB with shear connectors	8	18							
OB plain pipe	18	45							
HAC with shear connectors	8	15							
HAC plain pipe	18	30							

 Table 1 Comparison of typical displacement at ultimate load

Unlike Billington [4], Tebbett [9] stated that specimen size may influence test results and so recommended full-scale tests should be performed.

For fatigue loading in which the applied stress was less than 40% of the ultimate bond strength, no failures occurred at less than  $10^7$  cycles for shear-key connections, which is in agreement with the previous findings of Billington [4, 5] at the same test laboratories, but this is now substantiated by a greater number of experiments (ten samples). For higher loads, failure is due to degradation of the grout, through fatigue and void formation around the shear keys. The cyclic movement induced significant reduction in stiffness at both working and ultimate load, but only very close to the end of life. For plain-pipe connections subjected to cyclic axial testing, it was found that they are less susceptible to fatigue than connections with shear keys, but only one sample was tested and so no S-N relationship was presented and the significance of the results is limited.

It was also demonstrated that API's constant strength approach for plain-pipe connections overestimates the actual strength by an average of 12%, and the extrapolation of relationships from limited data sets was considered the main reason for such inaccuracies, again highlighting the importance of design limits.

Lamport, Jirsa and Yura [10] looked at determining the effects of relative shear key location, grout strength, pile to sleeve eccentricity and combined axial and proportionally applied moment loading on the resultant axial capacity of the connections. Testing was reasonably statistically significant, with 18 samples tested, but typically only two values of each variable were investigated, and so no trend could be determined. Overall, they found no noticeable effects of grout thickness or relative position of shear-keys on the overall capacity of the connection. This is in disagreement with the findings of Forsyth and Tebbett [11], which appear to demonstrate that grout thickness is an important contributory factor, as in a thicker grout annulus, grout compression struts are oriented closer to the radial direction and therefore shearing of the grout is more likely to happen. They also considered that the optimum value for the spacing of the shear keys will depend on their geometry, as well as on the radial stiffness of pile, sleeve and grout. Previous tests for a constant height-to-spacing ratio (h/s) showed large scatter, suggesting that small geometric variations can have an effect. Their results were reasonably significant with four h/s ratios, repeated at least three times using the same connection specifications. This provided an indication to the optimum height to spacing ratio of approximately 0.075, which being outside the limits of previous work [1, 6, 8, 9] (h/s = 0.04), therefore demonstrating the benefit of increasing such ratio in order to improve the efficiency of the connection.

Smith and Tebbett [12] presented findings of testing related to remediation works for what was the largest gas production platform in the world, North Rankin A, off the Northwest coast of Australia. Testing was required as the pile geometries were outside limits of existing design equations. These works included the use of grout plugs to improve the pile end bearing capacity and pile sleeve connections to transfer load from the piles to under-reamed pile bells via a tubular insert. As part of this research, they investigated scale effects with 0.25, 0.3 and 1.0 scale samples for grout plugs and the applicability of using conventional grouted pile-sleeve connections design codes. This validated the previous hypothesis of Tebbett [8, 9] that noticeable scale effects affect the experimental results, with a reduction factor of 0.80 between full and quarter scale. The investigation of varying the h/s ratio revealed the different failure modes, previously hypothesised by Krahl and Karsan [1], with shear failure of the grout across the tips of the shear keys, resulting in 35% of the capacity predicted by design guidelines available at that time, which assumed shear from pile to sleeve shear key tips. This is in agreement with the suggestion of Forsyth and Tebbett [11] that there is an optimum value for the ratio h/s. Results from the testing were reasonably significant, with four different shear-key height-to-spacing ratios being investigated, but with insufficient repetition to confirm any trend. As in most previous reported works, it was demonstrated that extrapolation beyond design code limits can lead to a reduction is safety factors.

Sele and Kjeøy [13] presented the background to design equations for the draft rules of fixed offshore structures developed by DNV (Det Norske Veritas). Friction tests were carried out on grout, based on oil well cement, under varying normal loads, which exhibited a friction coefficient  $\mu = 0.7$  and cohesion strength of 0.1MPa.

Key findings of axial capacity tests were that a small gap had formed between the grout and steel due to shrinkage, even with so called non-shrink grouts. For 30-50mm of grout, shrinkage was in the order of 0.01mm. These are significantly smaller than those reported by Billington [4] for normal grouts. Failure mode for shear-key connections were described and in agreement with previous papers [1, 4, 11, 12].

Testing showed pronounced slip then stick action, Figure 11, i.e. large displacement under a constant or reducing load followed by small displacement for an even higher load. This

suggests that significant displacement must occur in order to mobilise the capacity of the connection, which agrees with the findings of Billington and Tebbett [6, 9]. At ultimate axial load, shear is essentially mobilised from interlocking due to surface imperfections which induces normal stresses and therefore friction, with little or no effect from cohesion or adhesion.



Figure 47 Load deformation recording from a pile sleeve test [13]

This work also demonstrated the importance of surface irregularities' magnitude, with machined surfaces having a significantly reduced ultimate capacity and not showing any radical change in the coefficient of friction (dynamic to static), which was further elaborated in a later work by the same authors [14]. However, because this test comprised of only one sample, it had limited statistical significance.

Aritenang et al. [15] focussed their research into load transfer and failure mechanisms involved within shear-key connections, with the aim of deriving a numerical model that was to be calibrated against the results of a limited test programme; the derivation and validation of which are presented in [16]. A key point raised in this testing methodology was to ensure the boundary effects of the loading rig are minimised, with the load applied at one diameter from the connection end, as any additional confinement in close proximity to the connection will fictitiously increase its strength (a phenomenon well know, for instance, when cubic and

cylindrical concrete samples are tested). This was not mentioned in any of the other previous works where axial tests were undertaken, but are reported by the author as being a feature of the DOE testing. Only six samples were tested, investigating two weld bead heights and three levels of confinement of the pile, so limited significance can be drawn from these results. Detailed structural monitoring was also undertaken for the first time on an axially tested grouted connection, through installed strain gauges close to the weld beads in both the axial and hoop direction. This investigation, along with the post failure inspection of the sample, indicates that confinement is a key parameter to the ultimate strength, and the observed failure mechanism with 45° cracks between shear keys showed good agreement with the findings of other researchers [1, 4, 11-13]. However, for the first time they proposed a mechanism in which these cracks initiate at the centre of the grout annulus and then propagate towards the shear-key upon loading beyond ultimate failure, but shows good agreement with the later strut and tie model presented by Löhning and Muurholm [17]. The results of the influence of increased shear key height on connection strength were also less than the DOE predictions, for which shear-key height was directly proportional to connection strength. This was hypothesised as being a result of increased local plate bending due a larger shear-key height, reducing the effective stiffness of the pile.

Elnashai and Aritenang [16] used previous experimental testing [15] for validating a new numerical model. Comparison with the results of six samples, with three different pile thicknesses and two shear-key heights, showed a good agreement, with the greatest discrepancy being 18%. As limited samples for the number of investigated parameters were tested, more validation with experimental testing would be required before results from the model can be considered significant. The comparison of predicted results with the experimentation undertaken in Aritenang et al. [15] highlights the accuracy of the new numerical model over the API and DOE formulas through a less dispersed distribution of predictions, as shown within Figure 12. In addition, Elnashai and Aritenang [16] concluded that the exclusion of a radial

stiffness parameter from the API code could not be justified given the inconsistent results from the API formula.



Figure 48 Bond strength prediction a) proposed numerical methods b) DOE Formula c) API Formula [16] Lamport, Jirsa and Yura [18] looked at reducing the safety factors of future design in comparison to the high safety factors used by the API, DNV and DOE design codes. They extended the work presented in [10], considering the influence of factors that had not been previously tested in depth, such as the effects of moment loading, relative pile and sleeve shear key position and pile sleeve eccentricity. Testing was also undertaken to validate a proposed analytical model and to investigate the effect of these parameters, but had limited statistical significance, as there were only three variations for each parameter investigated. Unlike other testing methodologies reported in the reviewed literature to date, the tested samples were manufactured using the same procedure as in the actual offshore application, with the grout injected from the bottom of the connection, displacing water as it filled the annulus until a change in colour of the grout is noted in the overflow. Cube strengths were taken at the top, centre and bottom of both a 0.9m (3ft) and 1.8m (6ft) column, with only the lower part of the 1.8m column having a similar strength to that of the unconfined cube strength of samples prepared to ASTM C109 [19]. For the tall column, the top sample showed only 50% of the reference cube strength; more generally, the strength was seen to decrease with the height, as

shown in Figure 13. With three samples taken at each height, the results have minimum statistical significance. This finding is particularly relevant to the offshore wind energy industry, as typically grout used for such applications has to travel large heights within the connection and pumping is stopped as soon as overtopping is seen. It follows that a strong possibility exists of significant variation of grout properties over the height of the connection, particularly in top two metres. In the oil and gas industry much attention has been paid to ensuring the required quality of grout completely fills the connection, using density gauges and significant over-pumping.



**Figure 49 Histogram of injected grout column cubes versus ASTM C190 Cubes for 27–41MPa grout [18]** The other key finding in Ref. [18] was that, based on the limited experiments, eccentricity of the pile sleeve connection or relative shear key position had no noticeable effect on axial strength. Like Refs. [1, 5, 12–15] post- failure investigation revealed grout failure cracks between 20° and 60°. The use of the grout compression strut was then used to derive an analytical model, similar to Ref. [1]. Comparison with predicted results from DNV, API and DOE also highlighted that the DOE showed the lowest variation with respect to the measured values of strength.

Finally, similarly to the work of Forsyth and Tebbett [11], the analytical model by Lamport, Jirsa and Yura [18] suggested an optimum h/s value of about 0.075. Although not validated experimentally, this was explained by the change in the failure mode of the grout from

compression struts to pure cylindrical shear failure plane, as hypothesised by Krahl and Karsan [1] as well as by Forsyth and Tebbett [11].

Aritenang, Elnashai and Dowling [20] built on their previous work by investigating a larger range of parameters beyond those previously covered and extending it to a finite element (FE) model for welded shear keys. The FE model outputs showed good agreement with the derived analytical model. When compared to previous experimental testing, there was reasonable agreement for a variety of parameters. Like Krahl and Karsan [1], they suggested a grout strength limit above which bond strength does not increase; however this was seen to be 35MPa rather than 50MPa (i.e. 30% less). The experiments reported in Ref. [19] also investigated the effects of sleeve thickness on the bond strength in grouted connections. As shown in Figure 14, the tests conducted on five different sleeve thicknesses (from 4 to 20mm) and three values of pile thickness (from 12.5 to 25mm) revealed that sleeve thickness has a noticeable effect.



Figure 50 Effect of pile and sleeve thickness, adapted from [20]

Sele and Skjolde [14] used data from 750 tests to assess the predictiveness of available design equations and identify new trends. They concluded that the DNV equation provided more robust predictions than the DOE and API equations when compared to the test data set, which contradicts the finding of Ref. [18], where the DOE were shown to be more accurate, having the

lowest variation. However the statistical significance of Ref. [18] is lower as only 16 samples were used, while 258 formed the basis of in Sele and Skjolde's comparison.

It was found that a "wedging" action caused by the uneven surface of rolled steel generates hoop stresses in pile and sleeve and is the main source of axial strength for plain-pipe connections. This was demonstrated by tests performed by DNV using pipes, which have been turned down in a lathe to produce a uniform surface, showing a radical reduction in axial load carrying capacity. Strength of plain pile connections was therefore concluded to be dependent on the magnitude of the wall surface unevenness, as well as on the hoop stresses of the pile and sleeve.

Failure modes for shear-key connections were reported as either being grout compression struts with  $45^{\circ}$  failure planes between shear keys or cylindrical shear failure of the grout at the tip of shear keys, depending on the shear key h/s ratio and grout strength, which is in agreement with [1, 4, 12, 13, 15, 18].

Detailed investigation was also made into the nominal dimensions quoted for tube thickness and this showed variation of 5–15%, especially for smaller samples. Interestingly, this was rarely measured in previous testing, and could then account for some of the scatter in the data, as the confinement is one of the key parameters for the connection axial strength.

Harwood, Billington, Buitrago, Sele and Sharp [21] provided the background to the formation of the ISO standard. As this consisted of a review of 30 testing programmes with 626 individual tests screened to 193 results for axial capacity of grouted connections based on well-defined criteria, there is a good significance to their findings. A new interface transfer strength term was proposed to replace the traditionally adopted bond strength in order to highlight that little adhesion (bond) is experienced in practice. A statistical review of five design formulae highlighted that the predicted strength from a modified HSE design approach showed the best agreement with the experimental measurements. This therefore formed the basis for the ISO formulation, with exclusion of some parameters that showed little significance, such as load and length parameters, and modification of others such as radial stiffness, shear-key density and grout strength. The design formulae covered the two failure modes shown by Smith and Tebbett [12] of grout compression struts or cylindrical shearing of the grout matrix occurring at higher h/s values. The validity ranges based on the limits of the screened data and over which the formulae were shown to be accurate are clearly stated. Like Lamport et al. [18], Harwood et al. [21] note the variability on grout strength over the length of the connection and propose the use of an effective length to take account this. An h/s of 0.05-0.07 is presented for optimum connection capacity, which is lower than the 0.075 factor suggested by the authors of Refs. [11] and [18]. A detailed look at the influence of early age cyclic movement during curing on axial capacity showed agreement with the authors of Refs. [8] and [9] in terms of marginal influence on static strength at h/s= 0.012. However, for h/s > 0.05 adverse effects on static strength were observed and so a factor to account for this is included in the design equation, with clear guidance on the radial displacement magnitudes used its derivation (+/- 0.35% of D<sub>p</sub>).

### 3 **PRE-STRESS**

Dowling, Elnashai and Carroll [22] provided a good review of the previous research undertaken on grouted pile-sleeve connections, which highlighted that there had been no successful attempt to analytically model the connection. The experimentation showed the importance of confinement of the grout: that, by simply applying a radial pre-stress, the bond strength for a plain connection was increased by about six times over the unstressed connection, as the prestress overcomes the loss of radial confinement due to curing shrinkage (Figure 15). The proposed analytical model showed a reasonable agreement with experiments, but it was only applicable to a single value of material strength for grout and steel. Although four different levels of pre-stress were used to define the relationship, no attempts were made to demonstrate repeatability. Similarly to the authors of Refs. [1, 4-9, 11-16, 20], the experimentation does not consider the confinement provided by the test rig, as the loading is directly applied at the end of the connection for the push-out test performed.

The report also touches upon tensile loads resulting in a reduction of bond strength compared to compressive loads, which is in agreement with Billington [4].



**Figure 51** Comparison of the effect of applied pressure on bond strength; experimental and FEM [22] Elnashai, Carroll and Dowling [23] further investigated the effects of mechanical pre-stress on the capacity of grouted connections. They reported a minimum of six times improvement by using pre-stress in the ultimate capacity of plain connection for the same dimensions, which was based on seven tests and so with good statistical significance. They were the first to report reduction in capacity upon reload of around 23% for expansive grout tests. They also reported decreasing bond strength with increasing length-to-diameter L/D ratio of connection for both plain and pre-stressed connections, as the entire slip length L is not contributory to strength. This is in agreement with non-pre-stressed connection findings presented in Refs. [4-6, 20].

Elnashai et al. [24] further studied the use of pre-stress, both mechanically and chemically (expansive agent in grout), investigating the effects of confinement and length of connection through two different D/t (diameter-to-thickness) and L/D (length-to-diameter) ratios. Similar strength improvements were reported for both techniques, but are limited by the radial stiffness of the connection. The difference in axial strength of the two samples of different radial

stiffness was explained by Elnashai et al. [24] as being due to the areas near the end of the connection showing less separation between grout and steel, due to Poisson dilation effect being less for higher radial stiffness samples, resulting in lower loss of pre-stress. They also developed a FE model that showed a good agreement with the experimental testing, their prediction always being within 8% of the measured average bond strength for the six samples tested. However, only two samples were tested with the expansive grout, and so there is limited significance for this type of pre-stress.

Gundy and Kiu [25] also investigated the enhanced axial capacity that can be achieved through pre-stress. This was done both experimentally and analytically, based on shell bending theory and FE modelling. Their findings were in agreement with Elnashai et al. [20, 22] for plain connections, but also showed a 50% improvement of bond strength by pre-stress on shear-key connections. The results of this research were reasonably significant with limited scatter for the eight tests undertaken. They also studied the mechanics of connection strength, which indicated that capacity is mobilised over a finite length at the end of the connection until first slip occurs, with peak shear occurring at  $0.1D_p$  (35mm) from the end of the connection and negligible by  $0.56D_p$  (200mm) (Figure 16). This was used to explain the effective reduction in bond strength with increasing L/D ratios. The analytical approach based on classical shell theory was very closely matched to the FE results, but as can be seen from Figure 16, it only produced similar trends to the experimental findings of shear distribution, not magnitudes.



LM4:  $L/D_p = 1.0$  (shot blasted surface)

## Figure 52 Interface shear – analysis versus test [25] 4 DYNAMIC AXIAL LOADING

Boswell and D'Mello [26] investigated clamp fatigue performance based on experimental testing of ten samples with shear keys, using 0.1 and 0.5Hz as loading frequencies, which are typical of wave action in the North Sea. The key findings were that higher strength grout showed a relatively poorer fatigue performance, with a fatigue limit at around 20.7% of ultimate strength for the higher strength grout, compared to 32.5% for the lower strength. This was around half the critical value predicted by the work reported in Refs. [4, 6, 8, 9], but this could be down to variation in connection type, shear key configuration, etcetera, as the influence of these additional parameters was not investigated. The results themselves have reasonable significance with nine samples tested.

Ingebrigsen, Løset and Neilsen [27] investigated static and fatigue design of grouted pile sleeve connections. It consisted of over 150 tests, of both previous (Ref. [13]) and new test data, for 1:3 and 1:5 scale, with 68 plain-pipe samples and 96 shear-key. They highlighted the different definition of the ultimate load capacity within DNV (as the load at first slip) and DOE (as peak load, independently of the corresponding displacement).

# - Experimental Testing of Grouted Connections for Offshore Substructures: A Critical Review (Paper J1)

It was found that plain-pipe connections are highly resistant to dynamic loading, while connections with shear keys appeared to be less resistant compared to their respective higher static strength for tensile-compressive loading. However, only one plain-pipe connection was tested with reversed stress cyclic loading and so very little significance can be drawn from this. For axial compression-compression cyclic loading the same results were concluded and these were significant given at least nine samples for each connection type were tested. It was stated by the authors that fatigue is not an issue if tensile stress is less than 20% of the static stress for shear-key connections, which is in agreement with the findings of Boswell [26]. The fatigue design equations presented were based on 30 samples tested as part of this and previous research and so are statistically significant.

Harwood et al. [21] reported initial onset of fatigue damage being evident by small relative movements between the pile and grout which increased with increasing number of cycles. They reported a large reduction in fatigue performance of connections that had been subjected to early age cyclic movements and that there was evidence that fatigue performance reduced with increasing h/s. Similar to authors of Refs. [27] and [28], a mean fatigue endurance limit of 20% was indicated by the screened data for shear-key connections. It was also shown that the magnitudes of the of the reverse cycle loading previously used for axial fatigue testing were onerous compared to the actual service loading.

Etterdal et al. [28] investigated the use of high-strength grout for strengthening of offshore steel components using the commercially available Densit Ducorit® grout S5 and D4, as part of the research required for the strengthening of 75 braces/chords on Norwegian Ekofisk oil and gas platform jackets due to sea bed subsidence. Although not directly applicable to previous grouted connection testing, this experimental work highlighted the effect of the load history, with grout not very effective on first load-up, but marked improvement in capacity on repeat

load cycles in the same direction. However, if the load is reversed, efficiency is lost until the loading is repeated. This was in agreement with the findings of Sele and Kjeøy [13].

In terms of the relative displacement between grout and steel, Zhao, Grundy and Lee [29] investigated grouted sleeve connections under large-deformation cyclic axial loading, for applications in earthquake engineering. As a result, displacements were in the order of ten times those typically seen in monopile connections for offshore wind energy installations. They reported an increase peak load capacity with increasing cycle numbers for lower pre-stress levels, which was explained by thermal expansion of the inner pile increasing the pre-stress. However, this was counteracted by decreased capacity in the coefficient of friction due to powdering of the grout at the grout steel interface, along with degradation of the grout through cracking and spalling with increasing cycles. This was the dominant factor for high pre-stress samples where a reduction of capacity was seen from the outset. Figure 17 shows both the low and high pre-stress results. These rates of capacity reduction were found to be dependent on the magnitude of the axial displacement, with greater displacements, i.e. distance walked, showing quicker reductions in capacity. No investigation on the implication of surface finish was made. Only eight samples covering three different variables were used to derive the influence of each factor and so limited significance from the results can be drawn. It does however provide some useful insight and highlight some areas worth of further investigations, which have not been previously mentioned by other authors.



Figure 53 Load-displacement of low pre-stress (left) and high pre-stress (right) [29]

# - Experimental Testing of Grouted Connections for Offshore Substructures: A Critical Review (Paper J1)

Although the WTG grouted connections do not utilise pre-stress and the magnitudes of the relative displacements are considerably smaller, the normal compressive stress between the grout and the steel required for wear and grout powdering is provided by the large overturning moment experienced by the connection. The high number of such load cycles means that the findings of Zhao et al. [31] that the axial capacity tends to reduces with the number of cycles and the powdering of grout occurs are applicable to the overall lifetime of offshore WTG with plain-pipe grouted connections. This wear could not only result in exacerbation of the insufficient axial capacity already experienced, as surface irregularities become worn and the degree of interlocking between the grout and steel interface reduces, but material loss can also produce significant dynamic loads.

Lohaus and Anders [30] presented a comparison of high cycle fatigue response of different ultra-high performance concretes (135-225MPa), with static and fatigue testing performed on scaled shear-key connections. It was reported that the fatigue strength seemed to be marginally lower for ultra-high performance concrete (UHPC) (Figure 18), which is in agreement with the findings of Boswell and D'Mello [25].



Figure 54 Comparison of different fatigue tests on UHPC mixes [30]

They also hypothesised that a relative axial displacement of 2% of the connection length under ultimate axial load for a shear-key connection might have detrimental effect on fatigue strength,

which is similar to the findings reported within Refs. [8, 9], where it is hypothesised that under large loads and displacements the formation of voids due to grout crushing would be detrimental to fatigue behaviour.

The effect of temperature on grouted connections was investigated by Zhao et al. [31], with the focus being on the fire engineering aspects of composite steel-concrete tubular connections. The paper demonstrated that rising sleeve temperature, and therefore increasing the temperature difference between sleeve and pile, causes a decrease in the axial strength, which validates part of the hypothesis of Zhao et al. [29]. The testing was fairly significant with a minimum of three samples tested per variable. The developed model predicts the response of the tests with reasonable accuracy, but the properties of the grout are not considered, as no attempt was made to record properties such as elastic modulus, Poisson's ratio or strength.

Schaumann and Wilke [32] presented findings of numerical and experimental modelling of grouted connections with and without shear keys. Axial testing based on Densit Ducorit® S5 high-strength grout showed that the overall strength of the connection is considerably increased by the presence of shear keys (Figure 19), which is in agreement with all previous axial testing. The authors therefore recommend applying additional mechanical interlock even for monopiles with relatively low axial forces, given the significant gain in capacity shown by the addition of shear-keys.



Figure 55 Load-displacement curves for different small scale specimen [32]

Similar to previous work reported in Refs. [1, 4, 12, 13-15, 18], grout failure modes were investigated and showed that transverse cracking of the bottom compression strut occurs at 50% ultimate load, indicated by the kink in load response within Figure 19 [32] with magnitude of around 1.5% the length of the grouted connection. The observed degradation in the capacity of the connection validates the theories of Tebbett and Billington [8], Tebbett [9] and Lohaus and Anders [30].

Schaumann and Wilke [39] also stated that capacities of tested samples are reduced if bending moments due to loading plates are excluded from numerical analysis, which is in agreement with the findings of Aritenang et al. [15]. Numerical analysis was shown to give good agreement with experimental results, but this was only for one sample. After two million cycles, specimens almost reach the load of static tests, as shown in Figure 19, showing limited damage occurring at that load level for the number of significant load cycles expected in offshore environments; a result that is in agreement with early findings reported within Refs. [8, 9]. Additionally, it should be noted that this study only considered compression - compression cycles, which explains why the performance is better than what was reported by Boswell and D'Mello [26]. It was also found that local deterioration of the grout around shear keys, represented by a dimensionless damage parameter D, does not reduce the global capacity. They also demonstrated that the detrimental effect of shear keys, in terms of pile steel fatigue for predominantly bending moment loaded connections, can be avoided if they are placed in the middle third of the grouted sleeve, illustrated by comparison of graphs in Figure 20.



Figure 56 Fatigue damage comparison between shear keys concentrated to the middle third of the connection (left) and evenly distributed along the whole length (right) [32]

Anders and Lohaus [33] also investigated axial loading of grouted connections and the same aspect of the influence of the increase in compressive strength of grout and found similar failure planes to Refs. [1, 4, 12-15, 18, 32]. As well as this, the use of reinforcing fibres in the grout were considered and results showed an improvement in axial strength by about 25%. Details of the testing procedures were given in another paper, but a qualitative summary of fatigue results showed that there was significant scatter for the number of cycles survived for different grout strengths, but no significant difference in S-N curves for compression-compression loading, which is in agreement with Refs. [8, 9, 32]. The effects of confinement are mentioned as being of importance, but no specific investigation has been undertaken.

In 2010, after the unexpected settlement of large diameter grouted connections for offshore WTGs had been reported, Schaumann, Bechtel and Lochte-Holgreven [34] covered axially loaded testing in more detail, with a look at the results of the ULS (ultimate limit state) axial tests that were used to derive the International Organisation for Standardisation (ISO) design codes in comparison to their own tests and recent work with the same conditions, but higher compressive strengths of grout (Figure 21).



Figure 57 Test results from static axial force capacity tests, curves for shear and sliding failure acc. to EN ISO 19902 (2007) (Cp = 1, K = 0.015) [34]

Limitations in the design codes are stated as mainly being the use of grout strengths greater than those for which the codes were validated for. With reference to previous work by the authors [36] showing that the use of linear damage accumulation for fatigue life was not appropriate and present the possible use of an energy approach, referenced to seminal works. There were only three samples for each grout strength, Figure 22, and so the conclusions drawn by the authors of stronger grout providing better fatigue performance are of limited significance without further work. The findings were in contradiction to those of Refs. [8, 9, 26, 30], who reported reduced or negligible change to fatigue performance for high-strength grouts.



Figure 58 S-N curves for small scaled specimens [34]

Schaumann et al. [36] presented an overview of the unexpected settlement of the grouted connection and investigated possible solutions, such as the use of reinforcement fibres and remedial solutions. The experimental investigation on the use of shear keys indicated an increase in axial strength of approximately six times that of a plain connection, which is similar to the findings of previous axial testing. The effect of the compressive strength of grout was also investigated and found to increase the axial strength of the connection. In both these tests there were only two samples, and so limited significance can be taken from these tests, although they agree with all previous historic test data. The test results on four samples did show an increase in strength with the fibre content, but the reduced slump was stated as making it impractical for offshore pumping, as void formation and blockages would be likely to occur and so their use was not recommended.

### 5 BENDING / GAPPING

Billington [5] presented results of tests carried out by BP (British Petroleum), investigating the strength improvements of composite steel grout tubes at annular joint connections, subject to both axial compressive-tensile and combined axial-bending loading conditions. The results indicate that if a bending load is applied to the brace, the axial capacity of the connection is reduced by up to 18% if the bending stress is less than the axial stress loading (Figure 23). As only two samples were tested for each load condition and for only four different conditions, the overall significance is limited, but testing work was still being undertaken at the time of publishing.



Figure 59 Interaction curve for combined bending and tension specimens [5]

These findings are in disagreement with those reported by Lamport [18], who showed no detrimental effects of combined axial and bending actions, but reported instead an increase of 14% in the capacity. However, details of the load combinations were not presented, making it difficult to assess the validity of such conclusions. Rotation was also reported as being pronounced, but there was no mention if this lead to gapping between the grout and pile or sleeve faces.

Sele and Kjeøy [13] showed that the static capacity of a grouted clamp after a significant moment has been applied shows no change in axial capacity. However, this is not comparable with other experiments, as the loads were not simultaneously applied.

Andersen and Petersen [38] presented the findings of experimentation undertaken in early 2000's to document the performance of grouted connections used at Horns Rev, the first large-scale offshore wind farm (160MW), 15km off the coast of Denmark, along with development of a FE model to reduce the need for expensive laboratory experimentation. Testing consisted of a 1:8 scale connection, tested in both ULS and FLS (Fatigue Limit State). The loading regime was not however explicitly stated in this paper. Experimental gapping was reported under ULS conditions, equivalent to 6.4mm at full-scale. The FE model showed reasonable agreement with the experimental testing, predicting a gap of 5mm at ULS, as shown in Figure 24, but with only one experimental setup and dimensions modelled, further work would be required for a more

robust validation. The testing however only investigated the influence of pure bending, not combined axial and bending actions, as would be typically experienced in an offshore WTG foundation. Additionally, no account was made for the influence of surface finish, and the environmental conditions of the tested sample were significantly different from those typically expected in the operational life of the foundation (Figure 24, right).



**Figure 60** Gap between pile and grout at load equivalent to extreme Load [38] Schaumann and Wilke [39] presented findings of a 1:6.25 scale four-point bending test. As in Ref. [38], gapping was reported, but in this case it was noted that gapping also occurred under FLS, which increased in size with the number of cycles before eventually stabilising (Figure 25).



Figure 61 Measured relative displacements between outside surfaces of pile and sleeve (negative values denote gap) at the 270° position [39]

Unlike previous tests, fracture of the grout under FLS was also reported, with radial cracks due to tensile hoop stress in the grout (Figure 26). This resulted in a reduction of the bending stiffness, indicating the importance of grout properties, which is in agreement with previous

findings, e.g. Refs. [3, 9, 13-15, 32]. This could also lead to a reduction of the axial capacity of the connection. The presence of hairline radial cracks after curing is also reported, indicating that shrinkage has occurred, which in turn reduces the confinement and then the axial capacity (see Refs. [4, 6, 13, 14, 21-25]). However, the influence of the cracks on the overall structural bending behaviour is stated to be small (about 5%), with justification by FE analysis, as long as the grout remains able to transfer the lateral stress. As with other bending tests undertaken, apart from Lamport et al. [19] and Billington [5], there is no mention of the effect on combined axial and bending behaviour.



#### Figure 62 Observed cracking [39]

Interestingly, Figure 27 shows the pre-2007 connection parameters for offshore WTG (grey bars) and the extent to which they are outside the limit range of validity for the NORSOK N-004, 2004, Rev 2 (purple) and Det Norske Veritas-DNV-OS-J101, 2004 (blue) codes for both compressive strength of the grout and slenderness of the pile.



Figure 63 Limits for the application of design rules selected offshore standards [39]

The authors also noted that results of a number of research projects will have to be incorporated in the future design guide for grouted joints, especially test results in the tension-compression regime and the influence of test frequencies on the fatigue strength in order to better understand fatigue response of high-strength grouts.

There is a mention of the disadvantage of shear keys because of stress concentrations due to the joint geometry and the fatigue strength of the weld being reduced in comparison to that of the base metal of the pile and transition piece/sleeve walls, and the effect that surface irregularities have in transmitting the shear forces between the grout and steel, as mentioned in previous axial testing (e.g. Refs. [4, 6, 13, 14]).

Schaumann, Wilke and Lochte-Holgreven [36] investigated the influence of shear keys on bending stiffness through the same experimental test set-up used by Schaumann and Wilke [39]. It was shown that global bending stiffness of the connection was increased up to 20% and the gapping between the steel and the grout reduced by over 50% by the inclusion of shear-keys. A FE model was also developed, which showed good agreement with only 7% error, but this was only validated for one geometrical configuration. A parametric model was developed to model the global dynamic behaviour of an Offshore WTG based on various grouted connection parameters including steel diameters and thicknesses, coefficient of friction and connection length. This highlighted that the key parameter for the connections' dynamic behaviour is connection length, but no validation was reported.

The paper by Klose et al. [35] takes the form of a review of current research by developing FE models calibrated to large scale testing, so to be representative of the actual slenderness found in currently used transition piece dimensions. Tests were carried out for both ULS and FLS, but only considered single loading conditions, i.e. axial or bending, not the combination of both as would be found in operation. The loading cases undertaken do however represent a good example of current and future technology at the time, with consideration of application to 2.5MW and 6MW wind energy converters. Important points noted in the paper were the uncertainty associated with the high-strength grout properties, and therefore the higher material safety factors when compared to steel design. Current design formulae for grouted connections having only been validated for grout with compressive strengths up to 80MPa, whereas current grouts in use can have strengths up to 210MPa. The fatigue assessment for the grout being based on concepts originally developed for concrete structures, as there was no explicit S-N curve for high-strength grouts, which was particularly worrying as a slight change in the gradient of this curve can lead to fatigue calculations that vary by a factor of 100, or even more due to its logarithmic formulation [17]. Finally the fatigue formulae used for design are based on load cycle numbers which are only a fraction of the number seen in the 25-year design life span of the turbine foundations.

Lotsberg et al. [40] provided a general summary of the work that has been undertaken by DNV to investigate the capacity of grouted connections following the unexpected settlements reported in 2009. They are in agreement with previous research of Billington [5, 6] and Lamport et al. [18], on the mechanism behind the capacity of plain connections referring to surface tolerance and roughness and radial stiffness, but state that a minimum surface tolerance should be included in fabrication standards to ensure these are mobilised. As minimum surface tolerances

would be impractical, it is recommended that these are not considered in design, but kept in mind when assessing test results. The authors are also in agreement with Refs. [5, 6, 15, 18] for the capacity of shear-key connections, stating that radial stiffness of the steel is important and load transfer between pile and transition piece is via formation of compression struts within the grout between the shear keys. Upon failure of cylindrical grouted samples, similar failure planes to Refs. [1, 4, 12, 13-15, 18, 32] were reported. Like Refs. [38-40], gapping was evident and the resulting ovalisation due to the reduced confinement of the higher D/t ratio leads to relative sliding between the grout and steel. Consideration has been given to scale effects, which was stated to overestimate the capacity of full-scale equivalents, showing agreement with the findings of Smith and Tebbett [12]. The scaling effect was minimised through using equivalent stiffness box tests for the scaled tests, which represent a segment of the connection at full scale and was validated through comparison of a small-scale 800mm diameter connection and the equivalent stiffness box section. Good agreement in terms of relative displacement was reported for the cylindrical test and the proposed analytical equation. However, this was only for one sample geometry and so little significance can be drawn from the results. The smallscale 800mm connection with shear keys was tested under a constant axial load with reversed bending moment and so was representative of the loading conditions experienced by offshore WTG grouted connections. The implication of representing environmental conditions, such as the presence of water was not considered in terms of the influence on friction coefficient, but was considered for the influence on S-N curve for grouted connections, where the in-air curve should be reduced by a factor of 0.8 [41].

Lotsberg [42] offers more detail on the derivation of the analytical equations for capacities of grouted connections under combined axial and bending moment loading conditions with and without shear keys based on the principles described in Ref. [40]. The work improved on the significance of the results of the author's previous work [40] by comparing the analytic predictions with the experimental results from six additional cylindrical bending tests with

varying grout strength, connection lengths and number of shear-keys undertaken at the University of Leibniz, as reported in Refs. [36, 39]. This saw very good agreement in all, but one of the tests, which was explained by the variation in experimental testing. However, the gaps in the validation when representing environmental conditions in operation still remain. As part of the conclusions of this paper, it stated that non shear-key connections could not be recommended due to the low long term axial capacity, in agreement with Ref. [39], and that contact pressure should be limited to minimise the potential of cracking of the grout and abrasive wear.

The second secon		<b>_</b>	Con	nection	Туре	Axial	Loading		Combined	Limits of Validity				
Author(s)	Year	Title	Plain	Shear	Pre-	Static	Fatigue	Bending	Loading	(D/t) <sub>n</sub>	(D/t) <sub>s</sub>	L/D	(D/t)a	f <sub>cu</sub> (MPa)
Billington and Lewis	1978	The Strength of Large Diameter Grouted		Key	stress		-		-	, <i>г</i> р				
binington and Lewis	1570	Connections	1	1		1				9.6 <x<45< td=""><td>18<x<120< td=""><td>1<x<12< td=""><td>7.6<x<26< td=""><td>2&lt; x&lt;110</td></x<26<></td></x<12<></td></x<120<></td></x<45<>	18 <x<120< td=""><td>1<x<12< td=""><td>7.6<x<26< td=""><td>2&lt; x&lt;110</td></x<26<></td></x<12<></td></x<120<>	1 <x<12< td=""><td>7.6<x<26< td=""><td>2&lt; x&lt;110</td></x<26<></td></x<12<>	7.6 <x<26< td=""><td>2&lt; x&lt;110</td></x<26<>	2< x<110
Billington	1980	Research into Composite Tubular												
		Construction for Offshore Jacket Structures	1	1		1		1	1	<40	<80	>2	7 <x<45< td=""><td>17<x<110< td=""></x<110<></td></x<45<>	17 <x<110< td=""></x<110<>
Billington and Tebbet	1980	The Basis for new Design Formulae for												
Billington and rebbet	1900	Grouted Jacket to Pile Connections	1	1		1	1	1	1	<40	<80	>2	7 <x<45< td=""><td>17<x<110< td=""></x<110<></td></x<45<>	17 <x<110< td=""></x<110<>
Karsan and Krahl	1984	New API Equation for Grouted Pile to								2454540	50cxc140	1/1/12	10-1-15	2< ×<110
		Structure Connections	ľ	ľ		ľ				2412.40	50474140	1.4.412	10 47 45	212110
Krahl and Karsan	1985	Axial Strength of grouted Pile-to-Sleeve	1	1		1				24 <x<40< td=""><td>50<x<140< td=""><td>1<x<12< td=""><td>10<x<45< td=""><td>2&lt; x&lt;110</td></x<45<></td></x<12<></td></x<140<></td></x<40<>	50 <x<140< td=""><td>1<x<12< td=""><td>10<x<45< td=""><td>2&lt; x&lt;110</td></x<45<></td></x<12<></td></x<140<>	1 <x<12< td=""><td>10<x<45< td=""><td>2&lt; x&lt;110</td></x<45<></td></x<12<>	10 <x<45< td=""><td>2&lt; x&lt;110</td></x<45<>	2< x<110
Tebbett and Billington	1985	Recent Developments in the Design of	,			,				24.1.10	50		4045	2
_		Grouted Connections	~	-		-	~			24 <x<40< td=""><td>50<x<140< td=""><td>&lt;2</td><td>10<x<45< td=""><td>2&lt; x&lt;110</td></x<45<></td></x<140<></td></x<40<>	50 <x<140< td=""><td>&lt;2</td><td>10<x<45< td=""><td>2&lt; x&lt;110</td></x<45<></td></x<140<>	<2	10 <x<45< td=""><td>2&lt; x&lt;110</td></x<45<>	2< x<110
Tebbett	1987	Recent Developments in the Design of	1	1		1	1			24 <x<40< td=""><td>50<x<140< td=""><td>&lt;3</td><td>10<x<45< td=""><td>2&lt; x&lt;110</td></x<45<></td></x<140<></td></x<40<>	50 <x<140< td=""><td>&lt;3</td><td>10<x<45< td=""><td>2&lt; x&lt;110</td></x<45<></td></x<140<>	<3	10 <x<45< td=""><td>2&lt; x&lt;110</td></x<45<>	2< x<110
Lamport Jirca and	1097	Grouted Connections												
Yura	1907	Grouted File-to-Sieeve Connection Tests		1		1		1	1	28.8	42	1 <x<2< td=""><td>20</td><td>27<x<73< td=""></x<73<></td></x<2<>	20	27 <x<73< td=""></x<73<>
Forsyth and Tebbett	1988	New Test data on the Strength of Grouted												
		Connections With Closely Spaced Weld		1		1				34.1	88.5	0.9	15.7	9 <x<61< td=""></x<61<>
Smith and Tehhett	1989	Beads New Data on Grouted Connections with												
Sinthand rebbett	1909	Large Grout Dimensions		1		1				18.66	36	2	6.5	12 <x<45< td=""></x<45<>
Sele and Kjeøy	1989	Background for the New Design Equations												
		for grouted Connections in the DNV Draft	1	1		1		1		10 <x<60< td=""><td>18<x<140< td=""><td>1<x<12< td=""><td>-</td><td>4&lt; x&lt; 90</td></x<12<></td></x<140<></td></x<60<>	18 <x<140< td=""><td>1<x<12< td=""><td>-</td><td>4&lt; x&lt; 90</td></x<12<></td></x<140<>	1 <x<12< td=""><td>-</td><td>4&lt; x&lt; 90</td></x<12<>	-	4< x< 90
Aritopang Elpashai	1000	Rules for Fixed Offshore Structure												
Dowling and Caroll	1990	Grouted Pile/Sleeve Connections		1		1				8 <x<15< td=""><td>72</td><td>1.1</td><td>14</td><td>50&lt; x&lt;64</td></x<15<>	72	1.1	14	50< x<64
Elnashai and	1991	Nonlinear Modelling of Weld-beaded		1		1				8cvc15	72	11	14	50c xc64
Aritenang		Composite Tubular Connections		· ·		~				0~~13	12	1.1	14	30< X<04
Lamport, Jirsa and	1991	Strength and Behaviour of Grouted Pile-to-		1		1				28.8	42	1 <x<2< td=""><td>20</td><td>27<x<73< td=""></x<73<></td></x<2<>	20	27 <x<73< td=""></x<73<>
Aritenang, Elnashai	1992	Analysis-based Design Equations for												
and Dowling		Composite Tubular Connections		1		~				8 <x<15< td=""><td>72</td><td>1.1</td><td>14</td><td>50&lt; x&lt;64</td></x<15<>	72	1.1	14	50< x<64
Sele and Skjolde	1993	Design Provisions for Offshore Grouted	1	1		1				<40	<80	>2	7 <x<45< td=""><td>17<x<110< td=""></x<110<></td></x<45<>	17 <x<110< td=""></x<110<>
Hanwood Billington	1006	Construction											-	
Buitrago. Sele and	1990	Provisions for the New ISO Standard for	1	1		1	1			20 <x<40< td=""><td>30<x<140< td=""><td>1<x<10< td=""><td>10<x<50< td=""><td>10<x<80< td=""></x<80<></td></x<50<></td></x<10<></td></x<140<></td></x<40<>	30 <x<140< td=""><td>1<x<10< td=""><td>10<x<50< td=""><td>10<x<80< td=""></x<80<></td></x<50<></td></x<10<></td></x<140<>	1 <x<10< td=""><td>10<x<50< td=""><td>10<x<80< td=""></x<80<></td></x<50<></td></x<10<>	10 <x<50< td=""><td>10<x<80< td=""></x<80<></td></x<50<>	10 <x<80< td=""></x<80<>
Sharp		Offshore Structures												
Dowling, Elnashai and	1983	A New Pressurised Grouted Connection for			1					17.7	34.4	2.2	7.8	70
Carroll Elnashai, Carroll and	1095	Steel Tubulars												
Dowling	1903	Connection for Offshore Construction	1		1	1				17 <x<50< td=""><td>32.6<x<36< td=""><td>2<x<20< td=""><td>7.6<x<17< td=""><td>-</td></x<17<></td></x<20<></td></x<36<></td></x<50<>	32.6 <x<36< td=""><td>2<x<20< td=""><td>7.6<x<17< td=""><td>-</td></x<17<></td></x<20<></td></x<36<>	2 <x<20< td=""><td>7.6<x<17< td=""><td>-</td></x<17<></td></x<20<>	7.6 <x<17< td=""><td>-</td></x<17<>	-
Elnashai, Carroll,	1986	Full Scale Testing and Analysis of												
Dowling and Billington		Prestressed grouted Pile/Platform	1		1	1				25 <x<50< td=""><td>30.3<x<36< td=""><td>1<x<2< td=""><td>17</td><td>-</td></x<2<></td></x<36<></td></x<50<>	30.3 <x<36< td=""><td>1<x<2< td=""><td>17</td><td>-</td></x<2<></td></x<36<>	1 <x<2< td=""><td>17</td><td>-</td></x<2<>	17	-
Grupdy and Kiu	1001	Connections Prostross Enhancement of Grouted												
Grandy and Rid	1551	Pile/Sleeve Connections	1	1	1	1				30 <x<37< td=""><td>43<x<100< td=""><td>1<x<4< td=""><td>13<x<27< td=""><td>-</td></x<27<></td></x<4<></td></x<100<></td></x<37<>	43 <x<100< td=""><td>1<x<4< td=""><td>13<x<27< td=""><td>-</td></x<27<></td></x<4<></td></x<100<>	1 <x<4< td=""><td>13<x<27< td=""><td>-</td></x<27<></td></x<4<>	13 <x<27< td=""><td>-</td></x<27<>	-
Boswell and D'Mello	1986	The Fatigue Strength of Grouted Repaired		1			1			37 /	73.2	1.0	22.25	7254593
		Tubular Members		ľ						52.4	75.2	1.0	22.25	72-2-55
Ingebrigsen, Løset and	1990	Fatigue Design and Overall Safety of Grouted Pile Sleeve Connections	1	1		1	1			24.5 <x<28.8< td=""><td>29.5<x<65.5< td=""><td>2.0</td><td>28.2<x<33.8< td=""><td>27<x<49< td=""></x<49<></td></x<33.8<></td></x<65.5<></td></x<28.8<>	29.5 <x<65.5< td=""><td>2.0</td><td>28.2<x<33.8< td=""><td>27<x<49< td=""></x<49<></td></x<33.8<></td></x<65.5<>	2.0	28.2 <x<33.8< td=""><td>27<x<49< td=""></x<49<></td></x<33.8<>	27 <x<49< td=""></x<49<>
Etterdal, Askheim,	2001	Strengthening of Offshore Steel												
Grigorian and Gladsø		Components using High-Strength Grout	1			1	1	1	1	N / A	25.7	1.0	N/A	140, 213
		Component Testing Analytical Methods												
Zhao, Grundy and Lee	2002	Grout Sleeve Connections under Large	1		1		1			26.0, 33.0	19.0,27.0	1.3 <x<4.3< td=""><td>7.5,20.1</td><td>_</td></x<4.3<>	7.5,20.1	_
Andersen and	2004	Structural Design of Grouted Connections in												
Petersen		Offshore Steel Monopile Foundations	1			1	1	1		72.0, 84.4	- , 80.0	1.5	- , 68.0	140, 114
Lohaus and Anders	2006	High-cycle Fatigue of Ultra-high												
		Performance Concrete – Fatigue Strength		1			1			5.5	14.3	1.5	5.2	135 <x<225< td=""></x<225<>
Zhao, Ghoiel, Grundy	2006	Behaviour of Grouted Sleeve Connections at												
and Han	2000	Elevated Temperatures	1		1		1			21.2	33.0	1.4 <x<3.1< td=""><td>7.6</td><td>-</td></x<3.1<>	7.6	-
Schaumann and Wilke	2006	Fatigue of Grouted Joint Connections	1			1	1			5 5 100	14 3 107 0	1315	46420	133
										5.5, 200	1 113, 10710	1.0, 1.0	110, 1210	155
Schaumann and Wilke	2007	Design of Large Diameter Hybrid	lign of p	iain sideo	monop	lle tound	ations for	offshore w	lig	1		1		1
		Connections Grouted with High	1			1	1	1		100.0	107.0	1.3	42.0	130
		Performance concrete												
Anders and Lohaus	2008	Optimized High Performance Concrete in		1		1	1			5.5	14.3	1.5	5.2	60 <x<170< td=""></x<170<>
Schaumann, Wilke	2008	Grout-Verbindungen von Monopile-												
and Lochte-	2000	Gründungsstrukturen - Trag- und	1	1				1		100.0	107.0	1.3	42.0	130
Holtgreven		Ermüdungsverhalten												
Klose, Faber,	2008	Grouted Connections for Offshore Wind	,							100.0	107.0		42.0	120
Schaumann and		Turbines	~	· ·				~		100.0	107.0	1.3	42.0	130
Locate nongreven		1		Unexp	ected se	ttlemen	l ts occurred	1				1		
Schaumann, Bechtel	2010	Fatigue Design for Prevailing Axially Loaded												
and Lochte-		Grouted Connections of Offshore Wind	1	1		1	1			4.8	14.3	1.5	5.2	70 <x<115< td=""></x<115<>
Holtgreven		Turbine Support Structures in Deeper												
Schaumann, Lochte-	2010	Durchrutschende Grout-Verbindungen in												
Holtgreven, Lohaus		OWEA – Tragverhalten, Instandsetzung und	1	1		1				5.5	14.3	1.5	5.2	60 <x<170< td=""></x<170<>
and Lindschulte	201-	Optimierung												
Lotsberg, Serednicki,	2013	Connections with Shear Keys in Offshore		1		1	1	1	1					
and Lervik		Structures		ľ		ľ				-	-	-	-	-
Lotsberg	2013	Structural Mechanics for Design of Grouted												
		Connections in Monopile Wind Turbine		1		1	1	1	1	-	-	-	-	-
Typical Offenere WTC	Growt	Structures		I	I		I	I	I			necificatio	 n	I
Monopile	GIUU		1	1	1	1	1	1	1	90	86	1.5	63	130
Jacket Pin Pile			1	1		1	1	1	1	48	57	2	8.5	130

#### **Table 2 Comparison of Grout Papers**

\* Relative values for a typical offshore WTG grouted connection have been provided to assist in appreciation of the scale and potential validity, of research and development.

## 6 SUMMARY AND CONCLUSIONS

Table 2 summarises the main areas investigated by each of the papers reviewed in the previous sections, including their limits of validity, which play a crucial role on the applicability of any lesson learnt and conclusion drawn in each of these contributions.

The review of the publicly available technical papers indicates that there has been significant development within the area of grouted connections over the years, which is often driven by the need for optimisation, but lately to improve the understanding of the reported insufficient axial capacities in the offshore wind industry. A large amount of work was initially carried out on the axial capacity of grouted connections. Once the increased capacity of shear-key connections was proved, the research focus shifted onto other aspects, including, effects such as pre-stress and damage accumulation, with an increasing number of challenging applications in the oil, gas and offshore wind industry.

From the papers published before early-age unexpected settlements started to be reported in large diameter grouted connections for offshore WTG foundations in mid-2009 this review paper demonstrates that:

- A reason for this unsatisfactory performance could be an inadequate understanding of the limits of existing design codes, particularly because of the complex composite interaction under a high number of multi-axial stress cycles.
- 2. The review has highlighted that limited testing was undertaken that was representative of the actual loading conditions experienced by offshore WTG structures, and even less representative of the confinement similar to current monopile structures, and no testing under representative environmental conditions.

- 3. Evidence was provided suggesting that in some circumstances it may not be conservative to assume that bending and axial loads do not interact, as was assumed in the offshore WTG grouted connection designs that have experienced settlements.
- 4. Indeed, previous testing had indicated that combined axial and bending may lead to reduction of the axial capacity and that gapping and relative displacements could then occur well below the ultimate load capacity.
- 5. Evidence was also provided that offshore filling procedures and curing under changing environmental conditions could influence the grout properties and therefore the resulting connection strength, but these factors have not yet been considered when evaluating more recent experimental testing and WTG installation procedures.
- 6. If this is combined with the evidence of grout powder formation under cyclic deformation and the presence of water due to the submerged nature of the connections, potentially reducing the steel-grout coefficient of friction, it could show how the capacities of the connection may not have been conservative.

For these reasons, the occurrence of such conditions and the insufficient axial capacity in the offshore WTG should not have been completely unexpected in non-shear-key connections.

Current research has indicated that some of these previous assumptions and understanding of the behaviour may have been incorrect for such plain-pipe connections, including scaling effects for certain parameters and operational conditions, but it is evident that further work is still required to fully understand all the influencing factors. It is also worth emphasising the key observation that, as far as the applicability of design guidelines is concerned, this can only be guaranteed up to the original limits of derivation, e.g. the experimentation carried out; which have been far exceeded by today's dimensions of offshore WTG structures. - Experimental Testing of Grouted Connections for Offshore Substructures: A Critical Review (Paper J1)

It is therefore recommended that experimental campaigns are undertaken that are representative of environmental conditions, such as the presence of water and environmentally degraded steel, to investigate the influence of these factors on the capacity and durability of the connections over their design life. There also appears to be a gap in knowledge on material behaviour such as the effect of multi-axial stress states on the fatigue life of different strength grouts, and therefore also this aspect should be investigated. Structural monitoring equipment installed on plain-pipe connections exhibiting signs of insufficient axial capacity has detected continual relative movement. Although in itself, very small relative movements are not a sign of insufficient axial capacity, the influence of this movement on key parameters such as coefficient of friction and grout/steel integrity should also be investigated under representative conditions, testing for which is underway. In this respect, a possible experimental setup has been recently proposed in Ref. [41], and results of this testing campaign are currently being collected.

Overall, this review shows that

- 1. Further testing should have been undertaken to understand the behaviour beyond those limits, as also recommended by many authors in the reviewed material.
- 2. It also highlights the importance of undertaking an inter-industry review of previous experience, as historic oil and gas testing provided evidence that there was a risk of overestimating axial capacity and so would have justified the cost of undertaking testing applicable to the conditions of offshore WTG grout connections.
- 3. The review also highlights a lack of information flow between researchers, design standard organisations, designers and operators, which should not just be one way, but provide feedback throughout the knowledge loop to ensure testing, standards and design are relevant to operation performance of the structure and design conditions.

Not only does this bring the benefit of validation of design assumptions potentially from structural condition monitoring, but ensures the cost of such engineering challenges are minimised. This virtuous circle would also offer potential for optimising the design of future installations, therefore reducing the cost of offshore wind energy.

For more information on access to underlying research materials please contact the authors.

# 7 ACKNOWLEDGEMENTS

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# APPENDIX D - EXPERIMENTAL INVESTIGATION ON THE DEVELOPMENT OF GROUT WEAR IN GROUTED CONNECTIONS FOR OFFSHORE WIND TURBINE GENERATORS (PAPER J2)

### **Full Reference**

Dallyn, P., El-Hamalawi, A., Palmeri, A. and Knight, R., 2016. Experimental Investigation on the Development of Grout Wear in Grouted Connections for Offshore Wind Turbine Generators. *Engineering Structures, Vol. 11, pp. 89-102* 

### Abstract

Relative displacements between grout and steel have been observed in grouted connections used for offshore wind turbine substructures, which appear to be linked to the unexpected settlements that have occurred in some OWFs. A literature review has highlighted a lack of understanding of the implications that this relative movement has on the grout wear. Experimentation has therefore been undertaken to determine the influence of various factors on the wear development, including compressive stress, displacement amplitude, surface roughness and the presence of water, looking at conditions typically experienced by offshore grouted connections. These experiments have indicated that wear of the steel and grout surfaces occur, even at low magnitude compressive stresses. The presence of water has the most significant impact on wear rate, being up to 18 times higher than for the equivalent dry condition. The presence of water can also significantly reduce the coefficient of friction to values lower than typically recommended for evaluation of grouted connections. These findings demonstrate that wear of the grouted connection is likely to occur over the life of this type of offshore structures and should therefore be considered when evaluating their integrity and assessing their behaviour.

**Keywords** – Grouted connection, Integrity assessment, Offshore structures, Wear development, Wind turbines

**Paper type** – Journal, Research

# **1 INTRODUCTION**

Grouted connections have extensively been used in the oil and gas industry for decades, but in recent years their use has proliferated in the offshore wind industry as an efficient method of joining the monopile (MP), embedded in the sea bed, to the transition piece (TP), which connects to the wind turbine generator (WTG) tower. In comparison to grouted connections used in the oil and gas platforms, offshore WTG connections have considerably lower radial stiffness with pile diameter to thickness ratios greater than 85, compared to 45 typically for oil and gas. However, lower length to diameter ratios exist with WTG connections, having generally 1.5 times pile diameter overlap compared to oil and gas connections with up to six times overlap, and a higher ratio of moment to axial loads with WTG grouted connection typically experiencing twice the moment to axial force compared to a quarter in oil and gas connections. They consist of a larger diameter circular section placed with overlap, of typically greater than 1.5 diameters, over a smaller diameter circular section, with the resultant annulus between the two sections filled with high strength grout. A typical offshore wind turbine foundation example is depicted in Figure 1.



Figure 1 Typical grouted connection general arrangement

The concept of a straight-sided sleeved grouted connection without shear keys had been used for over 650 installed monopiles for several commercial offshore European wind farms, representing around 60% of all installations in Europe [1] up until 2011, when the last of the pre-2010 designed foundations were installed. Following the announcement in 2009 of unexpected settlements of the TP relative to the MP in many offshore wind farms, existing grouted connections have required extensive monitoring assessments and remedial works. This has resulted in a shift away from straight-sided grouted connections without shear keys as the primary load transfer mechanism for offshore wind turbine structures.

Site inspections have shown unexpected settlements resulting in hard contact and load transfer between verticality jacking brackets and the top of the MP, which indicate that the connection has an insufficient axial capacity. The capacity initially develops mainly as a shear resistance due to the surface irregularities mobilising friction, but partially due to adhesion between the grout and the steel. As a result of the overturning moment at the base of the tower, however, an increased shear stress as well as compressive stress is created between the grout and the steel and, if the shear stress at that position exceeds the grout-steel friction resistance, a relative displacement between the steel and grout occurs. These relative displacements between the MP and the TP are often in excess of 1mm. They have been observed by subsequent structural condition monitoring, and appear to occur during changes in overturning moment caused by turbine cut-in and cut-off as well as variations in the wind direction and wind speed. The load transfer mechanism is illustrated in Figure 2.

For full axial capacity of the connection to be mobilised, a relative movement between the steel and grout is required and so small relative displacements should be expected. However, due to the cyclic nature of the loading experienced by the grouted connections, this repeated relative movement has led to degradation of the axial capacity, with a global downward movement in the TP relative to the MP. Importantly, this combination of potentially high compressive stress and relatively large displacements could result in wear at the grout-steel interaction surfaces.



#### Figure 2 Load transfer mechanism

The remedial solutions that have been proposed so far to address this problem typically consist of additional steel brackets and elastomeric bearings installed between the TP and MP.

However, the connection still relies on the grout to transfer the bending moments from the TP to the MP and therefore its integrity over the design lifespan of the foundation remains crucial.

Further, the potential for wear, and insufficient axial capacity in non-shear-keyed grouted connections is potentially worsened by water ingress that has been reported at some sites, although not considered in the original design<sup>1</sup>. A literature review undertaken by the authors [2] has revealed that there is a lack of detailed knowledge of the behaviour of the grouted connections, not only for the scale and size of actual structures, but also under the loading and environmental conditions of operation, particularly because the design principles in the existing standards up to 2011 were based on small-scale experimental testing from the oil and gas industry [3-6]. High-strength grout had also only been tested for compressive strength and single axis fatigue by manufacturers and limited testing had been undertaken for some of the conditions relevant for offshore wind turbine foundations [7-10].

Overall, the behaviour of the grout-steel interface over long-term service operation is not fully understood within industry and scientific community. Testing has been recently carried out [11-15], but some areas of concern, such as grout wear and environmental conditions, had not been investigated. As a result of the JIP on grouted connections DNV amended DNV-OS-J101 standard [16] to ensure wear failure mode is considered during design. In particular, if wear is occurring and the water ingress provides transportation for the grout material worn down; gaps are likely to form between the grout and outer face of the monopile. This may lead to a lack of fit and some significant dynamic effects on the structure as the tower oscillates. There is also a risk that the overall length of the grouted connection reduces, due to fracture of the unconfined grout at the top and bottom of the annulus, reducing the lever arm

<sup>&</sup>lt;sup>1</sup> Due to commercial sensitivity the sites cannot be named.

over which the loading is transferred from the TP to the MP, and therefore further increasing the stress in the grout and the steel. With the combination of all these factors, grout wear could be a significant issue for the long-term integrity of the foundation. It is therefore necessary to improve our understanding of the wear failure mode in such situations.

Motivated by the above considerations, this paper details the methodology and results of the experimental campaign undertaken to quantify the wear rates of representative samples of grouted connections under typical offshore conditions, which can then be used to get a more accurate assessment of the wear over the remaining design life of the foundation. In tribology, wear rate is typically defined as volume lost per unit normal load per distance of relative displacement [17]. However, within this research wear rate is quantified as the average change in the measure thickness of the sample per 100m of cumulative relative displacement ("walked distance") of the interaction surfaces. This definition has been used in order to present the results of our experimental campaign directly into the context of the real-world applications in offshore grouted connections. Cumulative relative displacement is defined as the sum of the relative axial displacements at the grout-steel interface of the sample.

## 2 METHODOLOGY

The aim of this work is to understand the grout wear failure mode under conditions typically experienced during life-time operation of offshore wind turbines. As a necessary first step to achieve this, an experimental protocol has been designed to simulate realistically such challenging conditions. The next two subsections detail the testing apparatus and provide a summary of methodology used for the experimentation.

### 2.1. Apparatus

The test rig shown in Figure 3 has been designed in order to allow large variable lateral compressive forces that are operationally representative on the grout/steel interface surfaces (shown by the green line in Figure 3b), while applying a dynamic vertical displacement to shear the sample along this interface. The vertical load capacity of the testing rig is 160kN, which allowed testing of samples 150×150mm in size up to maximum compressive stress level of 2.5MPa, consistent with those indicated by the design load cases, which have subsequently been validated by structural condition monitoring and design checks. The biaxial stress state produced by this experimental arrangement allowed the reproduction of load conditions which were critical to wear. The tri-axial stress state that is experienced due to ovalisation under bending in operational WTG grouted connections has not been included within this experimentation due to its relative insignificance when determining wear.

Dimensions (Figure 7) and material properties (Table 3) of the outer and inner steel plates along with the grout have been chosen to be the same as used in typical offshore wind turbine foundations, so that their thickness and stiffness are properly represented, and produced in the same manner as used offshore, resulting in similar surface properties, helping to reduce scaling effects. The test samples were grouted in accordance with the manufacture's recommended procedures and approval, with Densit Ducorit® S5 grout cast onto the inner and outer steel plates using a formwork to ensure containment and dimensions of the grout, shown in Figure 7. Shear-keys between the outer steel plates and grout ensured de-bonding occurred along the interface between the grout and the inner steel plate, highlighted by the green lines (Figure 3b). The samples were wrapped in damp cloths and cured for 48 hours before being de-moulded and placed in a curing tank for an additional 26 days. Five 100 x 100mm test cubes and one 150 x 300mm cylinder were also cast per sample mix to assess the compressive strength, elastic modulus and tensile strength of the grout in each test.



Figure 3 Experimental test arrangement; side (a), front (b) drawings and front picture of one of the samples ready for testing (c)

The compressive force applied to the samples could be varied by tightening the compression bolts (Figure 3). Strain gauges attached to these bolts were calibrated with a load cell before testing commenced, so the compressive stress on the grout can be derived for a given bolt strain and surface area of the grout-steel interaction surface. The compression bolts were retightened after each test phase to the required compressive load and the continual monitoring of the strain allowed for compensation during analysis of the data if loss of compression occurred due to wear. The bottom mounting brackets and beam (Figure 3) have been

designed to allow for the horizontal compressive force to transfer wholly from the lateral compression plates to the grouted sample, while still being able to transfer the vertical displacement of the actuator.

To represent the presence of sea water and the implications this may have on the grout-steel interaction, an equivalent solution has been drip-fed onto the top surface of the grout and allowed to drain through the grout/steel interface. The controller software of the testing machine also logged the axial displacements and load required to achieve the desired relative displacements between the grout and steel surface. A vertical Linear Variable Differential Transformer (LVDT) recorded the actual axial relative displacements between the grout and central steel plate surfaces. Four horizontal LVDTs provided periodic monitoring of the relative lateral displacement between the two outer plates, and therefore any change in thickness of the grout and steel materials if wear occurred was measured. The lateral compression bolt strain was recorded via the same data logger as the displacement sensors, to ensure sufficient numerical data acquisition of the interaction of the steel/grout surfaces. This resulted in 19 channels of data being logged at a frequency of 20Hz during testing.

In addition to the LVDT measurements, at the end of each phase of testing the accumulated evacuated wear debris was collected and weighed to provide additional information on the loss of material. This was done either through the collection of powder formed above and below the sample in the dry tests or through filtration of the recirculated solution in the wet test. Pre- and post-test Vernier caliper thickness measurements were also taken at 14 circumferential points of each part of the sample at the same points at the beginning and end of each samples test to determine the total loss of thickness of each of the constituent parts. Visual indicators were also acquired pre- and post-test to indicate the change in surface finish and therefore visual indication if wear is occurring.

## 2.2. Testing Procedure

Site investigations have shown many factors, including steel corrosion, water ingress, surface finish and confinement, can significantly vary between different wind farms, and in the same wind farms between different foundations [2]. To study how each of these factors affect the wear, different samples were prepared and tested, as summarized in Table 1. The levels of corrosion were based on exposure to salt spray for a period of one month resulting in rust grade C to BS EN ISO 8501-1:2007 [18], the Sa 2½ finish to BS EN ISO 8501-1:2007 was achieved by grit blasting of the inner steel plates.

Table 1 Sample identification, test matrix

SAMPLE	CHARACTERISTICS	REASONING
<b>S1</b>	Mill Scale, Dry, Unconfined	Test logging equipment & Rig
<b>S2</b>	Mill Scale, Dry, Unconfined	Influence of controller amplitude and frequency
<b>S3</b>	Sa 2.5, Dry, Non-corroded, Confined	Influence of surface finish and higher loads
<b>S4</b>	Sa 2.5, Wet, Corroded, Confined	Influence of water and corrosion (Industry situation)
<b>S5</b>	Sa 2.5, Wet, Non-Corroded, Confined	Influence of corrosion
<b>S6</b>	Sa 2.5, Dry, Corroded, Confined	Influence of corrosion (Industry situation/maintained water tightness)

The amplitude of the test cycles was determined from available structural condition monitoring data collected from a typical offshore wind turbine grouted connection affected by insufficient axial capacity. Based on analysis of this data, it was found that maximum peak-to-peak amplitude of relative displacement between the top of the MP and TP of around 1.2mm was detected, providing an upper limit of the relative displacement between grout and steel chosen as the primary amplitude for testing. These large-magnitude relative displacements were detected on a daily basis during winter periods, the frequency of which was dependent on the wind conditions. The cycle frequency of 0.3Hz was determined as the typical natural frequency of the structure being monitored and to allow satisfactory behaviour of the samples without excessive heat generation.

Each sample was subjected to a minimum of seven phases of 8,000 cycles at 1.2mm peak-topeak axial amplitude for each 0.5MPa horizontal compressive stress increment, until either the grout failed under shear or the load capacity of the rig was reached. The number of cycles per phase and number of phases per load increment were chosen to ensure sufficient wear would occur to be detectible, allowing wear rates to be determined.

# **3 QUALITATIVE OBSERVATIONS**

#### 3.1. Adhesive Strength

Samples were de-moulded after 48 hours and qualitative observations were recorded on the amount of force required to separate the grout from the inner steel plates. In all but the non-corroded Sa2<sup>1</sup>/<sub>2</sub> cases, the 48 hour adhesive strength developed at the de-moulding stage was not sufficient to hold the samples together. As reported in Table 2, however, a noticeable difference was seen, depending on the surface finish of the inner steel plate, with high representing forced separation, medium - separation under self-weight and low - separation while de-moulding.

Table 2 Sample adhesion on de-moulding

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Sample	Surface Finish	Adhesion
<b>S</b> 1	Mill Scale	Medium
S2	Mill Scale	Medium
<b>S</b> 3	Sa 2.5, Non-corroded	High
S4	Sa 2.5, Corroded	Low
<b>S</b> 5	Sa 2.5, Non-corroded	High
<b>S</b> 6	Sa 2.5, Corroded	Low

Interestingly, the shot-blasted un-corroded surface finish shows the highest adhesive strength with the shot-blasted corroded surface being the lowest. This experimental observation (which however may need further investigations) may have some direct practical implications. Indeed, given design code equations used to derive the axial capacity of the connection are based on experimental testing which had non-corroded, shot blasted finishes, they did not account for a corroded surface that would be found offshore. Since the steel-169

grout adhesive strength is part of the total axial bond strength of a connection, the peak bond strength at first slip of a grouted connection in offshore conditions is likely to be lower than expected. These findings align with those of [3], who stated that grouted connections with shot-blasted finishes have a higher axial capacity than those with mill scale finishes. This is likely down to the increased surface roughness of the shot blasted finish and the partially rusty surfaces potentially providing a weaker surface layer. It should be noted that for offshore wind turbine grouted connections where significant bending moments are transferred and ovalisations occur the resultant tri-axial stress state.

## 3.2. Surface Finish

SAMPLEPRE-TESTPOST-TESTS3<br/>Steel Surface<br/>(Dry)Image: S3<br/>Steel Surface<br/>(Dry)Image: S3<br/>Steel Surface<br/>(Dry)Image: S3<br/>Steel Surface<br/>S3<br/>Grout Surface<br/>(Dry)

The pictures in Figure 4 show the surface finish for some of the samples pre- and post-test.



### Figure 4 Pre and post-test surface finishes

From Figure 4 it can be seen by the brown and dark grey areas that in the dry tests, a layer of compressed powder forms on the majority of both the steel and grout surfaces, with greatest thickness, up to 0.4mm, at the centre of the surfaces. At the top and bottom edges the inner steel plate shows signs of scoring and polishing, indicating that wear is occurring on the steel surface. Moreover an area of metallic sheen can be seen on the grout surface where the layer of compressed powder has been abrading the steel surface.



Figure 5 Post-test grout (left) and steel (right) surfaces of a wet sample

In the wet test samples, it is evident that there are no signs of compressed powder on either the grout or steel surfaces, with all wear debris appearing to have been evacuated. The interaction surfaces of both the grout and steel were also polished, evident by the reflection of light in the photos and emphasised in Figure 5, with no signs of the pre-test surface corrosion or finish, indicating that the wetting of the sample is resulting in a fine grinding-like paste being formed by the steel and grout particles that are quickly evacuated.

# 4 QUANTITATIVE RESULTS

## 4.1. Grout Properties

The density and mechanical properties of the grout for each mix have been recorded, and a summary of the results is presented in Table 3. The 28-day compressive strength of the Densit Ducorit® S5 grout has been calculated based on the average strength of five 100mm cubes crushed to the BS EN 12390-3:2009 [19] standard. The tensile strength and elastic modulus are based on tensile splitting and compressive moduli of 150 x 300mm cylinders to BS EN 12390-6:2009 [20] and BS 1881-121:1983 [21], respectively.

Table 3	Table 3 Densit Ducorit® S5 material properties of test samples							
Sample	Density (kg/m <sup>3</sup> )	Coefficient of Variation (%)	28 Day Compressive Strength (f <sub>c</sub> ) (MPa)	Coefficient of Variation (%)	Tensile Splitting Strength (MPa)	Elastic Modulus (GPa)		
1 and 2	2340 (-3.0%)	0.6	124.7 (0.1%)	1.3	-	-		
3	2420 (0.3%)	0.5	124.0 (-0.5%)	2.6	7.9 (3.6%)	54.50 (1.4%)		
4	2454 (1.7%)	0.7	110.1 (-11.7%)	5.6	7.7 (1.0%)	53.04 (-1.3%)		
5	2433 (0.9%)	0.2	129.5 (3.9%)	2.5	7.5 (-1.6%)	53.90 (0.3%)		
6	2413 (0.0%)	0.2	134.8 (8.2%)	6.1	7.4 (-3.0%)	53.47 (-0.5%)		
Average	2412	124.6 7.6						

4.2. Average Coefficient of Friction

Based on the maximum axial force recorded for the given displacement amplitude and compressive force, the resultant coefficient of friction has been calculated based on Equation 1 for the various surface finishes and environmental conditions.

$$\mu = \frac{F}{2R} \tag{1}$$

where *F* is the axial force, *R* is the compressive force,  $\mu$  is the coefficient of friction and a factor of 2 is included to account for the two interaction surfaces of the test arrangement.





It can be seen from Figure 6 that for all the dry samples (S2, S3 and S6), the coefficient of friction tends to increase over the first 50,000 cycles and then reduces with the total number of cycles experienced tending to the original value. The initial increase could be due to the tolerances of casting and aligning the samples resulting in non-parallel surfaces that over the 173

first 50,000 cycles undergo lapping, removing irregularities and increasing the contact area. The subsequent decrease in coefficient of friction could be due to the grout powder formed, evident in the post sample photos of section 3.2, forming a sufficiently thick shear layer, where there is particle rotation rather than abrasion in certain areas. The formation of this shear layer is also the likely cause for the results showing limited influence of surface finish on the coefficient of friction between the samples, with values generally being within the variance of the results.

For the wet samples (S4-5), there is an initial decrease in the coefficient of friction, which then tends to re-gain the original value. This reduction is likely due to the abrasion of the surfaces resulting in a smooth surface finish, as shown in Figure 4, as well as the presence of the water acting as a lubricant reducing the friction between the surfaces. The coefficient of friction results are summarised in Table 4.

Sample ID	Characteristics	Mean Coefficient of Friction	Standard Deviation	Coefficient of Variation (%)
S2	Mill Scale, Dry, Unconfined	1.02	0.14	13.7
<b>S</b> 3	Sa 2.5, Dry, Non-corroded,	1.00	0.13	13.2
	Confined			
<b>S</b> 4	Sa 2.5, Wet, Corroded,	0.76	0.08	10.1
	Confined	0.70	0.08	
<b>S</b> 5	Sa 2.5, Wet, Non-corroded,	0.70	0.08	12.4
	Confined	0.70	0.08	
<b>S</b> 6	Sa 2.5, Dry, Corroded,	0.07	0.10	10.1
	Confined	0.97	0.10	10.1

**Table 4 Coefficient of friction statistics** 

The dry samples indicate coefficients of friction that are above the maximum value of 0.4 to be used in the design of grouted connections in DNV-OS-J101, Section 9 [19], even after 350,000 cycles. The equation presented in Ref. [22] for the interface shear strength due to friction ( $\tau_{kf}$ ) for plain-pipe connections is shown in Equation 1.

$$\tau_{kf} = \frac{\pi E}{K} \frac{\delta}{R_p} \tag{1}$$

Where  $\pi$  is the coefficient of friction, *E* is the modulus of elasticity of steel,  $R_p$  is the pile outer diameter,  $\delta$  is the height of surface irregularities (0.00037 $R_p$ ) and *K* is a stiffness factor which is dependent on the geometry of the connection and elastic modulus of the grout and steel.

However, the presence of the sea-water equivalent solution in samples S4 and S5 tests clearly shows significantly lower minimum values for coefficient of friction, with values typically 40% lower than the equivalent dry samples for the majority of the test. This indicates that the presence of water will have a stronger influence on the long-term axial capacity of a plain-sided grouted connection than surface finish and presence of corrosion. Although the wet tests indicate that the assumed value of coefficient of friction for design of 0.4 is still conservative for the capacity of grouted connections, the 0.6 recommended for evaluation or modelling of grouted connections [22] may not be conservative. To the best of the authors' knowledge, this is the first time in which the importance of this factor has been experimentally demonstrated.

Based on these test results, it can be recommended that in the design and evaluation of submerged grouted connections a lower value of coefficient of friction is assumed, unless it can be guaranteed that water will not enter at the grout/steel interaction surface over the design life-span of the connection.

### 4.3. Ultimate failure

Samples S1 and S2 were cast and tested without confinement brackets on the top and bottom of the sample (Figure 7, left), so to be representative of the very edge of the grouted connection, and both samples have failed with fracture of the grout at compressive stress of 2 and 2.5MPa respectively. Samples S3 and onwards were cast and tested with confinement

(Figure 7, right), so to allow higher stress levels and represent grout further down the length of the grouted connection, and none of these samples fractured during the cyclic tests. Although further investigations may be needed to confirm these findings, the sharp difference in the performance of confined and unconfined grout seems to indicate that for this type of grout, compressive stress within the very top and bottom of the connection should be limited to less than 2MPa while in shear, if potential fracture and spalling of the grout is to be avoided under high cycle loading. However, it should be noted that the bi-axial stress state of the experimental arrangement is not representative of the tri-axial stress conditions experienced by offshore WTG grouted connections during operation, which could lead to reduced fatigue capacity. Fracture and spalling could result in a reduced connection length, increasing the stress in the remaining grout for a given load.



Figure 7 Picture of unconfined grout (left) and confined grout (right)

## 4.4. Loss in Thickness

To quantify the loss in thickness of the samples three alternative methods were used, namely: Vernier caliper; weight of evacuated material; LVDT. The results of these measurements are reported and discussed in what follows.

### 4.4.1 Vernier Caliper Measurements

Pre- and post-test thickness measurements of the samples were taken using a Vernier caliper at 14 circumferential points around the left steel and grout(L), right steel and grout (R) and middle steel (M) part of the samples S3 to S6, whose results are listed in Table 5.

Sample ID	Total Walked distance (m)	Loss in Thickness (mm)	Coefficient of Variation (%)
S3R		-0.04	0.2
S3L		0.11	0.1
S3M		0.16	0.2
S3 Total	1223	0.23	
S4R		0.45	0.2
S4L		0.39	0.2
S4M		0.37	0.1
S4 Total	402	1.21	
S5R		0.41	0.4
S5L		0.38	0.1
S5M		0.31	0.1
S5 Total	490	1.09	
S6R		0.03	0.2
S6L		0.07	0.4
S6M		-0.12	0.1
S6 Total	625	-0.02	

Table 5 Loss in thickness based on Vernier caliper measurements

It can be seen that for the two dry, samples (S3 and S6) there is minimal loss in thickness and even a slight increase in some parts of the samples. This aligns with the qualitative visual findings reported in Section 3.2, which showed a build-up of a layer of grout powder.

On the contrary, the two wet samples, S4 and S5, show a considerably higher loss of thickness. For instance, comparing samples S3 and S4 reveals that the loss of thickness occurring in the wet sample (S4) was almost six times higher, even though the walked distance for the dry sample (S3) was three times longer. This further confirms the importance of the presence of sea water on the amount of material loss and, based on the Vernier caliper results of our tests, the wet wear rate can be up to 18 times higher than the corresponding dry value.

From the breakdown in loss of thicknesses for the S4 and S5 samples it can be seen that the steel surfaces (S4M and S5M) show about 17% less loss than the grout surfaces. Given that both sides of the inner steel plate interact with the single grout surface on each of the outer parts of the sample, as shown in Figure 3, the loss in thickness of a single steel surface should be considered half of the value in Table 5. For the dry samples (S3 and S6) a similar ratio is seen although the uneven build-up of the grout powder layer increases the variance in the results.

### 4.4.2. Evacuated Material Weight

The material evacuated from the samples has been weighed in order to indirectly determine the loss in thickness of the samples. This has been based on the assumptions that: the steel-to-grout ratio in the collected material is the same as the final measured wear ratio indicated by the Vernier caliper measurements for samples S4 and S5; density of the grout as in Table 3; density of the steel of 7,850kg/m<sup>3</sup>.

For samples S3 to S6, the values of total loss in thickness indicated by the weight of material are all within 0.2mm of the corresponding values from the Vernier caliper measurements. The slight overestimate of this method is probably due to some of the debris coming from the rig attachments, whose steel powder was also collected. It should also be noted here that the assumption on the wear ratio between the grout and steel can have a considerable effect on the equivalent loss in thickness, due to a large difference in density between the grout and steel  $(2,420 \text{kg/m}^3 \text{ to } 7,850 \text{kg/m}^3)$ .



Figure 8 Loss in thickness based on weight of evacuated material for different conditions 179

Figure 8 shows that the rate of material loss (indicated by the gradient of the graphs) initially increases with the compressive load and appears to reach a peak before dropping off in all cases except S5. This is more clearly shown in Table 6, where it can be seen that above 2MPa of compressive stress there is no real increase in wear rate and up to this point there is an approximately linear increase in material loss. The table also shows that the wear rate is around 9 to 15 times higher for the wet samples for the same surface conditions and compressive stress.

Sample ID	Test Condition	Compressive Stress (MPa)	Loss in Thickness per 100m Walked Distance(mm)	Coefficient of Variation (%)
		0.7	0.019	12.1
	Draw Co 21/	1.1	0.027	20.8
<b>S</b> 3	Dry, Sa 272,	1.5	0.035	40.3
	Non-corroded	1.9	0.047	34.0
		2.2	0.050	13.5
		0.6	0.134	31.4
		0.7	0.215	27.1
S 4	Wet, Sa 21/2,	1.1	0.422	27.3
54	Corroded	1.5	0.440	17.4
		1.9	0.548	16.2
		2.4	0.405	28.8
	Wet, Sa 2½,	0.6	0.160	16.2
		0.7	0.254	24.9
85		1.1	0.073	15.7
35	Non-corroded	1.5	0.261	28.0
		1.9	0.349	15.0
		2.4	0.686	22.6
		0.6	0.004	21.0
		0.8	0.017	17.3
9.4	Dry, Sa 21/2.	1.2	0.014	19.7
86	Corroded	1.5	0.020	15.1
		2.0	0.034	16.2
		2.4	0.015	17.1

 Table 6 Comparison of wear rates based on evacuated debris weight

The greater coefficient of variation for the dry samples indicates slightly worse behaviour, which aligns with greater lateral movements being observed during the tests. This appeared to be caused by localised build-up of compressed wear debris in off-centre locations, creating high spots (evident in Figure 4) with greater resistance to axial movement, causing slight rotational movement around these points.

In terms of surface finish, once again there are marginal differences in wear rates between the corroded and un-corroded samples, with the corroded samples showing slightly lower rates, but within the variance of the data. The limited difference could be justified with the first few cycles of testing, in which the influence of the surface finish is significant, until a powder layer develops and the surface becomes smoothed through abrasion.

### 4.4.3. Linear Variable Displacement Transformer (LVDT) Measurements

The horizontal LVDTs provided measurements of the loss in thickness of the sample interface surfaces throughout testing of each sample. Based on the loss in thickness after each phase, the graphs within Figure 9 have been plotted.





Figure 9 LVDT measurement of loss of thickness

It can be observed that material loss is approximately linear with distance walked with very little difference in gradient for the different compressive stresses on either the dry (S3 and S6) or wet (S4 and S5) shot-blasted finishes, which shows reasonable agreement with the findings of the evacuated material weights. It is however evident that under the wet conditions (S4 and S5), loss in thickness is considerably more, with around 2-3 three times the rate of loss of thickness of the dry test for the various compressive stresses. This is clearly shown in Table 7.

Again, the influence of surface finish on wear rates appears to be a minimal. It is also evident that the total loss in thickness of the samples is up 1.4 times higher than when the weight of evacuated material method is applied to the wet samples, and up to 13 times for the dry samples.

Table 7 Comparison of wear rates based on LVDT measurements						
Sample ID	Test Condition	Compressive Stress (MPa)	Loss in Thickness per 100m walked distance(mm)	Coefficient of Variation (%)		
		0.7	0.26	3.0		
	$D_{mr}$ So 21/	1.1	0.21	2.6		
<b>S</b> 3	Dry, Sa 2 <sup>4</sup> 2,	1.5	0.24	4.6		
	Non-corroued	1.9	0.35	1.7		
		2.2	0.26	3.3		
		0.6	0.45	7.2		
		0.7	0.50	0.9		
S 4	Wet, Sa 21/2,	1.1	0.55	3.7		
54	Corroded	1.5	0.45	8.3		
		1.9	0.72	3.9		
		2.4	0.60	4.9		
		0.6	0.33	2.9		
		0.7	0.47	7.0		
S.5	Wet, Sa 2 <sup>1</sup> /2,	1.1	0.38	1.3		
33	Non-corroded	1.5	0.43	10.2		
		1.9	0.32	16.9		
		2.4	0.64	1.3		
		0.6	0.23	2.7		
		0.8	0.12	3.8		
9.6	Dry, Sa 21/2,	1.2	0.20	6.0		
86	Corroded	1.5	0.38	3.8		
		2.0	0.16	4.0		
		2.4	0.24	6.8		

### 4.4.4. Discussion

The values obtained for total loss of thickness based on the weight of evacuated material generally show good agreement with the Vernier caliper measurements, particularly for the wet tests (see Tables 8 and 9 and Figure 10), and these values would appear as the most reliable to assess the wear rate.

	Test	Compressive Stress (MPa)	Loss in Thickness per 100m walked distance(mm)		
Sample ID	Test Condition		LVDT	Weight of evacuated material	
		0.7	0.26	0.019	
	Draw Co 21/	1.1	0.21	0.027	
<b>S</b> 3	Dry, Sa 2 <sup>4</sup> 2,	1.5	0.24	0.035	
	Non-corroued	1.9	0.35	0.047	
		2.2	0.26	0.050	
		0.6	0.45	0.13	
		0.7	0.50	0.22	
<b>S</b> 4	Wet, Sa 21/2,	1.1	0.55	0.42	
34	Corroded	1.5	0.45	0.44	
		1.9	0.72	0.55	
		2.4	0.60	0.41	
	Wet, Sa 2½,	0.6	0.33	0.16	
		0.7	0.47	0.25	
85		1.1	0.38	0.073	
35	Non-corroded	1.5	0.43	0.26	
		1.9	0.32	0.35	
		2.4	0.64	0.69	
		0.6	0.23	0.004	
		0.8	0.12	0.017	
0.6	Drv. Sa 2½.	1.2	0.20	0.014	
50	Corroded	1.5	0.38	0.020	
		2.0	0.16	0.034	
		2.4	0.24	0.015	

 Table 8 Comparison of wear rates

The horizontal LVDT measurements on the contrary appear to considerably overestimate wear for dry conditions, with the room temperature fluctuations of  $\pm 6^{\circ}$ C recorded in the laboratory over one month not justifying such drift in the data, as typical temperature curves for the sensors would allow for an error which is less than 3%.

	TEST CONDITION	TOTAL	TOTAL LOSS IN THICKNESS (mm)		
Sample ID		WALKED DISTANCE (m)	LVDT	Weight of Evacuated Material	Vernier Caliper
<b>S</b> 3	Dry, Sa 2 <sup>1</sup> / <sub>2</sub> , Non-corroded	1236	2.71	0.20	0.23
S4	Wet, Sa 2 <sup>1</sup> / <sub>2</sub> , Corroded	402	1.85	1.31	1.21
S5	Wet, Sa 2½, Non-corroded	490	2.13	1.64	1.09
<b>S</b> 6	Dry, Sa 2 <sup>1</sup> / <sub>2</sub> , Corroded	625	1.38	0.11	-0.02

Table 9 Comparison of total loss in thickness

The slightly more variable lateral motion of the dry samples, mentioned previously, may have also contributed to increase the measured horizontal displacements. The effect of creep of the compression bolts has also been taken into account, based on the average of non-zero values of strain recorded at the end of testing when the compression is removed, and therefore no tensile load is acting on the bolts, and this effect is less than 3.5%.

Taking the gradient of the cumulative relative displacement and loss in thickness results to derive the wear rate may lead to an inaccurate estimation for some of the load levels, where steady state wear was not achieved within the first few test phases of that load increment. An example of this is shown in the results for the 0.7MPa compressive stress data derived from the weight of evacuated material method for S5 (Figure 8).



Figure 10 Wear rates based on LVDT and weight of evacuated material methods

From all the forms of measurements collected as part of our investigations, it is clear that the most significant factor on the wear rate is the presence of water, which at best doubles the wear rate (LVDT method), but at worst could be up to 18 times higher (Vernier caliper measurement) than for dry conditions. In comparison, the surface conditions of the steel appear to have only a marginal influence, although they are likely to affect initial bond strength of the grout/steel joint.

A possible explanation of such a significant impact of the water presence on the wear rate is the possibility of the wear debris to be evacuated from the interaction surfaces, which is therefore deemed to be critical to the loss in thickness. For dry connections, i.e. when the transition piece is not submerged, this can only happen at the very top or bottom of the connection, which will then exhibit more loss in thickness. However, for wet connections there is likely to be transportation of the wear debris over the whole length of the connection, and so more significant loss in thickness will occur over the entire length.

It is worth stressing here that, due to the experimental setup, the values of wear rates presented in the study have been obtained with two interaction surfaces because of the nature of the test setup. In the offshore connection, on the contrary, relative displacements tend only to occur at the inner steel-grout surface, due to the smaller area and therefore higher shear stresses. There is therefore only one interaction surface in the actual connections, so the wear rates indicated here should be halved if used to determine the expected wear for typical compressive stresses and environmental conditions of offshore foundations.

# 5 CONCLUSIONS

Unexpected settlements have occurred in large-diameter grouted connections for offshore wind turbines, which can mainly be attributed to: insufficient understanding of the limits and basis of previously used design codes; complex material interaction for a composite

connection that experiences a high number of stress cycles; environmental conditions that had not been fully accounted for.

A clear gap has been found in the existing technical literature on the long-term behaviour of plain grouted connections under loading and environmental conditions truly representative of such challenging applications. The testing programme documented in this paper has therefore been undertaken to determine the effects of these conditions on the grout wear failure mode, and to provide input to the foundation integrity assessments of existing foundations.

Our experimentation indicates that wear of the grout and steel interaction surfaces occurs even at low compressive stresses. Wear rates are influenced by the compressive stress with increasing rates up to 2MPa, after which rates appear to plateau or reduce. The presence of water in the grouted connection, which was not originally considered in design, has been shown to have a significant detrimental effect on wear rate, as it provides a transportation medium for the wear debris. The results show a minimum of twice the wear rate if water is present, but can possibly go up to 18 times, although greater repetition is required to provide significance to these indications. The presence of water also reduces the value of coefficient of friction below levels currently recommended for the evaluation of grouted connections. The influence of displacement amplitude has been investigated at the lower compressive stresses, but no correlation with wear rate was shown, while the influence of the surface finish of the steel is minimal in comparison to the presence of water. Under high cyclic dynamic loading, resulting in relative movements between the grout and steel, it is evident from the testing that fracture of unconfined grout is likely to occur above 2MPa compressive stress under shear loading. If this results in spalling, the connection length is likely to reduce, increasing the stress in the remaining grout for a given load and exacerbating the problem. The research presented also represents a small but necessary part of the puzzle in regards to the understanding of grouted connections behaviour and occurring mechanisms.

It is evident that wear has potential to influence the structural behaviour of the grouted connection through loss in thickness of the steel and grout, resulting in lack of fit. The influence of the wear should be assessed to determine the likely change in structural response over the life-time of the structure to ensure the natural frequency remains within acceptable limits.

This testing has not been validated by large-scale tests or a full scale WTG grouted connection due to the expense and availability of experimental testing at these scales and the long time period required to detect a significant loss in thickness offshore. Further work is therefore required due to the variation in compressive stresses within the grout over the length and circumference of the grouted connection for a given wind speed, direction and turbine operation. In addition, given these characteristics will vary over the design life of the connection, the amount of wear will inevitably vary around the diameter and across the length The value of normal compressive stress experienced will also be of the connection. influenced by the geometry and therefore radial stiffness of the grouted connection. The values of compressive stress and associated wear rates presented in this research can therefore not just be applied to a single location within the grouted connection, but must account for the variation in loading and geometry of the grouted connection. These aspects of the problem will therefore be assessed as part of further research, with the wear rates determined from the experimentation applied to monitored displacements and compressive stresses within a typical grouted connection, so as to predict the wear experienced over its design life.

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# APPENDIX E - PREDICTION OF WEAR IN GROUTED CONNECTIONS FOR OFFSHORE WIND TURBINE GENERATORS (PAPER J3)

### **Full Reference**

Dallyn, P., El-Hamalawi, A., Palmeri, A. and Knight, R., 2015. **Prediction of Wear in** Grouted Connections for Offshore Wind Turbine Generators. *Structures, (Under review)* 

#### Abstract

Insufficient axial capacity of large-diameter plain-pipe grouted connections has recently been observed in offshore wind turbine substructures across Europe. Aimed at understanding the implications of this phenomenon, a campaign of structural condition monitoring was undertaken. The measurements showed significant axial displacements occurring between the transition piece and the monopile, which in turn resulted in a considerable amount of wear. Given the existing lack of technical data on the implications that this relative movement has on the wear of grouted connections, a methodology was developed to quantify the likely risk to the foundation integrity of the wear failure mode. The proposed approach consists of a numerical model which applies the wear rate derived from previous experimental testing to the conditions experienced by typical offshore grouted connections, as indicated by the wind turbine generators' supervisory control and data acquisition systems. The output of this model showed that, for a representative sample of the wind farm substructures analysed as a case study, the accumulated lifetime wear would be minimal in the majority of the grouted connection, i.e. less than 0.4mm over 75% of the connection, but a much greater loss in thickness, of the order of 4mm, was predicted at the very top and bottom of the connection. This assessment is based on the assumptions that no significant changes occur in the surrounding environmental conditions and that the degradation in the grouted connection does not significantly affect the dynamic response of the foundation structure over its life span. Importantly, these assumptions may affect the model's predictions in terms of cumulated wear over time, not in terms of identifying the individual connections to be prioritised when performing remedial work, which is indeed the main intended use of the model.

## Keywords

Grouted connection, Integrity assessment, Offshore structures, Wear development, Wind turbines.

Paper type – Journal, Research

## **1 INTRODUCTION**

The concept of grouted connections has been extensively used in the oil and gas industry, and more recently in the offshore wind energy sector [1], as they offer an efficient solution to join the piles driven into the seabed with the top-side substructure, while accommodating significant installation tolerances [2]. Unfortunately, since late 2009, unexpected settlements of the transition piece (TP) relative to the monopile (MP) have been reported in many of the plain-pipe grouted connections for offshore wind turbine generator (WTG) constructed pre-2010, with designs similar to that one shown in Figure 1 [3]. This is due to a combination of: 1) incorrect scaling of properties from small and large samples tested in the labs to full scale connections, e.g. the size of surface finish irregularities [3]; 2) use of design equations beyond limits of validity established by the experimental data, without sufficient justification [4]; 3) use of design equations for connections experiencing operational conditions significantly different from those experimentally simulated when such equations were derived, e.g. submerged and corroded connections [4]. This has resulted in extensive and expensive remedial works to relieve such grouted connections from phenomena of damage accumulation.





Structural condition monitoring (SCM) and site investigations from late 2010 onwards have shown significant relative vertical displacements occurring between the inner surface of the grout annulus and outer steel surface of the MP. These displacements, in combination with relatively large compressive stresses and the presence of water, have been shown by experimental testing [5] to result in loss of thickness in both the steel and the grout at the grout-steel interaction surfaces. Remedial solutions so far proposed, such as the addition of elastomeric bearings and axial support structures, still rely on the grout transferring the bending moment from the TP to the MP. The grout's integrity over the design lifetime of the foundation thus remains crucial. Understanding the potential loss in thickness over the lifetime of grouted connection in offshore WTG applications is therefore an important stage in assessing the potential for their failure.

A recent review of technical literature [4] has highlighted a lack of research in this specific area, with historical experimental testing on grouted connections being predominantly related to the axial capacity [6-11]. More recent investigations on lateral loading of grouted connections under conditions relevant to their current use in the wind industry have been

undertaken [12-18], taking into account the influence of water and relative stiffness of the MP and TP, however the influence of abrasive wear has not been considered. Experimental testing of wear has been investigated in relation to low strength concretes and non-confined systems [19-23], but the mechanisms used are not representative if the conditions experienced in offshore WTG grouted connections. Therefore, to quantify the rate of potential wear, a novel experimental testing procedure was developed and undertaken for conditions representative of those experienced by offshore WTG grouted connections during their service life. The load transfer mechanism experienced by the grouted connections during operation that the experimentation replicates is shown in Figure 2. The procedure and results of this experimental campaign are presented in [5] and briefly summarised in the next section. Importantly, it has been demonstrated that the potential for wear could be significantly increased by the presence of water, which provides a transportation medium for the wear debris.



Figure 2 Simplified WTG structure bending and grouted connection load transfer

Having experimentally obtained representative wear rates for the actual conditions of the connections, a numerical model is presented in this paper, which applies these values for different compressive stresses and relative displacements, to predict the accumulated amount of wear over the operational lifetime of representative offshore WTGs. This model could then be used to predict wear in future for similar structures. Given that the environmental conditions will vary both spatially, from location to location, and temporally, due to the changes in direction and intensity of the wind loads [24]-[31], these variables were included as inputs in the proposed model. One way of achieving this goal would have been to deploy an extensive SCM campaign on each individual foundation, to directly measure the relative displacements and compressive stresses within the grout over a representative period of time. However, the cost would have been prohibitively expensive, and for this reason existing data provided by the WTG supervisory control and data acquisition (SCADA) system has been used instead. The available data has been transformed into relative displacements and compressive stresses within the grouted connections through relationships derived from SCM deployed on two WTG substructures and transfer functions based on structural analysis of the substructure.

Fed with this information, the proposed model provides an indication of the distribution of wear around the circumference and depth of the grouted connection, which will help to determine if further remediation work is going to be required within the remaining operational life of the WTG. It also provides a simple yet robust methodology for future designers and current operators of grouted connections to check designs against wear failure.

This paper will briefly describe the development and calibration of the proposed numerical model and the experimentation used to derive wear rates. It will also present the results for a representative case study, showing the distribution of wear around the depth and

circumference of the grouted connection, along with the variability of wear across a typical wind farm.

### 2 WEAR EXPERIMENTATION

To determine wear rates that are representative for the loading and environmental conditions experienced during the operational lifetime of offshore WTG structures, the experimental setup shown in Figure 3 was developed. This test arrangement allows for varying compressive stresses to be applied to the grout, while the inner steel plate undergoes cyclic relative displacements, resulting in two interaction surfaces represented by the green lines in Figure 2. Based on the analysis of condition monitoring data, from a typical offshore WTG grouted connection affected by insufficient axial capacity, a maximum peak-to-peak amplitude of about 1.2mm was detected for the relative displacement between the top of the MP and TP, which was then chosen as the reference amplitude for the cyclic relative displacements between grout and steel in the test samples. These large-magnitude relative displacements were detected on a daily basis during winter periods, the frequency of which was dependent on the wind conditions. The cycle frequency of 0.3Hz was determined as the typical natural frequency of the structure being monitored and to allow satisfactory behaviour of the samples without excessive heat generation. The vertical load capacity of the testing rig was 160kN, which allowed testing samples with a grout-steel interface of 150×150mm, up to maximum compressive stress level of 2.5MPa; the latter value is consistent with the calculations reported in the design of the WTG and has subsequently been validated by structural condition monitoring. Top and bottom grout confinement brackets were included in the test samples to enable increased compressive stresses without the grout fracturing, which is better representative of grout deeper within the grouted connection. Repetition of the wet, corroded and confined conditions was also undertaken to improve the significance of results.



(c)

Figure 3 Experimental test arrangement; side (a), front (b) drawings and front picture of one of the samples ready for testing (c)

The compressive force applied to the samples could be varied by tightening the compression bolts (Figure 3). Strain gauges attached to these bolts were calibrated with a load cell before testing commenced, so the compressive stress on the grout could be derived for a given bolt strain and surface area of the grout-steel interaction surface. The compression bolts were retightened after each test phase to the required compressive load and the continual monitoring of the strain allowed for compensation during the analysis of the data if loss of compression occurred due to wear. The bottom mounting brackets and beam (Figure 3) have been designed to allow for the full transfer of the horizontal compressive from the lateral compression plates to the grouted sample, while still being able to transfer the vertical displacement of the actuator.

To account for the presence of sea water and the implications this may have on the grout-steel interaction, an equivalent solution has been drip-fed onto the top surface of the grout and allowed to drain through the grout-steel interface. The controller software of the testing machine also logged the axial displacements and load required to achieve the desired relative displacements between the grout and steel surface. A vertical Linear Variable Differential Transformer (LVDT) recorded the axial relative displacements between the grout and central steel plate surfaces. Four horizontal LVDTs monitored the relative lateral displacement between the two outer plates, and therefore any change in thickness of the grout and steel materials if wear occurred was measured. The lateral compression bolt strain was recorded via the same data logger as the displacement sensors. This resulted in 19 channels of data being logged at a frequency of 20Hz during testing.

Eight samples were tested in order to be representative of the various surface and environmental conditions that the grouted connections would be subjected to (details are shown in Table 1). The steel samples were shot-blasted to a Sa 2<sup>1</sup>/<sub>2</sub> finish to BS EN ISO 8501-1:2007 [32], as required during grouted connection fabrication. Each sample was subjected to a minimum of seven phases of 8,000 cycles at 1.2mm peak-to-peak axial amplitude for each 0.5MPa horizontal compressive stress increment, until either the grout failed under shear or the load capacity of the rig was reached. The number of cycles per phase and number of phases per load increment were chosen to ensure sufficient wear would occur to be detectible, allowing wear rates to be determined.

To investigate implications of material properties, the measured grout compressive strength, tensile strength and elastic modulus were correlated to the wear rates for given sample conditions. Details of the experimental procedures and results can be found in [5].

Table 1 Test matrix						
Sample	Characteristics	Reasoning				
<b>S</b> 1	Mill scale, Dry, Unconfined	Test of logging equipment & Rig				
<b>S</b> 2	Mill scale, Dry, Unconfined	Test influence of controller amplitude and				
		frequency				
<b>S</b> 3	Sa 2.5, Dry, Non corroded, Confined	Influence of surface finish and higher loads				
<b>S</b> 4	Sa 2.5, Wet, Non corroded, Confined	Influence of water presence				
<b>S</b> 5	Sa 2.5, Wet, Corroded, Confined	Influence of corrosion				
<b>S</b> 6	Sa 2.5, Dry, Corroded, Confined	Influence of corrosion and water presence				
<b>S</b> 7	Repeat S5	Improve significance of results/determine				
<b>S</b> 8	Repeat S5	influence of grout material properties				

The resultant wear rates derived from the experimental testing for the wet and dry samples are shown in Figure 4. The weight of ejected material presented in Figure 4 represents one of the methods used to determine the loss in thickness. This involved collecting the wear debris ejected from the interaction surfaces of each sample and determining the equivalent loss in thickness based on the debris mass and density.



**Figure 4 Wear rates derived from experimental testing based on weight of evacuated material** It is worth noting here that, for the purposes of developing the numerical model of wear in grouted connections, the experimental wear rates have been halved because the samples were tested with two steel-grout interaction surfaces, resulting in twice the amount of wear for a given cumulative relative displacement when compared to a grouted connection.

## **3 NUMERICAL MODEL**

To determine wear distribution around the circumference and along the depth of the grouted connection, inputs from the SCADA system were used in the form of 10-minute average data intervals of wind speed, wind direction and power production from two full-scale offshore WTG substructures. The two WTGs are identified as 'H4' and 'K1' within an offshore wind farm comprising 60 units; K1 is peripheral in the prominent wind direction, while H4 has a more internal position (see Figure 16). The model uses these time series, along with relationships derived from the analysis of data recorded by SCM and SCADA systems, to determine the values of displacements and normal compressive stresses within the grouted connection. Appropriate transfer functions were derived, as the SCM was originally installed to understand the fatigue implications on the primary steel as a result of the unexpected load transfer between the installation jacking brackets and the top of the MP caused by the settlement of the TP, not the abrasive wear. For this reason some of the monitored points were not relevant to measure the normal compressive stresses in the grouted connection. The architecture of the model is shown in Figure 5 and summarised below. Importantly, the model assumes that the SCM data used to derive the relationships is representative of the structural response of the grouted connection over its whole 20-year lifetime, and the same has been assumed for the wind speed and direction used as inputs to the model.



Figure 5 Grout wear numerical model architecture

### 3.1. Inputs

To develop the relationships between the environmental inputs determined from the WTG's SCADA system and the structural response determined by the substructure's SCM system (Figure 6), the relevant time series were correlated for conditions of constant wind direction during either power or non-power generation of the WTG. Details of the systems are provided in Table 2, while the layouts of the SCM are shown in Figure 6.

Table 2 Data acquisition systems details										
System		SCM			SCADA					
Acquisition frequency		20Hz			20Hz					
Stored frequency		20Hz			0.0016Hz					
Periods of data analysis		01/2012 - 01/2013								
Instrumentation location		Top of grouted connection			Hub height					
		W-SX-Y-Z			Wind speed					
	W	Foundation	H4	- AP	Active					
		location	K1		power					
Instance station abbrassisticus	s X	Bracket	1-6	WD	Wind					
Instrumentation abbreviations		location			direction					
	Y	Gauge type	SGA - Axial strain RD - Radial displacement VD - Vertical displacement							
	Ζ	Orientation	V - Vertical	-						



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#### Figure 6 SCM layout

Before any relationship could be derived, initial screening of both data sets was undertaken to determine a suitable period for the analysis in terms of data quality and to minimise any drift effect due to settlement of the TP relative to the MP. As a result, a three-month time series from January to March 2012 of SCADA and SCM data were synchronized and analysed.

#### 3.2. Relationships

Data recorded for low wind speed (less than 1m/s) were initially used to correct vertical strain readings (SGA-V) (Figure 7) to account for any offset caused by datum setting of the SCM. These strains were measured on the inside wall of the TP 1.5m above the top of the MP (this

location is shown in Figure 6 and indicated by the K1-*S1*-6-SGA-V label). The correlation relationships between wind speed, strain and displacement, shown in Figures 7 and 8, were then derived. To achieve this, data was extracted and correlated on wind speed with vertical strain (SGA-V) and vertical relative displacement between the top of MP and TP (VD) for periods of wind direction aligned with the instrumentation orientation  $\pm 1^{\circ}$  for both power (Figure 8) and non-power generation events (Figure 9). Trend curves (plotted with solid lines in Figures 8 and 9) were derived to provide a conservative output, and were therefore consistently placed close to the upper bound of the data scatter. The scatter shown in the strain and displacement responses can be explained by: i) the 10-minute averaging; ii) the influence of wave loading (superimposed to the wind loading); iii) nacelle wind direction misalignment. The latter, based on the analysis of the SCADA data, was shown to be +/- 5°. Information on wave height and direction was not incorporated as it is not a major source of loading during WTG operation for the water depth and site location of the case-study structures.



**Figure 7 TP vertical strain responses (SGA-V) of K1 at low wind speeds to derive correction factors** Two trend curves have been used to model the structural responses of interest (strains and displacements) as the wind speed increases. During power generation (Figure 8), the turbine blades begin to rotate as soon as the wind speeds exceeds the cut-in value (marked by the end

of the flat curves at about 2.5m/s); further increases in the wind speed correspond to increases in the power output of the WTG, until a maximum value is reached (marked by the peak values of strains and displacements in Figures 8(a) and 8(b), respectively, occurring between 10 and 15m/s); for higher wind speeds, the pitch angle of the turbine blades is increased, so to maintain a constant power output, until the cut-out wind speed is reached (at 25m/s). As can be clearly seen in Figure 8, when the blades become increasingly pitched, the trust generated by the blades decreases, meaning that the resultant force on the nacelle reduces, and therefore lower strains and lower displacements are observed within the support structure and the grouted connection, respectively. During non-power generation, on the contrary, i.e. when there is a fault with the WTG, the correlations between wind speed, strain and displacement are significantly weaker, with more scatter. However, as the WTG availability is typically 95%, the influence of any inaccuracy associated with the non-power correlation will be very limited on the output of the proposed wear model over the 20-year design life of the structure. One explanation for the higher scatter during non-power events could be the lack of aerodynamic damping from the turbine, resulting in the wave and current loading having a more significant impact on the structure.



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Figure 864 Relationships derived from correlations of wind speed for (a) strain and (b) vertical displacement for power generation events with a constant wind direction



Figure 9 Relationships derived from correlations of wind speed for (a) strain and (b) vertical displacement for non-power generation events and a constant wind direction

### 3.3. Transfer function

In order to transform the vertical strain in the TP wall, at d=1.5m above the grouted connection (where the strain gauges have been positioned), to a compressive stress within the grouted connection, a transfer function was derived based on: i) the specifications of a typical

offshore WTG structure; ii) the application of the simple theory of beams in bending; and iii) the expression for the normal stress within a grouted connection (Eq. (1)) suggested by the Det Norske Veritas (DNV) joint industry project on the capacity of grouted connections [3]:

$$p_{nom} = \frac{3\pi M}{(\pi + 3\mu)R_p L_g^2 + 3\pi\mu R_p^2 L_g};$$
 (1)

where:  $p_{\text{nom}}$  is the normal stress within the grouted connection;  $R_p=2.15$ m is the radius of the monopile;  $L_g$  is the grout length;  $\mu=0.7$  is the coefficient of friction between steel and grout; and *M* is the applied bending moment.

Eq. (1) has been derived by rearranging the expression for the total moment capacity of a grouted connection, considering the vertical and horizontal shear stresses and the contact pressure (see Figure 10):

$$\boldsymbol{M} = \boldsymbol{M}_{\mathbf{p}} + \boldsymbol{M}_{\boldsymbol{\mu}\boldsymbol{h}} + \boldsymbol{M}_{\boldsymbol{\mu}\boldsymbol{v}} \,. \tag{2}$$

The moment due to the contact pressure,  $M_p$ , is derived from the integration of the pressure distributed along arc *bcd* in Figure 10:

$$M_{\rm p} = p_{\rm nom} \, \frac{R_{\rm p} \, L_{\rm g}^2}{3};$$
 (3)

the moment due to horizontal friction,  $M_{\mu h}$ , is derived from integrating the pressure within the green dashed line from *a* to *c*:

$$M_{\mu h} = \mu p_{nom} \frac{R_p L_g^2}{\pi}; \qquad (4)$$

finally, the moment due to vertical friction,  $M_{\mu\nu}$ , is derived from integrating the pressure outside the green dashed line from *a* to *c*.:

$$M_{\mu\nu} = \mu \, p_{nom} \, R_p^2 \, L_g \tag{5}$$

As the interface shear strength due to surface irregularities is considered to be negligible for large diameter grouted connections as in the monopile foundation case, this has not be included in these

calculations, but should be considered for jacket pile grouted connections [3].

In Eq. (1), let's now consider  $p_{nom}=p_{nom,tg}$  and  $M=M_{tg}$  as the values at the top of the grout ('tg') of nominal pressure and bending moment. The latter can be directly related to the value of the bending moment experienced by the transition piece ('tp') at d=1.5m above the top of the grout. Indeed, neglecting the effect of any distributed load along the height of the structure, the ratio of  $M_{tg}$  and  $M_{tp,d}$  is fixed and depends on the length of the structure above the grouted connection to the zero moment point at hub height of the WTG (H=79.4m) and the distance d=1.5m above the grouted connection:

$$M_{\text{tp},d} = \left(\frac{H-d}{H}\right) M_{\text{tg}} = \left(\frac{79.4 - 1.5}{79.4}\right) M_{\text{tg}} = 0.9811 M_{\text{tg}}.$$
 (6)

On the other hand,  $M_{tp,d}$  can be related to the bending strain  $\boldsymbol{\varepsilon}_d$  measured at the inside wall:

$$M_{\rm tp,d} = \frac{E_{\rm s} I_{\rm tp} \,\mathcal{E}_d}{R_{\rm tp,i}},\tag{7}$$

where  $E_s$ = 210GPa is the Young's modulus of the steel and  $I_{tp}$  is the second moment of area for a hollow circular cross section:

$$I_{\rm tp} = \frac{\pi}{4} \Big( R_{\rm tp,o}^4 - R_{\rm tp,i}^4 \Big) = 1.777 \,\mathrm{m}^4 \,, \tag{8}$$

 $R_{tp,o}=2.27$  m and  $R_{tp,i}=2.22$  m being the outer ('o') and inner ('i') radius of the TP.



Figure 65 Indicative pressure distribution of a grouted connection

Substituting now Eq. (7) into Eq. (6), and the result into Eq. (1), gives:

$$p_{nom,tg} = \frac{3 \pi E_s I_{tp}}{R_{tp,i} R_p L_g \frac{H-d}{H} [(\pi + 3\mu)L_g + 3\mu\pi R_p]} \varepsilon_d$$
<sup>(9)</sup>

To determine the (nominal) distribution of pressure vertically throughout the grouted connection, the following expression can be used:

$$p_{\text{nom},y} = \frac{M_y}{M_{\text{tg}}} p_{\text{nom},\text{tg}} = \frac{H+y}{H} p_{\text{nom},\text{tg}}, \qquad (10)$$

where *y* is the depth below the top of the grouted connection.

To account for the discontinuity of the end of the connection, a Stress Concentration Factor (SCF) has been included, so that:

$$p_{\text{local}} = SCF \, p_{\text{nom}}. \tag{11}$$

Based on DNV-OS-J101 B105 [33]:

$$SCF = 1 + 0.025 \left(\frac{R}{t}\right)^{1.5}$$
, (12)

where *R* is the radius of the TP ( $R_{tp,o}$ ) and *t* is the thickness of the TP ( $t_{tp}$ ).

This relationship along with outputs from the FEM design of the grouted connection was then used to derive the vertical relationship of radial stress with depth of connection as shown in Figure 11.



Figure 66 Variation of radial stress with depth of the grouted connection including SCF

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A cosine distribution for the stress magnitude was assumed along the circumference of the grouted connection due to the simple bending of the tube, which shows a good agreement when compared to the circumferential variation of strains detected by the SCM (see Figure 12). The location of theses strain gauge reading can be seen in Figure 6. A similar circumferential distribution was also assumed for the vertical displacements at the top of the grouted connection; these also showed good agreement with the SCM data.



Figure 67 Variation of TP vertical strain with circumferential location

#### 3.4 Wear Output

The next stage in the development of the proposed prediction model was to correlate the compressive stress in the grout with the accumulated wear, which required the following steps:

 For any given wind speed and direction, a resulting compressive stress was calculated (using Eqs. (9) and (12) and the relationships derived from Figures 8(a) and 9(a)). This then allowed the associated wear rate to be derived, considering a linear interpolation of the experimental values obtained for the S5 sample of the experimental campaign (corroded/confined/wet, as found in offshore WTG grouted connections), shown in Figure 4. The relative displacement between the TP and MP was calculated for the same wind speed and direction through the relationships derived from Figures 8(b) and 9(b).

- 2. The computed 10-minute average wear rates for each 0.1m depth and 10° circumferential location of the grouted connection were then averaged over the entire inputted data period and multiplied by the sum of the accumulated displacements.
- 3. The magnitude of the total wear predicted by the model over this period was calibrated against the SCM K1-S2-RD detected wear for the same period, in order to account for the high-frequency structural response that would not be detected by 10-minute average data.

#### 6.1 3.4 CALIBRATION

The calibration was achieved through analysis of the SCM data (five horizontal displacement gauges (RD) located between the top of the MP and the TP), for periods with constant strain of  $\pm 5$  microstrain and wind direction  $\pm 2.5^{\circ}$ , over 3 three-month periods; any detected change in displacement indicates potential wear, as the loading conditions should also be constant. An example of the wear detected is shown by the gradient of the data points in Figure 13.



Figure 68 Example of detected loss in thickness between MP and TP based on change in horizontal displacement readings over time for constant strain and wind direction for K1-S2-HD
Analysis of the SCM data showed an increase in horizontal displacements recorded after prolonged periods of the instrumented location being on the downwind side of the grouted connection. This highlighted that the initial assumption of zero wear if the area of the grouted

connection was not in contact to be incorrect and in fact that deposition of the wear debris was potentially occurring. A deposition rate was therefore applied to events when the normal compressive stress was less than zero, i.e. in tension, to account for this deposition and relative increase in thickness during these periods. The magnitude of 1/6<sup>th</sup> of the wear rate was derived for the latter, to ensure that the predicted wear on the predominantly downwind side of the connection (60°) matched the SCM (K1-S2-RD) detected wear. This resulted in the wear distributions shown in Figure 14.



Figure 69 Example of model outputs of wear distribution, a) calibration of model predicted wear at top of the grouted connection with SCM detected wear, b) comparison of wind and wear distributions and c) wear distribution over entire grouted connection

Figure 14(a) shows the good agreement between the model's predicted wear and the SCM detected wear for the two K1-RD locations. Figure 14b shows that the model's circumferential distribution of wear aligns well with the wind distribution for the inputted period.

In order to determine the robustness and accuracy of the model, the predicted wear was checked against the SCM detected wear for another two periods on both the H4 and K1 locations. This indicated that the initial calibration against the K1-S2-RD had resulted in over-prediction of the wear for all but one of the periods analysed. However, a model output calibrated against the detected wear from K1-S6-RD, resulted in an equivalent of 0.38Hz structural response, matching the magnitude of detected wear for all of the periods, examples of which are shown in Figure 15. This frequency coincides well with the first mode of the substructure's natural frequency response at 0.29-0.33Hz, and the blade passing (1P and 3P) driving frequencies from the WTG at 0.14-0.31Hz and 0.43-0.92Hz.



Figure 70 Comparison of numerical wear model outputs with SCM indicated wear for (a) K1 05-06/2012 and (b) H4 01-03/2012

This model is specific to the substructures used in this case study given it has been derived for the statistical relationships between the structures response and environmental data at a specific wind farm and for the specifications of the specific structures.

Since the model has been derived and validated from a single case study, the robustness of its predictions would be limited if directly applied to other sites without calibration for the change in site conditions. However, the devised methodology is not specific to the site and can be applied to other situations by incorporating site-specific relationships and specifications. If this could be done for a range of different sites, this would allow developing a tool that could be utilised directly across the offshore wind industry.

## 4 **RESULTS**

The wind data analysed for the WTGs H4 and K1 has demonstrated that, as expected, wind speed and direction can vary significantly within the wind farm. The wind data for 11 of the 60 WTGs were therefore inputted into the model to provide an indication of the spatial variation of wear around the foundations in the wind farm. The locations have been chosen to offer a good distribution across the wind farm, so that each foundation for which the expected wear has not been calculated is adjacent to at least one foundation for which the calculation has been done. This resulted in the locations shown in Figure 17 (namely turbine locations A1, C3, C6, D2, E5, F1, F7, G3 and J5, in addition to H4 and K1).

In order to determine the 20-year prediction of wear for each structure, three years of historical SCADA data from 2010 to 2012 were inputted into the model in the form of 10-minute average wind speed, wind direction and power generation for each location. The model output of wear distribution for this period is then scaled by the proportion of input data availability, as required by the sparse nature of some of the data periods, with up to 5 out of 12 months where no data was recorded. The computed wear was then scaled to a 20-year equivalent wear, based on the assumption that the three-year wind characteristics and three-

month structural response characteristics used to derive the model are representative over the entire 20-year design life. This assumption appears reasonable for the wind loading, as there is a good comparison between the wind speed and direction distribution measured over the three years of available data and those used in the design of the substructure (see Figure 16).





not indicated any significant change in the natural frequencies of the structures as a result of accumulated damage in the grouted connection.

The results of the numerical model's wear for each of the 11 selected locations are shown in Figure 17. These results are based on the value of outputted wear from the model calibrated for a 0.38Hz response, which was chosen given the accuracy it showed against the SCM detected wear.



Figure 72 Spatial variation of 20 years of accumulated wear for 11 foundations at a typical offshore wind farm

From Figure 17, it can be seen that the maximum wear at the very top of the grouted connections was found on the predominant wind direction and is in the order of 3.5 to 4.8mm, with around 1mm on the opposite side indicating a possible gap of dynamic movement of 5 to 6mm after 20 years of operation. However, it should be noted that, due to the incorporated SCF for the nominal contact stress, the magnitude of this wear reduces to a tenth of the value indicated at the top of the connection within 700mm of depth (as shown in Figure 11c), leaving three quarters of the connection barely affected by wear. Given the significance of the impact of the applied SCF, it is recommended that a more detailed analysis is undertaken to verify the accuracy of the linear approximation used in this study.

One outlier in the results appears to be C3, which shows much less wear than any other foundation. Upon investigation of the wind and power data for this period, it was found that C3 has slightly less wind data than other cases, while power data were incorrect when compared to the other WTGs, which has clearly affected the model's predictions.

## 5 CONCLUSION

Aimed at better understanding and quantifying the long-term implications of the grout wear failure in large-diameter plain-pipe grouted connections, a numerical model has been developed to predict the accumulation of wear in the grouted connections for the actual load conditions experienced over a given period. The proposed model has been derived and calibrated based on limited site SCM (structural condition monitoring) and SCADA (supervisory control and data acquisition) data of two operational WTG (wind turbine generator) substructures afflicted by wear of the grouted connections, along with experimentally-derived wear rates. Good agreement has been found between the model's predicted and SCM detected wear for the majority of instrumented locations and periods screened. Although the model is specific to the structural characteristics of the substructures used as a case study, the devised methodology can be applied to other wind farms by replacing the case-specific details, limiting the need for expensive SCM.

Through statistical analysis of the inputted period of wind data and comparison with historical wind statistics for the site, the model outputted wear for a 3-year period has been scaled to provide a prediction of the expected wear over the 20-year design life of the plant. By modelling a selection of WTG substructure locations across a typical wind farm, it has been shown that wear accumulation in the order of about 4mm at the very top of the predominant wind direction side of the grouted connection could be expected, assuming no significant change in the environmental or structural conditions. However, over the majority of the length of the connection, the wear is of the order of 0.4mm due to a large reduction in the stress concentration factor (SCF), which sharply increases the stress at the ends of the connection. Given the significance of the impact of the applied SCF, it is recommended that more detailed analyses are undertaken to verify the accuracy of the linear approximation used in this study. It is evident that wear has the potential to influence the structural behaviour of the grouted connections through loss in thickness of the steel and grout, resulting in lack of fit. The influence of the wear should be assessed to determine the likely change in the dynamic response over the lifetime of the structure, e.g. to ensure the natural frequency remains within acceptable limits. The findings of this work can then be used to indicate if further remedial work may be required to specific offshore substructures and, if so, allow for better planning, which would help in minimising the cost of the interventions. It will also allow for a reduction in site inspections by determining which structures are likely to experience the most significant loading conditions for wear.

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# APPENDIX F - COST-EFFECTIVE PARABOLIC TROUGH FOUNDATIONS FOR CONCENTRATED SOLAR POWER PLANTS (PAPER C1)

### **Full Reference**

Dallyn, P., El-Hamalawi, A., Palmeri, A., 2012. Cost-Effective Parabolic Trough Foundations for Concentrated Solar Power Plants. *Proceedings of the 3rd International Conference on Engineering, Project and Production Management 2012*, 10-11 September, Brighton, UK, pp 227-238.

### Abstract

As part of the continuing requirement for a broad sustainable energy mix, substantial investment is being currently made into renewable energy, including concentrated solar power (CSP). To improve the financial viability of this under-developed technology, research into optimising design is underway in order to reduce the large capital costs associated with CSP plants. At present around 30% of capital costs of a 50MW farm are in the solar field (Vallentine et al., 2009), due to the large number of solar collector assemblies (SCAs) required, and therefore there is a large potential to reduce the overall cost by optimising the design of the SCAs and their foundations.

The challenge arises in reducing material and weight, and in simplifying manufacture and assembly, while maintaining the structural rigidity, as the efficiency of the collectors is highly dependent on the optical accuracy. This can be potentially compromised by the wind loading, which is predicted to be the most significant source of optical error (Kolb and Diver, 2008), particularly for flexible systems. A literature review has highlighted that considerable effort has been put into optimising SCA design, but the foundations have been neglected. As well as this there is limited understanding of the loads experienced by the foundations of SCAs.

This paper will present the current level of understanding of SCA loading and how this knowledge can be used to derive the variation in foundation design across the solar field and

- Cost-Effective Parabolic Trough Foundations for Concentrated Solar Power Plants (Paper C1)

therefore allow optimisation of foundation design, highlighting potential capital savings that can be made in a typical 50MW CSP plant.

## Keywords

Concentrated Solar Power, Foundations, Optimisation, Parabolic Trough

**Paper type** – Conference, Research

# **1 INTRODUCTION**

There are four main commercial types of concentrating solar power (CSP) systems that use the sun's energy as a heat source through concentrating the sunlight onto solar receivers. The four types are parabolic trough, dish/engine, linear Fresnel reflector and power tower, as shown in Fig. 1, with a comparison given in Table 1.



Figure 1: Four main commercial types of CSP plants: Abengoa Solar's PS10 power tower, Seville, Spain (Top left) (Abengoa Solar, a, 2010); Flagsol parabolic troughs at Kramer Junction (SEGS V), California, USA (Bottom right) (Kearney, 2007);Sandia 10-kWe dish/String system (Top right) (Kearney, 2007); Linear Fresnel Reflectors at Kimberlina Solar power station, California, USA (Bottom left) (Areva Solar, 2010).

Focus will be on parabolic trough foundations due to the technology being the most mature of the CSP technologies, with its commercial introduction in 1984 with SEGS I in California, and therefore has undergone the most commercial development in order to optimise the technology and reduce the overall cost of energy (Price et al., 2010). - Cost-Effective Parabolic Trough Foundations for Concentrated Solar Power Plants (Paper C1)

Technology	Efficiency (%)	Ground Works	50MW Plant Area (Acres)	Capital Cost
Power Tower	17	Intermediate	590	Low
Parabolic Trough	14	Extensive	240	Low
Dish / Engine	30	Minimal	500	High
Linear Fresnel Reflector	11	Extensive	120	Low

Table 1 Comparison of CSP Technologies

Parabolic trough systems consist of parallel rows of troughs that have single axis tracking of the sun. They are curved in one axis to focus the sun's light onto an absorber tube that contains heat transfer fluid. This transfers heat via an exchanger to form steam to drive a conventional steam turbine power generation system. Parabolic troughs account for the largest share of the current CSP market and are the most mature technology (Kearney, 2007). They are around 14 % efficient in terms of solar radiation to net electricity output (Richter, 2009).

Site selection and ground works play a key role in the parabolic site as the site must preferably have less than 1% gradient to minimise shadowing (Abengoa Solar, b, 2010). For 50MW of power generation a site of around 240 acres and 620 collectors is required, based on 2,000kWh/m<sup>2</sup>/yr. Foundation types commonly used include mini piles, concrete slabs and concrete caissons. Extensive ground works are normally required to ensure the 1% gradient.

Research into optimising design is underway in order to reduce the large capital costs associated with this underdeveloped technology. Currently 31-35% of capital costs of a 50MW farm are in the solar field (Vallentine et al., 2009), due to the large number of solar collector assemblies (SCAs) required. This provides a huge potential to reduce the overall cost by optimising the design of the SCAs and foundations. The challenge arises in reducing material, weight, simplifying manufacture, and assembly while maintaining structural rigidity, as efficiency of the collectors is highly dependent on the optical accuracy (Kolb and Diver, 2008). The major factor that influences the structural design of the SCA is wind and therefore a thorough understanding of its behaviour across the expanse of the solar field is important. 225

The SCAs must be able to withstand the wind loading imposed on them while still maintaining their optical accuracy, as wind thrust is predicted by the Eurotrough consortium to be the most significant optical error source.

It has been shown that there is a considerable shielding effect by the first row of SCAs, (Naeeni and Yaghoubi, 2007; Holze et al., 2010 and Hosoya et al., 2008). The foundation design will thus vary according to the field position of the SCAs with the outer foundations being significantly larger in order to accommodate the higher wind loads, which generate larger pitching moments to the foundations. Differential settlement of foundations may also have an effect on optical efficiency over the design life of the plant and so there is a need for development of suitable cost-efficient foundation designs for various soil types and load conditions.

In order for a particular foundation type to be chosen and optimised, an understanding of the complex loading conditions seen by the parabolic trough is required. It has been found that limited work has been undertaken to obtain a clear understanding of the interaction of the troughs within an array and even less work to validate Computational Fluid Dynamic (CFD) models and wind tunnel tests with full scale testing. The most extensive research to date has been carried out by the National Renewable Energy Laboratory (NREL) in the United States of America (Hosoya et al., 2008) and this will form the basis for load calculations. From this estimate of loads, a comparison can be made between locations within the solar field and foundation types typically used for typical conditions of a solar plant found in a semi-arid area. Currently foundation solutions are not optimised for their location within the solar array and therefore the force experienced during operation. Through a detailed understanding of the variance in loads experienced, depending on their location with the field, by the SCAs it can be shown that considerable savings can be made on the capital cost of foundations in comparison to foundation designs that are currently being used within industry.
The paper will therefore summarise the background to the loading conditions and foundations currently used in industry before presenting the results of a design comparison between the optimized design based on the location specific loading and designs currently used in industry designs highlighting the potential capital savings that can be made.

# 2 WIND LOADING

One of the major structural design considerations in developing solar collector fields is wind loading. Not only must the collector be able to sustain maximum loads, but it must be able to operate accurately during tracking to maintain efficiency. Therefore an understanding of the wind behaviour across the solar field is essential.

The wind loading will be affected by the following factors;

- Collector shape,
- Collector height above ground,
- Collector pitch angle,
- Number and arrangements of collectors in an array,
- Wind direction.



Figure 2. Spatial variance of horizontal and vertical force coefficients

Research into wind loading by Naeeni and Yaghoubi (2007) concluded that the pressure field is very low at 15 - 20 times the collector aperture length from the collector assembly. Therefore the wind force reduces significantly on subsequent collectors. This is confirmed by Holze et al. (2010) who using scale models in wind tunnels to calibrate their numerical simulation results, found that on rows 2 to 10, a decrease of 40% and 70% for maximum moments and loads respectively was observed. They noted that change in the overall loads occurs at the 5<sup>th</sup> row from the upwind edge of the array field. The mean loads tend to decrease continually through to the 5<sup>th</sup> row where they reach their minimum, regardless of the pitch angle, but in some instances the dynamic loads are amplified within the interior of the

field, resulting in negligible reduction in the effective peak design load. This is due to the downstream turbulence caused by the collector row in front; it was noted as being especially true when the collectors are at a pitch angle of -60 degrees. This dynamic effect was also noted by Hosoya et al. (2008) in wind tunnel findings and is particularly prominent in the horizontal force on the 2<sup>nd</sup> row of collectors which experience upwind forces greater than anywhere else on the field. This can be seen in Fig. 2 which is derived from raw data presented in Hosoya et al. (2008).

Eupfet and Greyer (2001) concluded that 95% of the solar collector assemblies are within the shadowed inner field, 2.5% are in the transient area and 2.5% on the high load outer edge and therefore three different foundation structures are necessary to ensure efficient field design.

Hosova et al. (2008) reported on their results from wind tunnel testing of parabolic solar collectors. Within this report, detailed data was included on force and moment coefficients for various collector locations within a field array for different prominent wind orientations and throughout the operational range of collector pitches. From this, a good understanding of the varying forces and moments seen across the field for various operating conditions can be developed. Analysis of this coefficient data reported shows that the peak horizontal force and moment coefficients are seen on the corner assemblies of the field, with the horizontal force component being just over six times the value of the inner field. There is a dramatic drop in the horizontal peak coefficient with the 2<sup>nd</sup> row of central columns of collectors only having 23% of the value of the 1<sup>st</sup> row. The spatial variance of peak horizontal and vertical force coefficients can be seen in Fig. 2, based on worst case wind direction (yaw of 30° to first or last row of field). The results also indicate that there is a considerable drop in force and moment coefficients from the  $1^{st}$  outer row / column to the  $4^{th}$  row / column. With the rest of the inner field experiencing very little variation from the 4<sup>th</sup> row / column inwards, but the 2<sup>nd</sup> row can experience larger negative horizontal forces. Given the detail of Hosoya et al. (2008) 229

results and the general agreement with other findings, their data has used as the input for the wind loading which has been used for the design comparison and optimisation.

# **3** FOUNDATIONS

Based on reviews of current solar farms in operation it can be seen that there are a variety of foundation types in use. There is not only variation in design concepts from plant to plant, but also within the site to cater for the varying wind loading across the site, with perimeter foundations and inner field foundations. From the research undertaken, four main commercially used foundations have been found.

### **Reinforced Slab**

This is one of the more widely used foundation types seen at various sites across the world including Solnova 1. They only require simple excavation of a cuboid pit using standard and widely available excavators. Metal jigs are then used to locate the anchoring bolts accurately to ensure correct position and angle while the concrete is poured and sets.

Investigation of safety factors on site specifications of foundation design suggests that the slab design can be optimised, with a subsequent reduction in spoil removed and concrete volume used. Due to sliding having the lowest factor of safety, optimisation is likely to be achieved by reducing the length of the slab whilst increasing the width or depth to maintain the bearing and overturning capacity while increasing the sliding capacity. Optimisation will be discussed later in the comparison Chapter.

## Mini Pile

This is one of the simplest foundation types requiring minimal ground preparation and very little to no soil removal. It has demonstrated experience in the use of underpinning of buildings and for Arizona Western College Solar Project, Metal Foundation Industries (2012).

The mini pile design consists of steel piles driven into the ground in a rectangular grouping. Anchoring points are then grouted into the centre of the piles or an integrated connection on the pile top is used to fix and align the SCA.

Calculations carried out indicate that at a typical site depth there is insufficient friction to ensure uplift does not occur under the action of the vertical wind force and pitching moment for all load cases. Uplift is therefore the most critical failure mode for mini-pile design.

Based on API conditions, the pile separation for the inner field mini pile design as used on site is within the lower boundary of influence between piles, taken as 3-5*D*, which should be avoided for friction piles. Therefore more in depth soil to pile interaction using finite element analysis is required to accurately calculate pile capacities.

### **Pair of Drilled Caissons**

These foundations consist of two reinforced concrete caissons each containing two of the four SCA locating bolts. Two boreholes are excavated via a specialist rotary boring excavator; a metal jig is used to align the anchoring points across the pair of caissons and to hold the reinforcement in place while concrete is poured. As with the pile design, varying the foundation footprint to accommodate varying loads has the advantage of fixing the price of the foundation throughout the field due to material and installation cost being independent of loading conditions associated with location within the field.

The calculations assume there is no interaction between the adjacent caissons. Bowles (1988) states it is safe to make this assumption as long as caisson separation is greater than or equal to two diameters for no interaction to be considered to occur.

### **Single Drilled Caisson**

This type of foundation concept is used greatly in the USA, for example commercial development at Nevada Solar One, USA. Increase in wind loading can be catered for by 231

increasing the size of the caisson or by using slabs on the outer edge of the fields. Installation only requires one rotary bore for each foundation, reducing excavation time per foundation as repositioning of equipment is not required, unlike with the twin caisson design. Based on the foundation calculations, the critical failure mode was overturning which was therefore the driving criterion for design optimisation.

## **Design Methodology**

### Loading

Based on the extensive force and moment coefficients of Hosoya et al., (2008) the design loads for the foundations were calculated based on the following equations (Hosoya et al., 2008)

Horizontal Force, 
$$F_x$$
  $F_x = qLWC_{fx}$  (1)

Vertical Force, 
$$F_z$$
  $F_z = qLWC_{fz}$  (2)

Pitching Moment, 
$$M_y$$
  $My = qLW^2C_{my}$  (3)

Where 
$$q = \frac{1}{2}\rho U^2$$
 (4)

q is the mean dynamic pressure measured at the collector pivot height of the solar collector, L is the length of the collector, W is the collector surface height, U is the mean wind speed at pivot height and  $\rho$  is the density of air.

The mean wind speed at pivot height needs to be obtained from the design wind speed of the solar power plant in line with design codes. Due to limited availability of wind data and associated design codes for the site region, an assumption of wind speed at collector pivot height, U of 33 m/s has been used, as this has been the value used for other collector

assemblies with similar dimensions (Kearney, 2007). The use of 33m/s is also justified by the chimney basic design wind speed at site of 28.6m/s. Calculations have also been carried out with design wind speeds of 4m/s and 14m/s. These figures represent typical average site wind speed recorded during 2010 and maximum site wind speed prior to positioning the collector in the stow position during operation of the plant. As a considerable number of commercial plants use a variant of the Eurotrough, the dimensions of these troughs (Table 2) were used to determine the foundation loads.

It is understood that the solar collectors are placed into the stow position when site wind speeds are in excess of 14m/s and so for the calculations when the design wind speeds are greater than 14m/s, a stow position of -90° has been used. It is noted that during the erection phase, the collectors cannot be placed into the stow position, which raises potential issues with collectors being exposed to significantly higher wind speeds than design conditions anticipate.

able 2 Dimensions of typical Eurorough variant (Rearney 2007).						
COLLECTOR STRUCTURE	Torque tube + stamped steel cantilever					
WIND LOAD DESIGN BASIS (m/s)	33					
<b>APERTURE WIDTH (m)</b>	5.76					
FOCAL LENGTH (m)	1.71					
LENGTH PER COLLECTOR MODULE (m)	12					
LENGTH PER SCA (m)	148.5					
		_				

Table 2 Dimensions of typical Eurotrough variant (Kearney 2007).

Based on these dimensions and the design wind speed, the maximum horizontal and vertical forces, and pitching moments were obtained from Equations 1 - 4, and then used for the live load conditions on solar collector foundations within the field. Given the experimental setup used by Hosoya et al. (2008), it has been assumed that the forces derived from their coefficients would represent the forces seen at the top of the foundation, i.e. at 0.1m above ground level based on a ground clearance typically found on sites. From the findings of the various research into wind force coefficient distributions throughout the field, four main

loading conditions were considered to be representative; outer row/column fenced and unfenced (case 1 and 2), 2<sup>nd</sup> row (case 3), inner field (case 4).

Although most solar plant sites have surrounding wind barriers consisting of heaped spoil embankments, as a way of disposing of excess spoil, they are only of the order 4 m high and around 7 m from the outer collectors. It is because of uncertainty in the effect of wind breaks that the load case without the wind barrier has been considered to represent a worst case scenario, but given the correlation of Naeeni and Yaghoubi's and the NREL's findings, the load case with the wind barrier should give a reasonable estimate. There has been no study found related to the influence of types of barrier or distance of barrier from the collector to the forces seen by the SCAs.

### **Foundation Design**

Initial foundation specifications used for the comparison of foundation concepts are based on typical dimensions found on power plant sites. Typically there are two variations of each foundation design, one for the central field and the other for intermediate collectors. It is assumed that based on a worst case, the outer foundations are the same as the intermediate ones. For each of the locations, the maximum horizontal force was selected for all the operating pitches and wind directions, the corresponding vertical force and moment were then used.

Site specific soil characteristics were unavailable and so reference characteristics based on a compacted dry sandy soil similar to ground conditions at a typical semi-arid site were used in order to enable a comparison between the designs. From this information, basic bearing, uplift and overturning calculations were carried out for ultimate limit state for the four load cases on each of the foundation types according to the British Standards.

The calculations do not factor the loads or capacities in order for an overall factor of safety to be determined and then used for comparison of suitability of the foundation to four loading

conditions. A factor of safety of 2 has been chosen as a minimum acceptable design criterion. Self-weight of the collectors has not been incorporated in the design calculations due to the unavailability of SCA weight data. The introduction of self-weight into the calculations would be beneficial and result in a reduction of uplift and overturning moments for the foundations.

## Optimisation

A comparison of the four different types of foundations based on the assumption of 33 m/s design wind speed, the stow position of -90° and the soil characteristics mentioned previously can be seen in Table 3. Initial sizing is based on typical site design specifications, the  $2^{nd}$ sizing represents the required foundation to meet a safety factor of 2 based on increasing depths if an insufficient safety factor was achieved based on the site specifications. The 3<sup>rd</sup> is the fully optimised foundation specifications that produce the minimum volume of materials required and spoil produced while still achieving a sufficient factor of safety for the failure modes. Based on these quantities costs has been produced that take account of materials, The costs do not include associated costs such as labour and equipment. mobilisation/demobilisation, welfare, access routes, drainage costs etc. These costs have been derived from rates published in SPON's Civil Engineering and Highway Works Price Book by Langdon (2010), so it should be noted that although this allows for a good financial estimate, labour rates for the majority of countries where large solar projects are being installed tend to be cheaper and so more labour intensive foundations could perform better in the comparison. The costing also does not take account for the variation in SCA support structure that maybe required for the different foundation types.

In terms of optimisation of the existing foundations consideration of structural integrity becomes important, for example the caisson design, optimisation suggests that the diameter 235

could be reduced. However, reducing the diameter of the caisson reduces the self-weight so the factor of safety for uplift becomes important. As the slenderness increases, the structural integrity of the caisson will become important and will need verifying due to limit state design parameters.

	-	Mi	ni Pile		Slab		Single Caisson		2 x Caissons			
		Site Design	Site based with F.S ~2	Site Design	Site based with F.S ~2	Optimised Design	Site Design	Site diameter	Optimised Design	Site Design	Site based with F.S ~2	Optimised Design
	Footing Depth (m)		6.30	1.00	1.00	1.50		5.45	7.20	1.45	3.75	5.85
	Length , diameter (m)		0.15	4.00	4.60	3.30		0.85	0.50	0.86	0.86	0.50
	Width (m)			2.30	4.00	2.40						
	Volume of concrete (m <sup>3</sup> )		0.01	10.12	20.24	12.67		3.15	1.43	1.80	4.47	2.34
	Volume of spoil (m <sup>3</sup> )		0.00	9.20	18.40	11.88		3.04	1.39	1.68	4.36	2.30
Load	% Spon saving over Site				-100.00	-25.22					-148.39	-29.76
Case 1	% Spoil Saving over Site				100.00			<0.00	05.04			
Corner	Slab	N/A			-100.00	-25.22	N/A	68.88	85.84	82.21	55.80	76.91
outer	Volume of steel (m <sup>3</sup> )		0.01	0.30	0.61	0.38		0.06	0.03	0.04	0.09	0.05
neiu	Cost of Excavation/		75.29	87.95	175.90	113.57		728.11	950.74	218.63	565.43	862.86
	Cost of											
	concrete/placement		1.17	1244.86	2489.72	1558.78		327.82	433.08	67.66	174.98	272.96
	Cost of steel		375.00	3811.00	7622.00	4772.03		1185.98	539.77	678.12	1684.36	879.90
	Estimated cost /		1805.84	5143.81	10287.63	6444.39		2241.91	1923.59	964.41	2424.77	2015.72
	Denth (m)		4 10	1.00	1.00	0.90		3 30	4 70	1.45	2.25	5.15
	Length , diameter (m)		0.15	4.00	2.30	2.30		0.85	0.50	0.86	0.86	0.40
	Width (m)			2.30	1.80	2.00						
	Volume of concrete (m <sup>3</sup> )		0.01	10.12	4.55	4.60		1.93	0.94	1.80	2.73	1.32
	Volume of spoil (m <sup>3</sup> )		0.00	9.20	4.14	4.14		1.87	0.92	1.68	2.61	1.29
Load	% saving over Site				55.00	54.55					-51.61	26.73
Corner	% Saving over Site Slab	NT/ A			55.00	54.55	NT/A	80.94	90.69	82.21	73.02	86.96
outer	Volume of steel (m <sup>3</sup> )	N/A	0.01	0.30	0.14	0.14	N/A	0.04	0.02	0.04	0.05	0.03
field	Cost of Excavation/		49.00	87.05	30.58	30.58		441.02	346 62	387 73	601.65	606 33
fenced	Piling		49.00	87.95	59.56	59.50		441.02	540.02	367.75	001.05	000.55
	Cost of		1.17	1244.86	509.26	509.26		198.50	109.65	174.44	270.68	120.10
	Cost of steel		375.00	3811.00	1714.95	1732.27		705.18	347.53	634.37	984.37	487.42
	Estimated cost /		1700.68	5143.81	2263.79	2281.11		1344.70	803.79	1196.54	1856.70	1213.85
	foundation (£)	2.45	2.00	1.00	1.00	1 10		0.00	1.00	1.45	1.40	2.50
	Deptn (m) Length diameter (m)	2.45	2.90	1.00	3.30	1.10		0.90	0.25	1.45	1.40	2.50
	Width (m)	0.15	0.15	2.30	1.35	1.00		0.05	0.25	0.00	0.00	0.25
	Volume of concrete (m <sup>3</sup> )	0.01	0.01	10.12	4.90	1.20		0.57	0.09	1.80	1.74	0.26
	Volume of spoil (m <sup>3</sup> )	0.00	0.00	9.20	4.46	1.10		0.51	0.09	1.68	1.63	0.25
	% saving over Site				51.58	88.14					3.23	85.82
Load Case 3	design % Soving over Site Slob				51 59	99 14		04 20	00.08	82 25	82 78	07.49
2nd	Volume of steel (m <sup>3</sup> )	0.00	0.00	0.30	0.15	0.04	N/A	0.01	0.00	0.04	0.03	0.01
row	Cost of Excavation/	0.00	0.00		0.15				0.00			
	Piling	29.28	34.66	87.95	42.59	10.52		120.28	131.62	387.73	374.36	292.70
	Cost of	1.17	1.17	1244.86	548.01	135.31		54.14	41.99	174.44	168.42	58.30
	concrete/placement Cost of steel	375.00	375.00	3811.00	1845 44	451.90		192 32	33 27	634 37	612 50	92.43
	Estimated cost /	1621.01	1642.22	5142.91	2426.02			266.74	207.00	1106 54	1155 29	142.42
	foundation (£)	1021.81	1043.32	5145.81	2430.03	391.12		300.74	200.88	1190.34	1155.28	443.42
	Depth (m)	2.45	2.70	1.00	1.00	0.90		2.25	3.90	1.45	1.50	3.65
	Lengtn, diameter (m) Width (m)	0.15	0.13	3.50 2.30	2.00	2.00		0.85	0.30	0.86	0.86	0.25
	Volume of concrete $(m^3)$	0.01	0.01	8.86	2.42	2.40		1.33	0.28	1.80	1.86	0.35
	Volume of spoil (m <sup>3</sup> )	0.00	0.00	8.05	2.20	2.16		1.28	0.28	1.68	1.74	0.34
_	% saving over Site				72.67	72.90			78 80		.3.23	80 36
Load	design				. 2.07	72.90		04.04	06.00	00.00	-5.25	06.01
Lase 4	Volume of starl (m <sup>3</sup> )	0.00	0.00	0.27	0.07	0.07	N/A	<b>84.94</b>	90.81	<b>80.98</b>	79.01 0.04	90.01
fie ld	Cost of Excavation/	0.00	0.00	0.27	0.07	0.07		0.05	0.01	0.04	0.04	0.01
	Piling	29.28	32.27	76.96	21.03	20.65		300.70	285.53	387.73	401.10	427.28
	Cost of	1.17	0.88	1089.25	270.62	265.70		135.34	90.99	174.44	180.45	85.12
	concrete/placement Cost of steel	375.00	375.00	3334 62	911 33	903 79		480.80	103.81	634 37	656 25	129.60
	Estimated cost /	1621.01	1/20 =0	4500.04	1202.00	1100 17		016.00	100.01	1106.54	1007.00	(40.00
	foundation (£)	1621.81	1632.59	4500.84	1202.98	1190.15		916.84	480.33	1196.54	1237.80	642.00

Table 3 (	Intimisation	of Typical	CSA Fo	undations
I able 5 V	Jpunnsauon	UI I ypical	COATU	unuations

For the purposes of this design, basic limit state design principles have been adhered to and the concrete member length to lateral diameter has been limited to less than 15 to avoid buckling. If the foundation diameter < 0.75m and <  $\frac{1}{4}$  of the depth of the structure, it has been modelled as a pile. Classification of caissons and slabs are follows; a caisson is defined by a ratio of 1 < d:B < 4 and a pad / slab foundation defined by d:B < 1, where *d* is the depth and *B* is the breadth of the foundation, Institute of Structural Engineers (2002).

By moving to a pile design it is possible that borehole excavations may require support through the introduction of a casing or slurry e.g. bentonite. If the use of slurry is required to support the excavation in addition to initial spoil there will also be slurry waste to dispose of. The majority of materials currently used as slurry must be disposed of at suitable, licensed waste facilities and cannot be re-used on site, this adds additional cost, especially given the remoteness of typical sites. Initial design considerations indicate that potential additional costs and time that may be incurred, due to depth of excavation and the requirement to use stabilising slurry for excavations, may invalidate any savings achieved through design optimisation. The use of driven piles may prove more cost-effective than bored piles and should be given further consideration at concept design stage.

In terms of material volume, it can be seen from table 3 that the caisson design represents the optimum design in terms of minimisation of concrete and spoil. In terms of an overall cost estimate per foundation the caisson design is also the cheapest of the four options for all cases apart from the unfenced outer location. For this location, the two caisson designs offer the most cost-effective solution due to the horizontal loads being too high for a single caisson to withstand structurally without becoming very large in diameter. It should be noted that the stability of the bore hole was not considered at this stage due to the uncertainty of soil conditions.

A comparison between the designs found on the case study site, with dimensions adequate enough to meet a minimum factor of safety of 2, indicates that the use of mini piles is the most cost-effective solution in most of the load conditions considered, apart from the inner field where a small slab foundation could offer a 7.5% saving over the mini piles.

The basic optimisation that has been undertaken shows that significant cost savings could be made for all locations throughout the field and in some locations such as the  $2^{nd}$  row optimisation could lead to savings of up to 60% over the designs currently being used on typical sites. For a typical 50 MW site this potential saving for the three different foundation locations could represent a total saving of around £1.8m for the foundation works over the typical site foundation design currently used based on providing the same factor of safety, the assumptions stated earlier and 2 foundations per collector. Given that solar projects are only getting larger in scale, the potential for financial savings in the solar field would only increase further in the future.

A factor that has not been considered in great detail is the effect of foundation settlement which will affect collector efficiency, as this is highly dependent on line and level of the collector assembly. Misalignments of the collectors can significantly reduce output efficiency with minor adjustment to correct this adding to maintenance costs. Settlement of foundations and differential settlements between foundation groups could therefore be critical in achieving overall plant performance. As the majority of solar projects are sited on granular soils there is unlikely to be foundation settlement issues (Bowles, 1988) with the majority of ground settlement occurring within 7 days of the collector being placed onto the structural support frame and foundation. Final line and level checks of the collectors are often not carried out immediately and as such by the time final checks are carried out all settlement will have taken place. Developments on cohesive fine grained soils are at increased risk of suffering from settlement issues due to the majority of settlement occurring over an extended period of time.

In the case of solar collectors, this translates to a requirement to check line and level of the collectors at regular intervals to correct any misalignments and maintain efficiency if differential settlement occurs. In the case of slab foundations, it is also necessary to check for differential settlements resulting from foundation rotation at the edge of the foundations due to high horizontal wind load. Initial calculations indicate that differential settlement of the outer perimeter slab foundations is not of concern, but if a lifetime cost analysis is to be performed, more detailed investigation will be required to determine the frequency of alignment checks and adjustment.

Another potential for improvement is the accuracy of the design wind loads assumed for calculation purposes as this was based on limited design data supplied by site operators and limits used throughout the industry. In order to gain a more accurate assessment of the likely risk of extreme gusts of different magnitudes at site, it is recommended that local wind speed measurement data is collected from site for at least one complete year, and at a representative location and height above ground level. This data could then be analysed and correlated against a suitable local meteorological station for which long-term (up to 20-year) data is available, in order to develop a long-term wind resource distribution at the site to provide a more accurate estimate of the probability of critical wind speeds being exceeded at the site. Significant improvements could also be made to the derived loads through the production of a CFD model calibrated to full scale site experimental results as there is no evidence of full scale testing to date.

Investigation of the suitability of a stow angle of  $-90^{\circ}$  indicates that  $-100^{\circ}$  produces minimum forces, with the magnitude difference between  $-90^{\circ}$  and  $-100^{\circ}$  for the mini pile design indicating that wind speeds of up to 26.5 m/s can be accommodated at  $-100^{\circ}$  compared to 23 m/s at the  $-90^{\circ}$  stow position.

# 4 CONCLUSION

This paper has reviewed currently available data on solar collector foundations and has presented an initial assessment of the foundation designs at a typical example site. There is relatively limited information available on foundation design for solar collectors as CSP technology remains in the early stages of development. The literature review has highlighted that little research has been carried out to assess suitable and cost-effective foundation types such as the four main types of foundations currently in use, i.e. slab base, pair of drilled caissons, single caisson and mini pile/pile. It suggests that the types of foundations used as examples of site foundations are typical of those adopted for CSP plants currently being developed. There is limited practical experience on the performance of foundations in the field, although there are no reports of failures or extensive remedial works required.

Wind loading is the primary design load for consideration, particularly for seismic-inactive areas, but there is limited understanding of the variance of wind load across the collector array, given the complex shape and changing position of the collectors throughout the day. As such wind loading has been calculated in this paper from available laboratory data and applied to a typical trough to carry out the foundation design. The assumed design value of the wind speed is based on design parameters adopted in similar sites, and is consistent with the basic wind speed adopted for chimney design at site. The maximum operating wind speed before the stow position is adopted is 14m/s and less than half the design speed.

The results of the analysis of initial typical site designs indicate that the mini-pile design could be susceptible to uplift when site wind speeds are greater than the maximum operating wind speed of 14m/s and the collector is in the stow position, but stow position optimisation could improve the situation.

The caisson design achieves minimum desirable factors of safety for all failure modes at operational wind speeds and therefore can be optimised.

The slab foundation has considerable factors of safety and so considerable optimisation can be achieved on this foundation type.

A financial comparison indicates that a single caisson design could represent the most costeffective solution for CSP foundations, based on the assumptions made.

There is limited confidence in using traditional foundation modelling for the uniqueness of parabolic trough collector foundations and there would be considerable benefit to carrying out further, more in depth research into the behaviour of the collectors within the field. It is recommended that any further modelling work is validated through site data collection.

Settlement of foundations is not considered to be an issue, as any potential settlement effects could be addressed at the detailed design stage.

Overall the calculations suggest that foundation design optimisation is possible and significant financial savings can be made in the construction of foundations in the solar field of parabolic trough concentrated solar power projects, with this work indicating a possible saving of around £1.8m on a small 50MW example case study. This saving could be delivered through further modelling work to achieve greater confidence in results and a reduction of allowable factors of safety to achieve even greater cost savings.

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# APPENDIX G - MARCON WIND POWER OFFSHORE WIND FOUNDATION REVIEW (PAPER 5)

### **Full Reference**

Dallyn, P., 2014. Marcon Wind Power Offshore Wind Foundation Review. E.ON Technologies Limited, ETG/14/TCG/PB/706/R

### Abstract

E.ON Climate & Renewables (EC&R) Nordic had previously engaged MarCon Wind Power (MWP) to design and deploy an 80m high meteorological mast at their Södra Midsjöbanken site in a water depth of approximately 15m. The meteorological mast foundation that has been deployed, MWP Mark 2, represents a novel concept that can be floated out and selfinstalled through the use of a jack-up mechanism, it therefore has the potential to be redeployed at a later date and if the foundation concept proves to be successful as a meteorological mast could also be scaled to accommodate a wind turbine generator. This foundation concept could offer considerable potential benefits at future E.ON sites e.g. cost savings, installation optimisation. EC&R have engaged E.ON New Build and Technology (ENT) to facilitate knowledge transfer and ensure optimum data capture of this demonstration project occurs and investigate it's potential as a full scale offshore wind turbine foundation.

The basis of the concept is a three-legged jack-up foundation that floats to site with the turbine or meteorological mast pre-installed at the harbour. The Y-shaped hull is designed to provide sufficient righting arm for stability, allowing it to be transported in reasonable sea states. After transporting the platform to the desired location, the legs are lowered to the sea floor, the hull jacked up out of the water, the water tank compartments ballasted to provide sufficient penetration of the legs into the sea bed and dead weight and then final jack up to achieve the desired air gap.

Various studies undertaken both internally and externally have been reviewed to collate all information and allow an assessment of the maturity of the concept as both a meteorological mast and offshore wind turbine generator (WTG) foundation. The review has identified that technically the foundation concept is reasonably mature and considered technically competitive against other novel foundations, although fatigue concerns have been raised in regards to both the WTG and meteorological mast foundation types. Commercially under the majority of conditions investigated the concept is one of the more expensive. It appears to be only far from port sites that the MWP concept may be a commercially favourable foundation option for both meteorological mast and WTG foundations compared to both traditional and current novel foundations. Ongoing developments in the jack up system have potential to reduce the fabrication costs considerably, which may help to improve the commercial competitiveness of the MWP concept.

The deployment of the Södra Midsjöbanken meteorological mast has enabled de-risking of aspects of the fabrication, transportation and installation processes and also allowed optimised design and cost reductions to be identified and developed. The met. mast has also demonstrated that use of a mobile meteorological mast or offshore wind turbine is feasible. The deployment of the met mast has provided confidence in the design approach of MWP along with confidence with material take offs and cost estimates presented in documentation.

During 2013 ENT considered installation of condition monitoring on the met mast to enable validation of design models and provide further scope for de-risking of the concept. The project was not completed due to costs of measurement device installations at sea outweighing the value to the project. However, should the opportunity arise to complete this work it is recommended that installation of condition monitoring data would be extremely beneficial.

Given the potentially favourable CapEx possibility of the MWP Mk2 concept for far shore sites it is recommended that

- MWP be included in future foundation screening for both meteorological mast and WTG foundations for far shore sites
- Continued technology tracking undertaken through supplier engagement to ensure the latest developments are available for foundation option decisions, improve certainty in cost estimates and ensure any opportunities to realise potential cost savings are utilised
- Consider the feasibility of structural condition monitoring of the MWP Mk. 2 Meteorological mast at Södra Midsjöbanken if circumstances change, MWP demonstrate fatigue requirements can be met and cost benefits of the revised jacking system. As this will aid in de-risking of the structure and improve knowledge on structural performance and certainty in environmental limits of the foundation.

## Keywords

Self-installing, Offshore Wind Foundation, Review

Paper type – Technical Report, Review

# **1 INTRODUCTION**

E.ON Climate & Renewables (EC&R) Nordic had previously engaged MarCon Wind Power (MWP) to design and deploy an 80m high meteorological mast at their Södra Midsjöbanken site in a water depth of approximately 15m.

The meteorological mast foundation that has been deployed, MWP Mark 2, represents a novel concept that can be floated out and self-installed through the use of a jack-up mechanism, it therefore has the potential to be redeployed at a later date and if the foundation concept proves to be successful as a meteorological mast could also be scaled to accommodate a wind turbine generator. This foundation concept could offer considerable potential benefits at future E.ON sites e.g. cost savings, installation optimisation. EC&R have engaged E.ON New Build and Technology (ENT) to facilitate knowledge transfer and ensure optimum data capture of this demonstration project occurs and investigate it's potential as a full scale offshore wind turbine foundation.

This report will summarise the knowledge gained over 2012-2013 and detail recommendations for the future activities.



Figure 1 Installed Södra Midsjöbanken meteorological mast (EMMA) and artists impression of MWP Mk 2 offshore wind turbine foundation

The basis of the concept is a three-legged jack-up foundation that floats to site with the turbine or meteorological mast pre-installed at the harbour. The Y-shaped hull is designed to provide sufficient righting arm for stability, allowing it to be transported in reasonable sea states. The hull consists of three pontoons which have compartments that provide the buoyancy for self-float out transportation and ballasting tanks for water during installation to achieve sufficient leg penetration into the sea bed and operational dead load. The pontoons also house the jack up legs at one end and converge on a central hull section to transfer the loads from the turbine or meteorological mast to the legs. The pontoons also provide sufficient deck area for housing the jacking control systems and power generation equipment in the meteorological mast case. The hull layout is shown in Figure 2. After transporting the platform to the desired location with a minimal vessel requirements of two tugs (one for tow out and one for positioning), the legs are lowered to the sea floor the hull jacked up out of the water, the water tank compartments ballasted to provide sufficient penetration and dead weight and then final jack up to achieve the desired air gap.

The rights to the concept are owned by MarCon Wind Power AB, a subsidiary of MarCon Gruppen AB, who's operations branch Svensk Sjöentreprenad (SS) undertake the operations and maintenance of the structure. MWP typically seek to offer a complete design, build, installation and operations package for the MWP Mk. 2 concept. This approach has been adopted for the meteorological mast at Södra Midsjöbanken. The design work is contracted out to Bassoe Technology who have a strong background in offshore oil and gas vessel and structure design. The meteorological mast was fabricated at Oresund Steel Construction yard in Landskrona, Sweden. The design and fabrication of the foundation utilises the American Bureau of Shipping (ABS) Rules for Building and Classing Mobile Offshore Units (MOU)

and achieved full class certification in 2012 as an A1 self-elevating unit. The concept has also been approved in principle as a WTG foundation by ABS.



**Figure 2 Södra Midsjöbanken meteorological mast hull layout** Key advantages of the design are

- Three-legged jack-up structure results in minimal environmental impact with no piling or drilling noise and complete removal at decommissioning
- Potential for integrated installation with complete construction onshore eliminating the need for expensive heavy lift vessels, resulting in minimal installation vessel spread requirements
- Potential for serial fabrication benefits through modular design
- Relocation possible after two year meteorological monitoring campaign for the meteorological mast case.

# 2 SUMMARY OF DEVELOPMENT

The concept first appeared in the early stages of development as part of the initial Carbon Trust Offshore Wind Accelerator Programme (OWA) foundations competition in late 2009, where it made it to the final seven concepts, but was dropped due to higher capital expenditure (CapEx) than any other design at £0.95million/MW, including both fabrication and installation costs of the foundation. MWP have challenged the OWA estimates on cost, stating that the lack of relevant jack up construction vessel experience had resulted in unreasonably high fabrication costs. MWP therefore believe the true costs should be less than the figure quoted.

Considerable development of this concept has been undertaken since the initial OWA competition to enable deployment of a meteorological mast foundation at Södra Midsjöbanken. This was successfully installed in the end of March 2012 with the mast pre-assembled to the foundation onshore and then towed and erected without reported incident. The performance of the MWP met mast during tow-out and installation is considered impressive compared to other novel foundation concept meteorological mast installations e.g. Universal Foundation installation at Dogger Bank.

Based on the successful deployment of the meteorological mast, Norwegian offshore wind developer Statkraft took interest in the concept and asked MWP to undertake a design basis for a 5MW WTG for Round 3 development site Dogger Bank conditions in mid-2012. Statkraft then undertook an independent review on the initial design for fatigue limit state, ultimate limit state and frequency requirements, the results of which are discussed in detail later, but highlighted potential insufficient fatigue capacity in the structure.

In early 2013 Renewable Energy Systems Holdings Limited (RES), a global renewable energy developer who's core activity is development, design, construction, financing and

operation of wind farm projects, undertook a feasibility study of the MWP concept for a joint venture development site, St Brieuc, France. Although detailed information on the design was not released, cost estimates benchmarked against a jacket structure were released in May to the OWA. These indicated significant savings of the MWP structure over a jacket structure (10%) for the site specific conditions. RES have also signed a distribution deal with MWP to act as a UK agent for the MWP Mk. 2 for both meteorological mast and WTG structures.

At a similar time ENT undertook a detailed design review of provided design documentation for the MWP Mk. 2 meteorological mast located at Södra Midsjöbanken, Appendix A-1 [removed]. This provided useful clarification on design and installation procedures and highlighted a need to understand the structures operational behaviour and identify any design 'conservatism'. The review identified high fatigue and ultimate utilisations in some areas, helping to identify the areas crucial to structural performance. This detailed design information was also used as the basis for the specification of a structural condition monitoring system, Appendix A-2 [removed], to better understand these utilisations, performance of the meteorological mast structure and accuracy of design models. The structural condition monitoring of the structure was to be undertaken as a collaborative research project between MWP, Statkraft and E.ON, with installation of the system planned for autumn 2013. However, the cost of undertaking works offshore coupled with the announcement of the divestment of the Södra Midsjöbanken development site introduced significant uncertainty with regards the future of the foundation which has delayed the project in the short term.

The Carbon Trust OWA foundation technical working group included the MWP Mk. 2 structure as part of steel structure benchmarking study [1] undertaken by Grontmij in mid-2013, where a cost comparison was produced in July against other novel and traditional

foundation concepts for typical UK Round 3 conditions. This indicated that the MWP structure was one of the least cost competitive solutions for these conditions.

ENT invited MWP to undertake a design basis for the deeper waters of the E.ON Round 3 development site, Rampion, Appendix B-1[Removed]. As part of development of the Rampion site a number of novel concepts have been previously assessed for design suitability and cost; it was envisaged that the data supplied by MWP would enable comparison of the MWP concept against these other novel concepts. The level of detail provided in the MWP submission was only sufficient to prove the feasibility of the concept for Rampion site conditions of 30m water depth, detailed cost information was not included. MWP advised that further information could be supplied at a cost; ECR have not pursued this any further to date.

In late 2013 EC&R undertook a request for information from various foundation designers for Arkona and Kiegers Flak site conditions with water depths of 25 and 20m respectively. This included the MWP concept along with other foundation types, both traditional and novel. Initial returns have been received and evaluated. The MWP submission, Appendix C-1 [removed], included cost estimate information based on the RES study completed in early 2013 rather than the for the site specific details of Arkona and Kriegers Flak. This lack of detailed, site specific information, limited the effectiveness of any comparisons drawn between the concepts. The review concluded that despite the lack of detailed information provided, the MWP concept was unlikely to be cost competitive.

Since 2012 developments of the structure have mainly focused on reducing the CapEx of the fabrication of the foundation. The main development has been around the jack up system used, as the jacking units represent around 20% of the total cost of the meteorological mast contract.

The proposed new system has not only changed in design, from the pin and hole to clamping system, but in installation ethos, with the complete hydraulic system taken from foundation to foundation after each installation, rather than being a permanent part of each foundation.

# **3 EVALUATION**

In order to form a basis for the assessment of the MWP, as both a meteorological and WTG foundation, various published information through both OWA involvement and MWP engagement have been reviewed with the key points raised, summarised below.

## 3.1 RES St Brieuc Design Basis

An independent cost comparison study performed by RES on the MWP concept verses a traditional jacket based on St Brieuc site conditions has been undertaken. This is based on site conditions of 35m water depth, 280km from port with 100 foundations suitable for 5MW WTG [2]. The reference jacket used in the study was not identified. Table 1 shows the indicated cost of the reference Jacket and MWP foundation.

5 MW	DEPTH (M)	35
	COST	M£/MW
MWP	Fabrication	1.00
	Installation	1.00
	Total	1.00
JACKET	Fabrication	0.70
	Installation	10.75
	Total	1.10

 Table 1 Normalised MWP and jacket costs

From Table 1 it can be seen that the MWP concept is slightly more cost competitive than the reference jacket for these design conditions. In comparison to the other studies undertaken this site is almost twice as far from the construction port and as the water depth and turbine size are similar to the other studies this potentially shows the benefits of the MWP at far shore sites. However the jacket fabrication costs are around 30% higher than any jacket costs estimated by the OWA foundation benchmarking study discussed later. Although the MWP

fabrication cost is 15% higher than previous studies have indicated, it could still suggest that the jacket used by RES to complete this study is not an optimised design that is competitive with jacket designs assessed by the OWA. Conversely, it could also be assumed that the MWP design is more suited to onerous metrological conditions given the apparently lower sensitivity of fabrication cost to the change in site conditions.

If installation and fabrication cost estimates for distance to shore from this study are included with the costs estimates from the OWA benchmarking study, it can be seen that for the 35m water depth and 5MW WTG case, the distance from shore at which the MWP foundation becomes cost effective against a jacket structure is around 190 to 260km, shown by the intersections in Figure 3, depending on site specific conditions. This serves to highlight the importance of a case by case evaluation of concepts. Given the integrated installation philosophy of the MWP concept requiring only inexpensive tugs, compared to the separate heavy lift vessel (HLV) requirements of traditional jacket and WTG installation, a lower increase in cost with distance offshore would be expected. If compared to the distance to shore sensitivity analysis undertaken by GL for OWA as part of the installation cost study [3] the rate of increase of the jacket structure costs seem quite high, with an increase of £0.15million/MW shown below compared to the GL report of around £0.04million/MW, although the GL report did not consider the WTG installation. Note: the OWEC jacket (referenced in the second graph) has been used as the reference case for the OWA data, as it is deemed to be the most competitive of the traditional jacket structures assessed in the benchmarking study.



Figure 3 Cost comparison of MWP with traditional jacket structures against distance to shore

# **3.2 Rampion Design Basis**

As part of engagement with MWP for the design review of the Södra Midsjöbanken meteorological mast and to assess the maturity of the concept, MWP were asked to produce a design for Rampion site conditions in order to benchmark the concept against other foundations which had submitted designs for the Rampion foundation benchmarking study [4].

The design conditions used by MWP for this study were;

- 100 foundations
- 5MW WTG
- 33.5m water depth

Analysis for these conditions was undertaken for the ultimate limit stated (ULS) and indicated a maximum utilisation of 60% in both sand or clay soil conditions for the leg section just below the hull during operation and 80% for the hull during preload operations during installation. This results in a hull and total leg weight of x and x tonnes respectively. Frequency analysis showed that for both soil conditions investigated the 1st Eigen Frequency is in the allowable spectra between the 1P and 3P WTG requirements.

Although no details on the cost of the foundation were provided, based on the above weights, estimations for the unit costs of fabrication provided in the RES study [2] and the OWA installation unit costs [1] the total cost of the concept was estimated. A comparison of this cost to the other foundation costs benchmarked in the Rampion foundation benchmarking study [4] is shown in Table 2. Unfortunately the WTG size used by MWP (5MW) was not the same as the other submissions, which were based on a 6MW WTG, for the benchmarking study so the comparison is not completely accurate and is intended to highlight that MWP costs, as shown in previous studies, are at the higher end of costs. A £50million installation of the MWP. This value is based on MWP estimates [2].

FOUNDATION CONCEPT	Түре	DEPTH (M)	35
		COST	M£/MW
MWP	Jack up	Fabrication	Х
		Installation	Х
		Total	X
KEYSTONE	Jacket	Total	X
BAM	Gravity	Total	X
GRAVITAS	Gravity	Total	X
SKANSKA	Gravity	Total	X

Table 2 Cost comparison for Rampion design requirements

Table 2 highlights that the MWP concept is reasonably competitive compared to gravity foundations, but costs are still 13% higher than for the Keystone (twisted jacket) foundation.

It should be noted that the MWP sizing estimates (and costs) are based on a 5MW WTG; the cost per megawatt is likely to be lower for a foundation designed for a larger WTG. The sensitivity of the MWP foundation to WTG size has not been assessed. A review of the OWA large rotor study [5] enables an approximation of sensitivity to be made, as cost estimates are available for some of the steel foundations used in the OWA foundation benchmarking report [1] with an 8MW WTG. If the values for the various foundations are averaged and assumed to scale linearly, the interpolated results suggest an average reduction in cost per MW of £0.05million going from foundation designed for 5MW to 6MW WTG, see Figure 4. This would bring the cost of the MWP foundation down to within 6% of the Keystone foundation costs, which is more than likely within the error of the approximations and variation of site condition requirements the concepts have been designed to.

It is worth noting that the design submitted for the MWP Rampion study was only screened against ULS and frequency requirements. Therefore as fatigue limit states can often be the design driver for parts of offshore structures, if the fatigue life of key structural areas is insufficient this may result in increased steel weight and therefore cost to ensure a 25 year design life. The nature of the MWP design with its relatively unsupported subsea structure means it is likely to scale less favourably to larger turbines than the likes of the Keystone and other jacket structures.



Figure 4 Sensitivity of OWA screened foundation options to turbine size

3.3 Arkona/Kriegers Flak Design Basis

As part of EC&R's Offshore Pre-Construction (OPC) and in preparation for potential Baltic Sea development sites, the foundations work stream has undertaken a request for information (RFI) with various foundation designers for Arkona and Kriegers Flak site requirements. This has resulted in four design case requirements, Table 3, used to benchmark the foundation concepts that responded to the request and were deemed feasible, which included

- Grbv XL monopile
- Ramboll jacket
- Keystone
- Boskalis/OWEC Tower
- Hexabase
- Marcon Wind Power
- Aarsleff gravity

Unfortunately due to the short timescale at which MWP were invited to submit information, one week before the submission deadline, a design could not be performed and so indicative costs were provided based on the RES study conditions. Therefore as the costs are based on a smaller turbine, in deeper waters, further from shore there is little merit in undertaking a cost comparison with the other foundations that submitted cost estimates. It was suggested to MWP that if the concept is to be correctly considered in the evaluation, costs and designs specific to the study should be submitted, but at the time of writing of this report no additional information has been received from MWP.

Table 3 Design requirements for OPC foundations work stream

	6MW	8MW
<b>DESIGN SOIL PROFILE 1</b>	Scenario 1	Scenario 2
20m water depth		
<b>DESIGN SOIL PROFILE 2</b>	Scenario 3	Scenario 4
25m water depth		

From a technical perspective an evaluation matrix has been produced as part of the OPC foundation review process based on the opinion of the various participants of the foundation work stream and the information submitted as part of the RFI. The technical scoring is based on

- Concept maturity
- Design variance
- Fabrication complexity
- Degree of standardisation
- Flexibility on fabricators
- Flexibility on installation vessel
- Extent of pre installation activities
- Offshore installation time
- Extent of proven concepts

• Environmental constraints

Table 4 summarises the overall technical scores of the concepts that were deemed to be feasible for the site conditions and responded to the RFI. Scores are based from one to five, with five being the most favourable and no weighting factors included.

### Table 4 OPC foundations work stream technical evaluation scores

### [REMOVED]

From this table it can be seen that the foundations work stream evaluated the MWP concept as being the least technically favourable of the concepts, which is mainly down to its unproven nature and fabrication complexity.

By the subjective nature of this evaluation process there will always be differences in opinion and as part of the ongoing clarification process, MWP raised concerns over the relative scoring of fabrication flexibility, complexity, degree of standardisation and concept maturity.

As the evaluation process was undertaken before ENTs involvement, the matrix has also been scored as part of ENT's assessment on the suitability of the MWP Mk. 2 concept, Table 5, for use as a WTG foundation based on ENT's knowledge of the concepts through OWA and previous engagements with concept designers. Details on the derivation of the scores can be found in Appendix C-2 [removed].

Concept	DESCRIPTION	DESIGN	FABRICATION	LOGISTICS & INSTALLATION	TOTAL
AARSLEFF	Lifted gravity	3.3	3.3	2.4	9
BOSKALIS/OWEC	Four leg jacket	3.3	3.0	3.0	9.3
HEXABASE	Six leg jacket	2.8	3.0	2.7	8.5
KEYSTONE	Three leg twisted jacket	3.3	3.2	2.7	9.2
MWP	Jack up	3.0	3.0	3.3	9.3
XL MONOPILE	Monopile	3.5	3.3	2.7	9.5
UNIVERSAL FOUNDATION	Single suction caisson	3.3	2.8	3.3	9.5

 Table 5 ENT technical evaluation scores

Based on Table 5 it can be seen that from a technical perspective the lower fabrication and design maturity is outweighed by the installation and logistic benefits, resulting in one of the more technically favourable of the concepts reviewed as part of the Arkona/Kriegers Flak design requirements, but with very little to distinguish between concepts.

## 3.4 Carbon Trust Steel Foundation Benchmarking Study

The review undertaken by Grontmij was carried out for what are deemed to be the most promising steel structure concepts. Emphasis was given to the technical content of the concepts, as well as to the fabrication, marine operations and technical capabilities of the developers behind the concepts. This ensured that the most favourable concepts were selected for the initial shortlist before a more detailed evaluation was undertaken. The reviewed concepts can be divided into the following categories;

- Monopiles
- Jacket structures
- Mudline stiffened monopiles
- Tripods
- Suction buckets
- Structures supported by moorings
- Articulated towers
- Special steel structures

Of the 20 shortlisted concepts the installation and fabrication costs of nine concepts are listed in Table 6 for typical Round 3 conditions [6], with a 5MW WTG for 25 to 55 m water depths base on 100 units. These concepts represent reference cases and the most promising of traditional jacket designs and novel steel foundation designs.

#### Table 6 Foundation benchmarking study cost comparison

### [REMOVED]

### [REMOVED]

#### Figure 5 Cost comparison of OWA screened concepts at various water depths at 150km from port

From Table 6 and Figure 5 it can be seen that the Marcon Wind Power concept is the second most expensive concept at 25m water depth and at greater water depths the most; even with its low installation cost it does not compensate for the high fabrication cost. Unlike most studies the installation cost saving of integrated WTG and foundation installation has been included in this study at circa £30million per 100 units, i.e. a cost increase of £0.007million/MW. Distance to port used is 150km, it is therefore likely that for sites further from the fabrication port, the installation advantages of the MWP will become increasingly significant and may enable the concept to be competitive, as demonstrated in the RES St Brieuc study. To provide another indication of the extent of this competitiveness the cost increase for the various foundation concepts has been made based on the vessel requirements indicated in the Grontmij study [1]. This approach assumes the vessels transit speeds in Table 7, the additional 300km round trip distance and that the HLV can transport 4 foundations or WTGs at time. This equates to approximately an additional 0.4 days of transit time per return trip at the 8m/s transit speed. Vessel day rates are the same as those used in the Grontmij study and include fuel, Table 7.

VESSEL	TRANSIT SPEED (M/S)	DAY RATE (K£)
SMALL TUG	12	18
60T BOLLARD PULL TUG	12	36
PILE DRIVING	8.5	144
WORK BOAT	12	10
WTG INSTALLATION	8.5	122
SCOUR PROTECTION	12	62
JACKET INSTALLATION	8.5	140

Table 7 Vessel assumptions for 300km cost comparison

Based on these assumptions Figure 6 has been produced. As in the 150km case the MWP concept appears to be the most expensive foundation option under these conditions, with the 261

increase in distance only increasing the total cost by around £0.03million/MW for the other foundation concepts. It should be noted that this analysis does not account for the more exposed conditions further from shore which are likely to lead to increased weather downtime and potentially a more significant cost increase than just increase in transit time.

### [REMOVED]

### Figure 6 Cost comparison of OWA screened concepts at various water depths at 300km from port

It should also be noted that as this study was undertaken for UK conditions, no consideration is given to the potential benefit of minimal environmental impact and therefore reduced impact of piling restrictions or cost of noise mitigations systems that are required for other sites. Potential estimates for the cost of piling noise mitigation systems are circa £3.7million for 100 units [7], but this doesn't take account of the cost to the project for the additional installation time required due to the additional offshore operations. Accounting for these costs it is only likely to account for a few percentage points change to costs and so will be insignificant to the relative competiveness of the concepts.

From the gradient of the MarCon cost line it would appear that for less than 55m water depth cases the buckling of the legs does not become a significant issue and drive a non-linear cost increase with increasing water depth.

### 3.5 Statkraft Independent Design Review

As part of Statkrafts interest in the MWP Mk. 2 concept, they engaged with MWP to produce a design basis for UK Round 3 Dogger Bank site requirements. The design is based on 5MW WTG for 35 and 45m water depths. Statkraft then undertook a detailed design review including modelling of the structures to check ULS utilisation, dynamic response and fatigue. To ensure accuracy of the models, Statkraft engaged directly with Bassoe Technology, who
undertook all the design work for the MWP concept for Macron Wind Power. Based on the Dogger Bank requirements the weights of the two structures are shown in Table 8.

Table 8 MWP Normalised weights for Dogger Bank design requirements			
	5 MW	DEPTH (M)	
		35	45
	HULL WEIGHT	1.00	1.05
	TOTAL LEG WEIGHT	1.00	1.13

For the ULS check, DNV-OS-J101 load cases 1.3 and 6.1 were checked for both standard wave loading and higher order wave loading based on Faltinsen, Newman and Vinje (FNV) theory [8]. This showed that only small "ringing" effects were evident of around 10-20% for the 35m water depth and ULS load case 1.3. The utilisation of the structure was around 50-60% and met the 1P and 3P frequency requirements. For FLS the 35m water depth structure was checked, with preliminary results indicating that the structure has insufficient fatigue life at the mudline and just below the hull on the leg sections, with fatigue lives of 6 and 10 years respectively. The implication of this is that the structural weight is likely to increase through the use of stiffeners and increased steel thickness of the legs to reduce the stresses within the steel. Given that the preliminary designs submitted for all of MWP studies have only been based on ULS and frequency requirements it is likely that the structural weight and therefore cost is underestimated, the degree to which cannot be determined without detailed modelling of the structure under site specific fatigue loads.

### 3.6 Södra Midsjöbanken Meteorological Mast Detailed Design Review

Through engagement with SS (operations division of Marcon Gruppen) and a review of design documentation, an understanding of the design approach and assumptions used for the meteorological mast case, based on the ABS jack up vessel requirements, and the concepts maturity has been improved. Based on the information reviewed the level of maturity of the concept appears quite high when compared to other novel offshore WTG foundations, with

detailed design undertaken and certified to meet the ABS requirements in all the key areas including hull yield and buckling, stability analysis, fatigue analysis etc., for tow-out, jack up and operational load cases. The detailed design review, Appendix A-1 [removed], has however raised the following queries;

- Simplified fatigue analysis approach indicates areas where detailed fatigue analysis is required, but not presented
- Wind and current loads not demonstrated to result in insignificant stress and therefore need to be assessed in the fatigue analysis
- Structural utilisation indicated by finite element method (FEM) is greater than allowable stress in some areas
- The hybrid nature of this concept means that transportation, installation and operation influence the design and as a result care needs to be taken with the interaction of the various design standards
- Damping effect of water as ballast
- Corrosion allowances
- General inconsistency between reports on weights, steel thicknesses, loads and coefficients

These queries and comments have been raised with MWP, but a formal response has yet to be received. Some aspects were discussed in a design review meeting with MWP, but the presence of Statkraft at the meeting limited the detail which could be acquired because of the sensitive intellectual property of the design. Given the structure has been certified by ABS, with all design documentation reviewed against vessel requirements, it is likely that the majority of the queries are answered in documentation that was not provided as part of this

design review, although areas such as fatigue, which are more critical for offshore wind structures still may pose a risk.

Based on engagement with MWP it has been commented that the predicted weight was within 0.5% of completed light unit weight (x tonnes which includes ballast system, jetting system, jacking system, marine out fittings, leg well, hull and legs). This provides confidence in their ability to provide accurate quantity estimates in design and so reasonable confidence in cost estimates. Although the reports reviewed had inconsistent weights, it is believed these errors are a result of not updating all figures when revisions to other reports have occurred. From a geotechnical design ability point of view, the penetration of the legs of within 0.5m of the predicted 3m value, with 2.5m achieved, provides reasonable confidence in their abilities. Given the Eigen period analysis assumes a penetration of 3m it is likely that the overall structural stiffness will be slightly less and so the 1st period will be slightly longer than the 0.312Hz (3.21s) design. Hull and leg sizing and general design is also based on an assumption of 3m penetration and so conservativeness in the structural utilisation maybe reduced, as the rigidity of the leg penetrations is shown to have reasonable influence on the leg stresses, but without detailed structure-soil interaction modelling the extent of which is hard to determine.

During tow-out of the meteorological mast foundation, it was stated that tow out stability was better than predicted, which could potentially reduce weather sensitivity during transportation, or reduce the hull sizing for the cases where transportation stability is the design driver. It was also stated that frequency response monitoring was incorporated with periodic acquisition of tow-out, jack up and pre load data. Access to this data has not been available to provide validation for floating stability; however, MWP reported that the measurement data showed good agreement with predicted responses. The stability design report indicated that the meteorological mast tower top accelerations are greater than various reported WTG allowable accelerations [9], Table 8, at transit in 3m significant wave height (Hs). The acceleration limits can be achieved by restricting transportation to sea states less than 1.5m Hs or by increasing hull stability through standard shipping practices such as

- Bilge keel a long fin of metal, often in a "V" shape, welded along the length of the ship at the turn of the bilge
- Outriggers rolling is reduced either by the force required to submerge buoyant floats or by hydrodynamic foils
- Antiroll tanks tanks within the vessel fitted with baffles intended to slow the rate of water transfer from the port side of the tank to the starboard side. The tank is designed such that a larger amount of water is trapped on the higher side of the vessel. This is intended to have an effect completely opposite to that of the free surface effect

Paravanes may be employed by slow-moving vessels (such as fishing vessels) to reduce roll Stabilizer fins (fixed or active) - active fin stabiliser will effectively counteract roll for ships under way, some modern active fin systems have been shown capable of reducing roll motion when vessels are stationary.

Use of the passive systems would be likely to come at limited cost in comparison to the possible increase in installation and transport stability. With the structure already consisting of jack up legs, one possibility could be to incorporate lowering of these in deeper water, outside of port, for increased stability through a keel like affect, but at the expense of tow resistance. Given the structure also has large ballast tanks for the pre-loading installation procedure; another possibility could be to exploit them as anti-roll tanks at minimal cost. It

should also be noted that during traditional installation of WTG, accelerations have been reported that are ten times those indicated in Table 9. It is therefore likely that there may be flexibility in these limits, but until detailed integrated design between MWP and a WTG manufacture is undertaken the extent of these limits remains unknown.

#### Table 9 Turbine Nacelle Transport Acceleration limits [9]

#### [REMOVED]

During installation there were no reported issues and complete installation of the mast and the foundation was achieved within 16 hours of arriving on site, demonstrating the potential installation benefits of such an integrated installation technique. Eight hours of this was due to ballasting operations and so a potential optimisation could be achieved through larger pumps and valves. On a commercial WTG scale the additional cost, would be more than offset by the reduced installation time. This represents a significant achievement given the issues reported during other novel meteorological mast foundation installations, such as Keystone and Universal Foundation. In comparison the Universal Foundation meteorological mast installation at Dogger Bank was reported to take 28 hours including the top side, but not the mast.

The main lesson learnt from fabrication of the meteorological mast was achieving the alignment of the leg well and jacking system fabrication timing to minimise delays. From a client perspective the provision of a daily site representative to ensure timely response to technical queries and delivery of quality is important.

Fabrication and installation costs from the meteorological mast have been used to provide more accurate inputs in to the costing model used by the OWA and MWP studies for the wind turbine foundation. From the breakdown of costs provided by MWP for the Södra Midsjöbanken meteorological mast project costs (Table 10) it can be seen that the percentage breakdowns are in rough proportion to the OWA estimates made.

COMPONENT	Cost Percentage (%)
Hull	Х
Jacking Units	х
Hydraulic Power Pack	х
Lattice tower	х
Meteorological Equipment	х
Classification/certification costs	х
Basic design/engineering	х
Installation/marine operations	х

Table 10 Cost break down for Södra Midsjöbanken

Based on Table 10 it can be seen that the jacking units represents a significant cost and as such MWP have investigated the option of them being detachable, so that it is transferred from foundation to foundation as part of the installation process. Originally the jacking units remained part of the foundation. They also believe going to a clamping system will result in reduced costs, as leg strength will be improved by omission of the pin holes, therefore reducing material and fabrication costs. Although there is the increased risk of slippage and design issues to be resolved with regards to procedures, reliability, safety and inspection requirements, the system has been proven in jack up vessels and is certified by GL.

The operation of the meteorological mast for almost two years has also proved to help de-risk the structure, as there have been no reported issues with settlement or unexpected degradation during the bi yearly service visits.

Based on the review taking the step form meteorological mast to wind turbine foundation raises the following key questions;

• The effect of stiffness and material requirements (and subsequent cost implications) of avoiding 1P and 3P frequencies e.g. is a sizeable increase in material volumes required to achieve required stiffness

- The effect of variation in penetration depths from design values to achieved installed values on system response such as natural frequency and leg strength utilisation
- The extent of torsional analysis carried out and how torsion loads from a wind turbine will affect the design of the hull and legs.

Based on the findings of the review it was recommended that:

- Queries raised throughout the document shall be raised with SSE at a design review meeting
- If design risks raised are not satisfactorily answered additional work should be undertaken by MWP
- Condition monitoring should be deployed to understand the behaviour of the structure and the degree of risk.

Condition monitoring will have the benefits of:

- Structural validation
- Stability validation during transport
- Model validation of wave loading, soil-structure interaction, wind loading and FE models
- Conservativeness in design
- Extension of service life
- Potential broadening of transportation, installation and site conditions the structure can be deployed in.

## 3.7 Södra Midsjöbanken Meteorological Mast Structural Condition Monitoring

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As part of the outcome of the detailed design review, Statkraft's independent design review of a WTG structure and the benefits of condition monitoring of the structure highlighted in Section 3.6, it was decided to proceed with condition monitoring in June 2013. ENT were advised that EC&R would be reducing the number of Nordic development sites, which included Södra Midsjöbanken, introducing uncertainty in the future of the meteorological mast. It therefore became apparent that if any useful information was to be gathered, the structural condition monitoring system needed to be installed by the autumn of 2013 to capture the large storm events before the end of the meteorological monitoring campaign in April 2014.

As part of this process, ENT produced a technical specification, Appendix A-2 [removed], based on experience from other offshore wind structural condition monitoring projects and the detailed design review of the meteorological mast. Initial budget for completion of the monitoring works was circa £250,000. A tendering process was undertaken with Strainstall and Fugro, who had both demonstrated their abilities in offshore structural condition monitoring through various E.ON projects. Tender returns identified the actual costs for the proposed system in the region of  $\oplus$ 00k. ENT reduced the scope of the monitoring campaign to bring the costs in line with the available budget. The revised costs returned were in the order of  $\oplus$ 50k plus vessel costs at around  $\oplus$ 50k including 50% weather delay. Based on evaluation of the reduced scope submission, Fugro were selected as the preferred bidder due to their superior technical and HSSE methods, see Appendix A-3 [removed] for more details.

A detailed design meeting and site visit was held in Malmo in early August with MWP, Statkraft, ENT, Bassoe Technology and Fugro, where final details were clarified and any access and installation challenges discussed and resolved. A project proposal including costs, expected outcomes and benefits were presented to Statkraft, MWP and RES, who all expressed interest in participating in the research project. Unfortunately the planned use of one of SS's jack up vessels, due to visit Södra Midsjöbanken at a time when it could also be used for the condition monitoring installation, was delayed on another job. Requiring an alternative vessel to be sourced which was less suitable for the job. Despite MWP formally agreeing to cover all vessel costs and Statkraft all data analysis and modelling costs, Stratkraft's and RES's initial indications of financial contributions were not confirmed and so the remaining cost was beyond what was internally agreed to be cost effective. A decision was therefore made in October 2013 not to install the condition monitoring until the future of the structure is confirmed in mid-2014 and outcomes of the various cost benchmarking and meteorological mast evaluation studies finalised.

# 4 CONCLUSION

The deployment and successful operation of the Södra Midsjöbanken meteorological mast has enabled aspects of the fabrication, transportation and installation of an offshore foundation suitable for a mobile meteorological mast or offshore wind turbine to be de-risked.

As a meteorological mast the competitive tender process used for the Södra Midsjöbanken development site demonstrated that for these conditions the MWP foundation was commercially favourable to a traditional monopile foundation and has the additional benefit of being able to be redeployed at another location. Technically its successful, transportation, installation and operation over the last two years along with fewer installation issues reported than other novel met mast foundations installed, including Keystone and Universal Foundation, has demonstrated its maturity as a meteorological mast structure.

The detailed design reviews of the MWP Mk. two concept highlights that fundamentally the concept is feasible and can meet the key ultimate limit state and frequency requirements.

However, they indicate that fatigue limit state requirements are challenging to achieve, so further work is required to ensure these are satisfactory and the implications are included in the structural weight and therefore cost estimates.

For a WTG foundation, technically the concept provides a feasible alternative to traditional foundations offering minimal noise impacts and ease of installation.

From a commercial perspective, engagement with Marcon Wind Power and an independent study by RES suggests there could be potential for cheaper cost / MW for an offshore WTG foundation for far shore sites than traditional jacket structures in deeper waters. However the Grontmij study, as part of the OWA programme, has indicated that at less than 150km from shore and typical UK Round 3 conditions the comparatively high fabrication costs outweigh the installation benefits. In other locations, where piling restrictions may apply and given the uncertainty in the direction the governing bodies may take with environmental impact restrictions and therefore the mitigation required, the MWP concept may become slightly more favourable. Based on the available information, ENT has attempted to estimate the distance from port at which the concept may become financially competitive. This indicates under favourable conditions, the concept could become competitive at greater than 200km, but for Round 3 conditions this may be more like 300km in only very shallow water depths. The number of potential sites is therefore likely to limited and would be unlikely to be high on the development priority.

It is therefore evident that significant fabrication cost reductions need to made to enable any serious consideration to be provided. Utilisation of a temporary jacking system may help to achieve this, but new technical challenges will need to be addressed.

These studies serve to highlight the importance of the site specific conditions when considering foundation options and the potential for novel foundations including the MWP to have a lower CapEx than more traditional foundation options.

Compared to other novel foundations the lack of structural condition monitoring validation and lack of use of this type of foundation concept as WTG foundation, mean that its maturity falls slightly behind that of Keystone and Universal Foundation, but given the additional support that has been provided by the OWA, this bodes well for achieving commercial maturity relatively quickly if this momentum can be maintained and support can be found.

If structural condition monitoring could be implemented it would not only bring the technical maturity up to a similar level, but it would also help to validate fatigue and ultimate limit state utilisations allowing for more accurate cost estimates and improved comparisons to be made.

### RECOMMENDATIONS

Given the potentially favourable CapEx possibility of the MWP Mk. 2 concept for far shore sites it is recommended that

- MWP be included in future foundation screening for both meteorological mast and WTG foundations for far shore sites
- Continued technology tracking undertaken through supplier engagement to ensure the latest developments are available for foundation option decisions, improve certainty in cost estimates and ensure any opportunities to realise potential cost savings are utilised
- Consider the feasibility of structural condition monitoring of the MWP Mk. 2 Meteorological mast at Södra Midsjöbanken if circumstances change, MWP demonstrate fatigue requirements can be met and cost benefits of the revised jacking

system. As this will aid in de-risking of the structure and improve knowledge on

structural performance and certainty in environmental limits of the foundation.

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