Prediction of Wear in Grouted Connections for Offshore Wind Turbine Generators

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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 0.4 mm over 75% of the connection, but a much greater loss in thickness, of the order of 4 mm, 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.

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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 Fig. 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 Fig. 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.

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 Fig. 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 Fig. 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 1.2 mm 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 0.3 Hz 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 160 kN, which allowed testing samples with a grout-steel interface of 150 × 150 mm, up to maximum compressive stress level of

![Experimental test arrangement](image)

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

The compressive force applied to the samples could be varied by tightening the compression bolts (Fig. 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 re-tightened 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 (Fig. 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 20 Hz during testing.

Eight samples were tested in order to be representative of the various surface and environmental conditions that the grouted connection would be subjected to (details are shown in Table 1). The steel samples were shot-blasted to a Sa 2 1/2 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 8000 cycles at 1.2 mm peak-to-peak axial amplitude for each 0.5 MPa 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 detectable, 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].

The resultant wear rates derived from the experimental testing for the wet and dry samples are shown in Fig. 4. The weight of ejected material presented in Fig. 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.

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 Fig. 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 Fig. 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.

#### 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 (Fig. 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 Fig. 6.

### Table 1

| Sample | Characteristics | Reasoning |
|--------|-----------------|-----------|
| S1     | Mill scale, dry, unconfined | Test of logging equipment & rig |
| S2     | Mill scale, dry, unconfined | Test influence of controller amplitude and frequency |
| S3     | Sa 2.5, dry, non-corroded, confined | Influence of surface finish and higher loads |
| S4     | Sa 2.5, wet, non-corroded, confined | Influence of water presence |
| S5     | Sa 2.5, wet, corroded, confined | Influence of corrosion |
| S6     | Sa 2.5, dry, corroded, confined | Influence of corrosion and water presence |
| S7     | Repeat S5 | Improve significance of results/determine |
| S8     | Repeat S5 | Influence of grout material properties |

Fig. 4. Wear rates derived from experimental testing based on weight of ejected material.
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 1 m/s) were initially used to correct vertical strain readings (SGA-V) (Fig. 7) to account for any offset caused by datum setting of the SCM. These strains were measured on the inside wall of the TP 1.5 m above the top of the MP (this location is shown in Fig. 6 and indicated by the K1-S1-6-SGA-V label). The correlation relationships between wind speed, strain and displacement, shown in Figs. 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 ± 1° for both power (Fig. 8) and non-power generation events (Fig. 9). Trend curves (plotted with solid lines in Figs. 8 and 9) were derived to provide a conservative output, and were therefore consistently placed.
The moment due to the contact pressure, $M_p$, is derived from the integration of the pressure distribution along arc $bcd$ in Fig. 10:

$$M_p = \frac{p_{\text{nom}} R_g L_g^2}{3};$$

(3)

where $p_{\text{nom}}$ is the normal stress within the grouted connection; $R_g = 2.15$ 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. 

Equation (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 Fig. 10): 

$$M = M_p + M_{\text{uth}} + M_{\text{uv}}. $$

(2)

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 been included in these calculations, but should be considered for jacket pile grouted connections [3]. In Eq. (1), let’s now consider $p_{\text{nom}} = p_{\text{nom,0g}}$ and $M = M_{\text{tg}}$ 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 $d = 1.5$ m above the top of the grout. Indeed, neglecting the effect of any distributed load along the height of the structure, the ratio of $M_{\text{tg}}$ and $M_{\text{tp}}$ 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 ($H = 79.4$ m) and the distance $d = 1.5$ m 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)
Fig. 8. Relationships derived from correlations of wind speed for (a) strain and (b) vertical displacement for power generation events with a constant wind direction.

Fig. 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.
On the other hand, $M_{tp,d}$ can be related to the bending strain $\varepsilon_d$ measured at the inside wall:

$$M_{tp,d} = \frac{E_s I_{tp}}{R_{tp,i}} \varepsilon_d, \tag{7}$$

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

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

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

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

$$p_{\text{nom},tg} = \frac{3 \pi E_s I_{tp}}{R_{tp,i} R_p l_g \frac{H - d}{H} \left(\frac{1}{\pi} + \frac{\mu}{\mu_H} + \frac{3\mu_H R_p}{H}\right)} \varepsilon_d \tag{9}$$

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

$$p_{\text{nom},y} = \frac{M_F}{M_{tg}} p_{\text{nom},tg} = \frac{H + y}{H} p_{\text{nom},tg}, \tag{10}$$

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:

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

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

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

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 Fig. 11.

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 Fig. 12). The location of these strain gauge reading can be seen in Fig. 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.

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:

1. For any given wind speed and direction, a resulting compressive stress was calculated (using Eqs. (9) and (12) and the relationships derived from Figs. 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.
2. The computed 10-minute average wear rates for each 0.1 m 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.

3.5. 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 ±5 microstrain and wind direction ±2.5°, 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 Fig. 13.

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 1/6th 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 Fig. 14.

Fig. 14(a) shows the good agreement between the model’s predicted wear and the SCM detected wear for the two K1-RD locations. Fig. 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 0.38 Hz structural response, matching the magnitude of detected wear for all of the periods, examples of which are shown in Fig. 15. This frequency coincides well with the first mode of the substructure’s natural frequency response at 0.29–0.33 Hz, and the blade passing (1P and 3P) driving frequencies from the WTG at 0.14–0.31 Hz and 0.43–0.92 Hz.

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 Fig. 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 Fig. 16).

In addition, given that the duration of the observation period is three years, data automatically take into account inter-annual and inter-seasonal variations [30] and are well above the minimum duration of six months stated in reference [34] to achieve an acceptable level of representativeness. Regarding the foundations themselves, the SCM data analysed to date has 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 Fig. 17. These results are based on the value of outputted wear from the model calibrated for a 0.38 Hz response, which was chosen given the accuracy it showed against the SCM detected wear.

From Fig. 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 4.8 mm, with around 1 mm on the opposite side indicating a possible gap of dynamic movement of 5 to 6 mm 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 700 mm of depth (as shown in Fig. 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. Conclusions

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 modeling a selection of WTG substructure locations across a typical wind farm, it has been shown that wear accumulation in the order of about 4 mm 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 0.4 mm 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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Fig. 17. Spatial variation of 20 years of accumulated wear for 11 foundations at a typical offshore wind farm.
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