RLT on prefab concrete piles of wind turbine foundations in tidal sands in the Netherlands

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ABSTRACT

During pile driving for wind turbine foundations in pre-dominantly tidal sands, unexpected low blow counts as low as one blow per 0.25 m were observed. To restore confidence in the foundation various aspects were investigated. Within the scope of this investigation several rapid load tests (RLT) have been executed in accordance with NPR7201:2017 and ISO22477-10:2016. From these tests it was concluded that the pile bearing capacity was sufficient. The maximum applied test loads, without geotechnical failure occurring, exceeded the maximum pile bearing capacity predicted by the calculation methods of the Dutch geotechnical standard NEN9997-1. Alongside bearing capacity, the dynamic axial stiffness of the pile in unloading – reloading is of great importance in the design of piled wind turbine foundations. A new simple method has been proposed to directly extract the unloading-reloading stiffness from the RLT test.

Keywords: Onshore wind turbine foundations, cyclic loading, dynamic stiffness, RLT, pile foundations, Gaag effect

1 INTRODUCTION

Wind farm Oosterhorn, situated in the north-east of the Netherlands (Groningen), is a project of several stakeholders (Windpark Mondriaan BV, Eneco windpark Delfzijl BV). The overall supervision of the project for these stakeholders was performed by Natural Elements BV.

Wind farm Oosterhorn consists of 18 turbines of the Vestas V136 type with a capacity of 4.3 MW and hub heights of 145 meters. Windbase/ ABT BV was responsible for the wind turbine foundation design. The responsible contractors for the foundation works, crane hardstands and pile driving were VHB/ VSF. Finally, the rapid load testing was performed by Allnamics BV.

2 FOUNDATION DESIGN

2.1 Soil conditions

Typical to this part of the country is the variability and heterogeneity in soil stratigraphy dating back to glacial periods. Geological processes created a landscape with huge gullies, which can be several kilometres wide and up to a hundred meters deep and which are usually filled with stiff clay sediments called 'Potklei'. Furthermore, closer to the Waddenzee the soil stratigraphy is dominated by the tidal influences of the sea in the past. A typical set of CPT's from this area is shown in fig. 1.

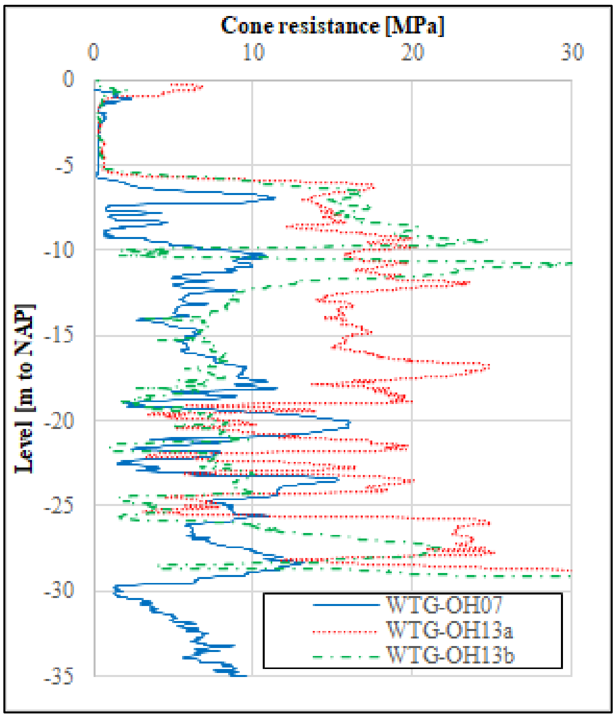

Fig. 1. Typical soil profile consisting of Holocene layers overlying tidal sands.

From the CPT's the heterogeneity of the soil profile with strong and weak zones as function of depth is visible due to the large differences in the measured cone resistances. Moreover, the variability in soil strength is significant, even within one site. This is indicated by the large difference in the cone resistance from two CPT's at turbine location OH13. Typical for the project area are the relatively loosely packed sand layers with cone resistances being well below 10 MPa.

2.2 Foundation design

Given the shallow weak clay and peat top layers, the most common foundation method for wind turbines in this part of the Netherlands is the use of a concrete pile cap founded on friction piles. The number of piles is based on the required bearing capacity as well as the (dynamic) stiffness demands of the given turbine. The maximum uplift and compressive capacities of the piles must be designed to be able to withstand the maximum overturning moment of the turbine. In this case the piles have been designed according to the Dutch geotechnical standard NEN9997-1.

The foundation design also complies with the "no gapping criterium" according to the IEC61400-6, also known in the industry as the SLS S3 state. This criterium limits the number of "two-way loading" actions on the piles from the total fatigue load spectrum to a maximum of 87 hours each year, i.e., 1%. This criterium is a conservative and pragmatic approach to prevent the risk of significant cyclic degradation of the piles. The axial behaviour of a single pile under a wind turbine foundation, given no significant cyclic degradation takes place, is shown in fig. 2.

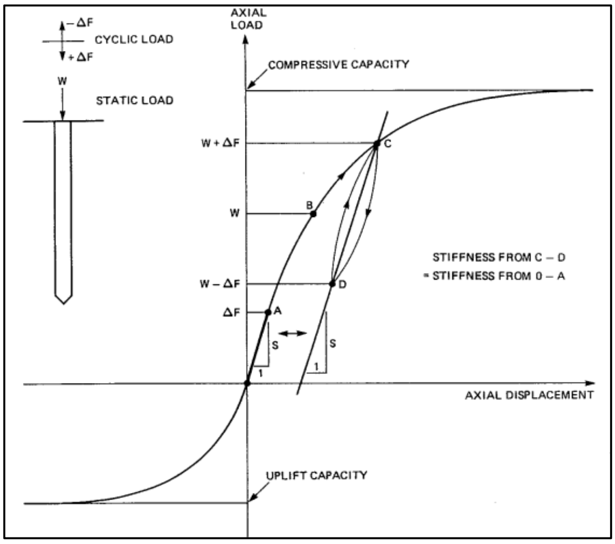

Fig. 2. Load settlement model for piles used in wind turbine foundations (Lam and Martin 1986).

To obtain this load-settlement curve of the pile in Dutch practise the simple design rules from the NEN9997-1 are often used. In case of a more complex soil stratigraphy, as is the case here, load-transfer methods using T-Z curves for the local pile-soil interaction and Q-B curves for the pile-tip mobilisation curves should be used (Tomlinson and Woodward 1977).

Given that no significant cyclic degradation of the pile's stiffness takes place, the stabilized response of the circular total pile group stiffness can, with reasonable accuracy, be determined with eq. 1. In this expression the rotational stiffness of the pile system is defined as the summation of the individual stiffness contributions of each pile *i* with its inclination factor s_i and distance to the centre axis R. The hypothesis herein is that, all piles subjected to an alternating frequently occurring wind load, will, after a few load cycles, react nearly elastic and show a stabilized response (Lam and Martin 1995) with the average stiffness being $k_{v;cycl}$. This is visualized by the trajectory C-D in fig. 1. The total stiffness of the foundation however not only depends on the piles but also on the behaviour of the concrete foundation cap. The resulting stiffness of the total system ($K_{r;dyn}$) is computed with eq. 2:

$$K_{r;dyn;GEO} = \sum_{i=1}^{n} {R_i}^2 \cdot s_i \cdot k_{v;cycl;i} \qquad (1)$$

$$\frac{1}{K_{r;dyn}} = \frac{1}{K_{r;dyn;GEO}} + \frac{1}{K_{r;STR}} \qquad (2)$$

The stiffness component of the concrete cap, denoted $K_{r;STR}$, is best determined with FEA methods in which a displacement-controlled rotation is applied. The resulting stiffness of the cap is simply determined from the resulting moment – rotation relation.

For this project, based on the design calculations, a total of 42 prefabricated concrete piles with dimensions of 450x450 mm with lengths up to 30 m were sufficient to meet the total stiffness and bearing capacity demands. This leads to piles with a slenderness ratio of approximately 60. The diameter of the foundation equals 19.0 m.

2.3 Pile driving predictions

Given the relatively slender piles and the large variability in soil strength, as discussed in chapter 2, detailed site to site pile driving predictions were performed adopting Smith's model (1960) and using the SRD-model of Alm and Hamre (2001) to select an appropriate pile driving hammer for the project. For GRLWEAP a shaft and base damping of 0.16 and 0.5 s/m in sandy soil was used. For the quake's a value of 2.5 mm for the shaft and 2.5 mm for the pile toe were adopted.

For almost half the turbine sites the piles had to be driven in loose sands. At some sites dense sands -with cone resistance up to 40 MPa over several meters- were encountered. This resulted in a large spread of blow-count predictions. A D46-32 diesel hammer would be sufficient for half of the sites, but insufficient for the sites with dense sand. In these cases, a D62-22 was advised to drive the piles to design depth while minimising risks of refusal and/or damage to the prefabricated piles due to high total blow counts.

3 FIELD MEASSUREMENTS

3.1 Blow counts

Pile installation started at the sites with the dense sand profile by using a D62-22 hammer. Expected normal blow counts of 10-50 were recorded. Subsequently sites with loosely packed sands resulted in lower and on some occasions (4 turbine locations) unexpected low blow counts of 1 blow per 0.25 m for both the D62-22 and D46-32. Low blow counts generally indicate low pile bearing capacities. Therefore, further investigation into this problem was necessary.

The set-up process of the piles on these 4 locations was subsequently investigated by stopping pile driving a few meters above required pile tip level and redrive them respectively 1 day and 4 days later to the desired depth. From the redrive tests an increase in blow count with time was observed as shown in fig. 3 for location OH07.

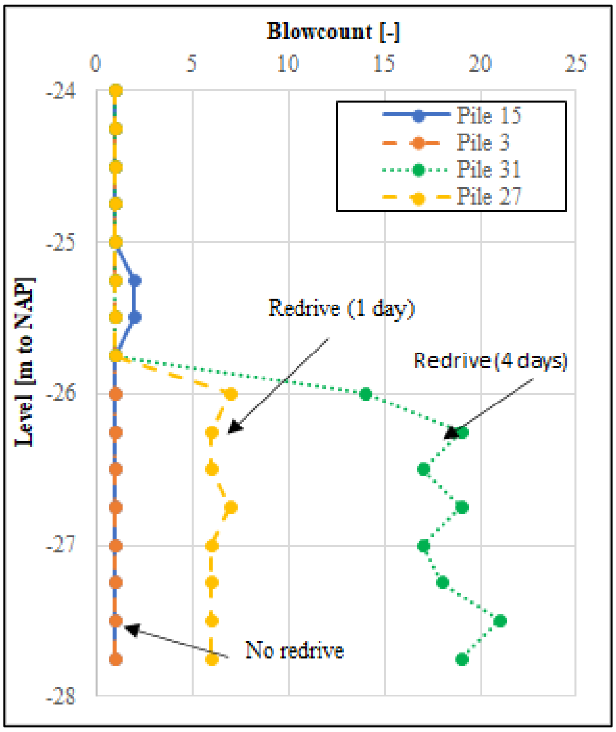

Fig. 3. Observed blow count and set up pattern using a D62-22 impact hammer for the redriven piles.

3.2 Verification CPT's

Thereafter additional verification CPT's were installed just adjacent to the driven piles. Typical results of the CPTU's before and after pile driving are shown in fig. 4. In this graph the displayed CPTU after driving is located at approx. 2.0 m from the initial CPT's position for a direct as possible comparison.

It is evident from fig 4 that a reduction of the cone resistance can be seen after pile driving. In Dutch engineering practise usually an increase in the qc is observed after pile driving of precast driven displacement piles. Given that the qc-reduction could be seen in most of the sets of initial and verification CPT's, the cause is unlikely to be the natural spatial variability of the sand layers under consideration. The observed phenomenon of loss of driving resistance in combination with a decrease in cone resistance has rarely occurred in the past. The phenomenon is in Dutch geotechnical practise known as the 'Gaag effect', though the mechanism that is causing the effect is not yet fully understood (Geerling et al 1997).

3.3 RLT test campaign

Given that both pile stiffness and total bearing capacity are critical in turbine foundation design additional investigation was required. To proceed with confidence and to verify the bearing capacity of the piles several rapid load tests (RLT) have been executed in accordance with NPR7201:2017 and ISO22477-10:2016. The turbine location where the lowest overall blow count was observed, was designated as the normative test site. The test layout is shown in fig. 5, the red circles indicate the tested piles, the blue triangles on the inside of the foundation are the additional verification CPT's. For practical reasons only the vertical piles can be tested with the RLT.

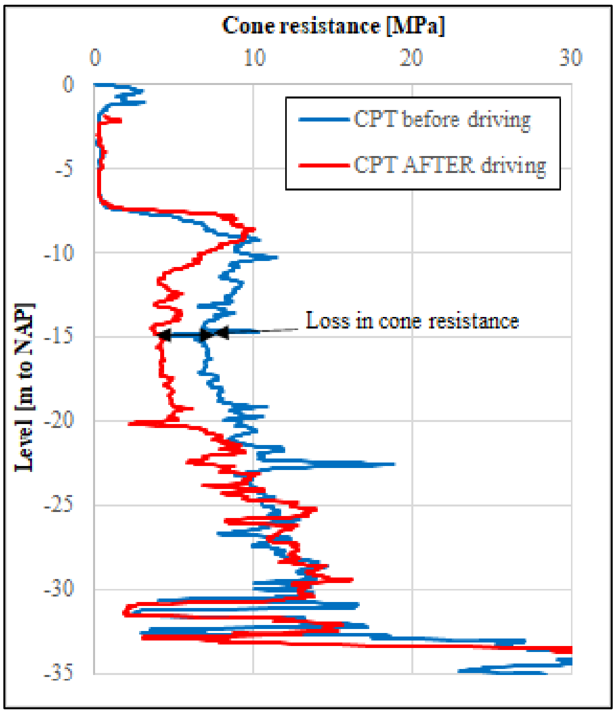

Fig. 4. Observed net cone resistance before and after pile driving.

Rapid Load Testing has been performed with StatRapid, a modular drop weight system, a further development of the work of Gonin et al (1984). In the set-up deployed in this project (Fig. 6) -20 tons drop weight in combination with a spring system with a combined stiffness of 16.8MN/m- the device can generate test loads up to 5 MN.

During impact the load on the pile head, the acceleration and displacement of the pile head are recorded with a sample rate of 12.5 kHz, allowing for evaluation and interpretation of the results following the Unloading Point Method (Middendorp et al 1992) in accordance with ISO22477-10. The force on the pile (*F*) can be described by the sum of the static resistance $k{\cdot}u$, the damping component $c{\cdot}v$, and the inertia component $m{\cdot}a$ (eq.3).

$$F = k \cdot u + c \cdot v + m \cdot a \tag{3}$$

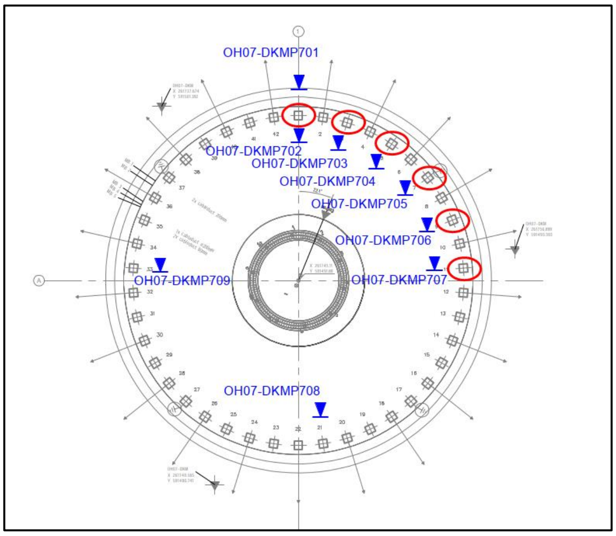

Fig. 5. RLT test layout and additional CPT's.

At maximum displacement the velocity equals zero. Since the mass is known and the acceleration and load on the pile are recorded, the static resistance can be determined (static point).

Fig. 6. StatRapid system positioned over the test pile.

4 RESULTS

4.1 RLT Pile bearing capacity

Dutch code of practice NPR7201 covers 4 test classes of different complexity and purpose. Class C allows for verification of pile capacity for a designated location. The basis for the test load, to be applied in no less than 5 cycles of increasing magnitude, is the design load F_d multiplied by a variation factor κ dependent on the number of tests executed. The test load is subsequently corrected for the possible development of negative skin friction and, in case of Rapid Load Testing, divided by the quotient of the loading rate reduction factor η and a correlation factor ξ_{RLT}.

$$F_{test;RLT} = (\kappa \times F_d + 2 \times F_{nk;d})/(\eta/\xi_{RLT}) \tag{4}$$

Provided this test load can be applied without the occurrence of geotechnical failure according to the definition of NEN-9997-1 (maximum pile toe displacement less than 10% of the equivalent toe diameter – in practical sense being equal to the permanent deformations recorded at the pile head) for all tested piles, the foundation can be deemed sufficient in terms of bearing capacity. For this project the test load was determined at 3,117 kN. The required test load was achieved for all six piles without exceeding the threshold for the pile toe displacement as displayed in fig. 7.

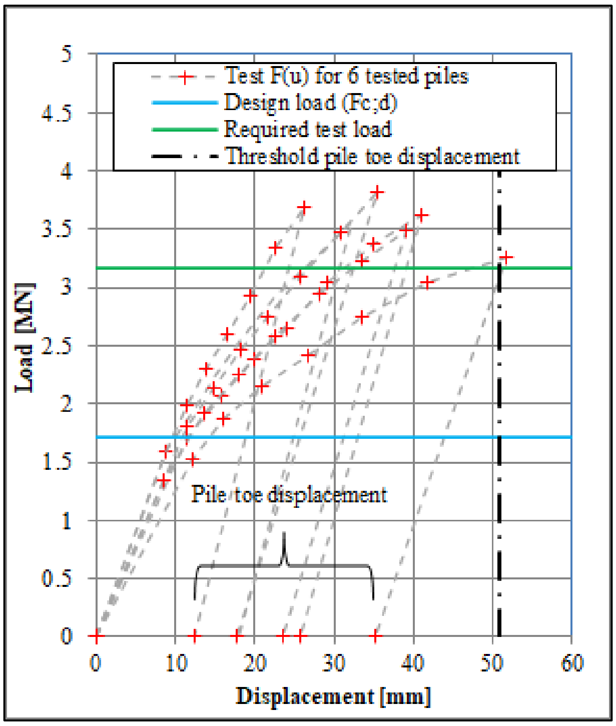

Fig. 7. Results Class C Rapid Load Tests.

4.2 RLT pile stiffness

From the RLT-tests the unloading/reloading stiffness of the pile can be determined from the static point to the permanent set of a load cycle. Because the characteristic cyclic stiffness is of interest, only the loading rate factor η of 0.94 has been applied in the analyses.

This procedure can be repeated for all load cycles, the result of this procedure for one tested pile is shown in fig. 8. The procedure follows the theoretical hypothesis of fig. 2 and the stabilized response method which states that the unloading-reloading stiffness of the pile should be used for the stiffness verification of the foundation. From fig. 8 it can be deduced that all unloading lines, by near virtue of the plasticity theory are almost parallel to each other. The resulting average unloading stiffness of pile 5 equals in this case 220 MN/m. From fig. 8 it is also apparent that the next loading cycle again passes through the previous static point. This indicates that the unloading stiffness is in fact the resulting governing stiffness for a pile loaded by a “sinusoidal” load such as wind. The same pattern in loading and unloading can be deduced from all RLT tests.

Table 1. Characteristic of the cyclic stiffness determination.

μ (MN/m)	σ (MN/m)	c_v (%)	$k_{v;cycl;char}$ [MN/m]
211,5	23	10,9	174

The resulting sample mean, standard deviation and coefficient of variation is shown in table 1. From the statistics of the test results, it can be deduced, provided a Gaussian distribution is assumed, that the characteristic value (5% lower limit) of the cyclic stiffness of the pile’s is equal to 174 MN/m. The calculated characteristic pile stiffness in the design calculations for this location equaled $k_{v;cycl;char}$ = 200 MN/m.

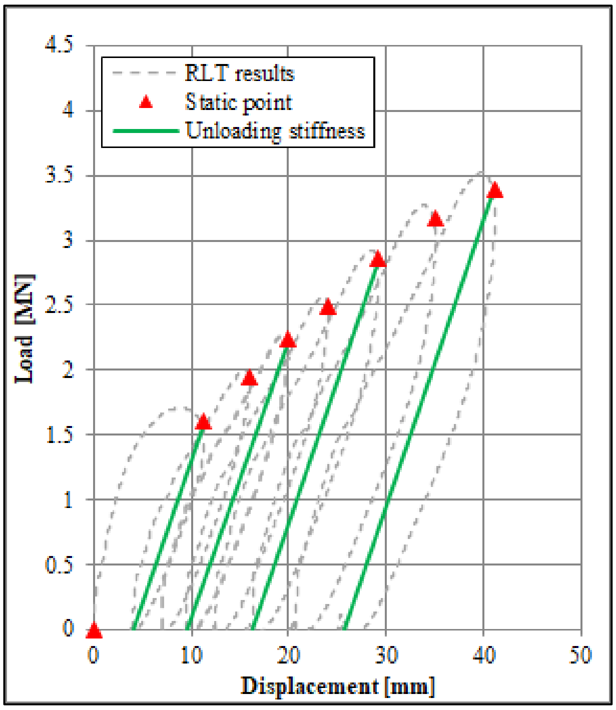

Fig. 8. Procedure of determining the unloading-reloading stiffness for pile 5.

5 ANALYSES AND BACK CALCULATIONS

5.1 Blow counts

In standard engineering practise the CPT, with maximum cone resistance, i.e., maximum expected soil resistance, is used to determine the proper hammer to reduce the risk of refusal. Within this project the variety in soil profiles, sometimes even within the same site, was relatively large. Therefore, lower blow count predictions were already acknowledged before pile driving. However, the encountered blow count of 1 in sandy soil was considered abnormally low. Repeating the prediction based on the verification CPT -with reduction in cone resistance- still resulted in expected blow counts of 5-6.

The most likely cause of the low blow count is therefore the specific soil type at the site. This concerns a loose and silty type of sand, deposited under tidal influence of the sea, in engineering practise known to be sensitive for liquefaction. Build-up of water pressure while pile driving leads to loss of effective stress and, as a result, negligible shaft friction. Limited applied energy will consequently lead to large deformation of the soil, hence a low blow count. This is previously also described by Jacobse and Van Dalen (2013) for the same type of tidal sands, in Dutch also called ‘wadzand’ sand layers.

5.2 Verification of foundation design

From the RLT test it was concluded that the bearing capacity of the piles was sufficient and the actual sustained loads by the piles were higher than calculated according to the calculation methods of the Dutch geotechnical standard applied to the verification CPT’s.

The derived characteristic cyclic stiffness of the piles $k_{v;cycl;char}$ can be used to directly determine the overall rotational stiffness of the pile system using eq. 1. The rake of the piles results in the factor *s* being equal to 0.99 for the batter piles and 1.0 for the vertical piles. The resulting rotational pile group stiffness therefore equals $K_{r;dyn;geo}$ = 296 GNm/rad. The stiffness of the block itself based on the structural analyses is found to be $K_{r;str}$ = 300 GNm/rad. The resulting rotational stiffness can now be determined by eq. 2 and computation leads to $K_{r;dyn}$ = 149 GNm/rad. In this case this is still sufficient for the applied turbine as a stiffness of 110 GNm/rad was required.

Focus of the RLT campaign has been on the verification of bearing capacity. In case RLT will be performed dedicated to the assessment of the unloading-reloading behaviour, a loading scheme with multi-cycle testing at constant test load -to better reflect the sinusoidal character of the wind loads- would be preferred.

6 CONCLUSIONS

The wind farm consisting of 16 4.3 MW turbines, is situated in a complex post glacial geological location in the north-east of the Netherlands. During pile driving unexpected low blow counts and a decrease in cone resistance were observed at 4 of the 16 locations for piles driven in tidal sands. The issue with low blow counts and loss in cone resistance could have been foreseen based on past experiences in case this specific type of soil would have been recognised from the existing soil investigation. Conventional blow count prediction methods based on the cone resistance using the Alm & Hamre SRD model (2001) proved to overestimate driving resistance and may need to be adapted for these specific soils in the future.

The redrive tests on the piles revealed a significant increase in blow counts after a set-up period of about 4 days. Given that the overall bearing capacity of the piles was verified with the RLT-tests, on-site redrive tests may proof useful in future practise to validate that pile set-up develops rapidly and the piles therefore regain their bearing capacity.

The RLT test results also proved to be useful in determining the unloading/reloading stiffness of the piles, which is important in wind turbine design. Using the described simple method, the resulting pile's unloading/reloading stiffness can easily be determined from an RLT. This stiffness can directly be applied in the design as described in this paper. Therefore, RLT is an interesting optimization option given the ever-increasing demand and size of modern (onshore) wind turbines.

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