DESIGN METHODS FOR SHORT SLENDER STEEL PILES IN CLAY

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KEYWORDS

Piles, pile buckling, steel pile design, clay, design method.

ABSTRACT

Slender steel piles in clay, which are driven or drilled to a firm soil stratum, are frequently used in Nordic soil conditions. The design is normally carried out with an analytical calculation model in which the soil response is included as an equivalent spring stiffness for a beam in an elastic medium. However, the natural alterations in the level of the bedrock frequently results in pile group configurations in which some of the piles are shorter than the elastic buckling length. The theoretical base of the pile buckling method is consequently not fulfilled. In this paper, the standard buckling method and the steel column method according to Eurocode are compared to a finite element model of the pile in the soil. The results show that quite different bearing capacities can be calculated, and some recommendations for practical design are given.

1. INTRODUCTION

The soil strata in most of the Nordic countries is characterized by the geological processes during the Weichsel ice age, resulting in soft Holocene sediments deposited on the very hard Precambrian bedrock [1]. In Figure 1, a typical soil profile from South-East of Sweden is illustrated, consisting of fill, dry crust, clay, silt and moraine. The clay layer is frequently very soft with an undrained shear strength, c_u, between 5 - 20 kPa, while the Precambrian rock is very hard with an unconfined compression strength of 150 - 250 MPa. For such a soil profile, piles are drilled or driven into the bedrock and transfer the pile load as a column.

The geotechnical bearing capacity, $N_{Rd.GEO}$, is frequently verified by dynamic pile load tests, allowing a very high utilization of the steel material [2]. The structural design resistance of the pile, $N_{Rd.STR}$, is often calculated by an analytical model in which the soil support is modelled as an equivalent spring stiffness with displacement included softening [3]. The calculation model yields similar results as the full numerical models as studied in [4] and [5]. However, the analytical model developed in [3] considers the buckling of a pile segment equivalent to the elastic buckling length, $l_{cr.s}$. The changes in the depth of the soil frequently results in that many of the piles have a length, l, which is shorter than $l_{cr.s}$ calculated from the c_u of the clay. This raises the question of how $N_{Rd.STR}$ should be calculated. There are essentially three options:

- The pile is assumed to have a length, l, which exceeds the elastic buckling length, $l_{cr.s}$, and the calculation model in [3] is valid.

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- The pile is considered as a column without any support and the calculation model for the specified material is valid, e.g. Eurocode 3 for steel piles [6]
- A full numerical model of the pile and soil is used.

The different options have advantages and disadvantages. The calculation model in [3] is used in practice and the soil parameters can be found from site investigation tests [1]. The calculation model for free columns is found in the literature [6] but is most probably too conservative. The full numerical model of the interaction between the soil and the structure is cumbersome for the practical use.

A comparison of the calculation models has been developed in [7] and is described herein. Initially the methodology is developed, followed by calculation results and conclusions.

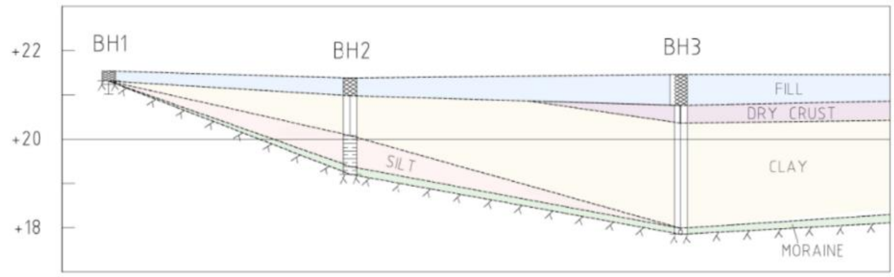

Figure 1. A typical soil profile in the South-East of Sweden, consisting of fill, dry crust, clay, silt and moraine deposited on bedrock.

2. METHODOLOGY

A parameter investigation was conducted to compare the assessed $N_{\mathrm{Rd.STR}}$ (minimum value of the critical buckling load, N_{cr}, and the cross-sectional resistance, N_{Rd}) between the two analytical models [3], [6] and the numerical model. The axially loaded pile that was studied was a 4 m long circular steel pipe pile characterized by a diameter, d, ranging from 110 mm to 220 mm. The pile was presumed to be embedded within a stratum of uniform clay exhibiting a variable c_{u} between 4 to 40 kPa. Initial deflection of the pile, denoted as δ_0, was assigned according to the guidelines outlined by the Commission of Pile Research in Sweden [9] accounting for initial imperfections, residual stresses and pile driving. In both the model according to reference [3] and the full numerical model, δ_0 was implemented in accordance with its specifications as outlined in Table 1, alongside the additional material parameters employed in reference [7]. Since δ_0 is dependent on $l_{\mathrm{cr.s}}$, it varied when different clays and diameters were considered. To consider the effect of δ_0 in the model from [6], buckling curves are used which reduces the design resistance of the pile. However, a notable limitation is that the magnitude of δ_0 cannot be controlled as it is already incorporated in the buckling curves, emanating from experimental studies of steel elements in [8]. In this study, buckling curve d was used for the calculations.

Table 1. Input parameters

Parameter	**Range of Values**
Thickness, t	10 mm
Length, l	4 m
Undrained shear strength, c_u	4, 6, 8, 10, 12, 15, 20, 30, 40 kPa
Diameter, d	110, 140, 170, 220 mm
Initial deflection, δ_0	$2 * \frac{l_{cr.s}}{600} + 0.013 * l_{cr.s}$ [mm]
Modulus of elasticity, E	189* GPa
Poisson's ratio, ν	0.3 [-]
Yield stress, f_y	460 MPa
Ultimate strength, f_u	540 MPa

* 210GPa x 0.9 = 189 GPa [9]

The finite element software ABAQUS 6.21-1 [10] was used for modelling the pile and Winkler springs were used to represent the clay in the full numerical model. These springs were defined as ideal-elastic-plastic to account for the nonlinear behavior of the soil, which is the same procedure as described in [3]. Eigenvalue buckling analysis combined with Riks method were used in the finite element analysis since this combination of methods considers the plasticity and second order effects in the pile. In order to make sure that the model was representative, a simulation without springs was executed and compared to a classical Euler case, in which the pile is defined as a beam with pinned supports. The difference in terms of N_{cr} was 1.3%, hence the model gave a good representation of the free beam considering the elastic stability of the structure. A convergence analysis was also performed to decide the distance between the Winkler springs as well as the size and type of mesh.

As the prerequisites in terms of δ_0 varies between the Eurocode 3 model [6] and the other two studied models, an alternative model was developed in [7]. This model, referred to as the EC3 equivalent model in [7], was a variation of the model in [6] where the reduction from the buckling curves were manipulated in order to replicate initial deflections according to the Commission of Pile Research in Sweden [9]. In this way a more realistic initial deflection of the pile was considered rather than considering an initial deflection used for a steel element.

3. RESULTS AND DISCUSSION

Figure 2 and Figure 3 illustrate the variation of the design resistance, denoted as $N_{Rd.STR}$, in relation to $l_{cr.s}$ for the three computational methodologies investigated. The outcomes for the four slender pile diameters are illustrated in the graphical representations, wherein solely c_u is subjected to variation, thereby influencing $l_{cr.s}$. The x-axis is normalized to facilitate the identification of scenarios where the theory, as outlined in reference [3], is considered appropriate for implementation.

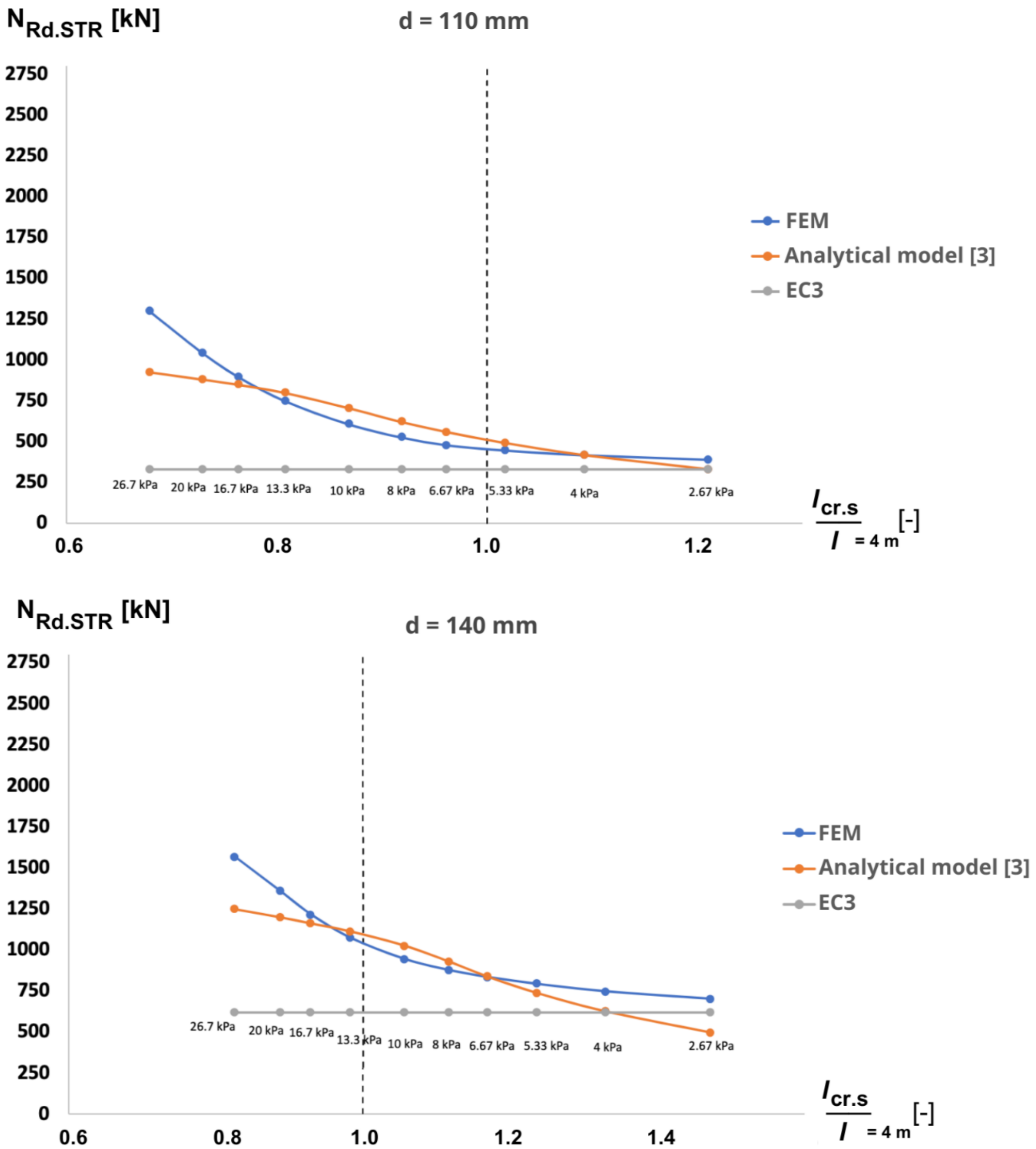

Figure 2. Evaluated design resistance compared to the ratio between the elastic buckling length, $l_{cr.s}$, and the physical length of the pile, l, for test cases where d = 110/140 mm.

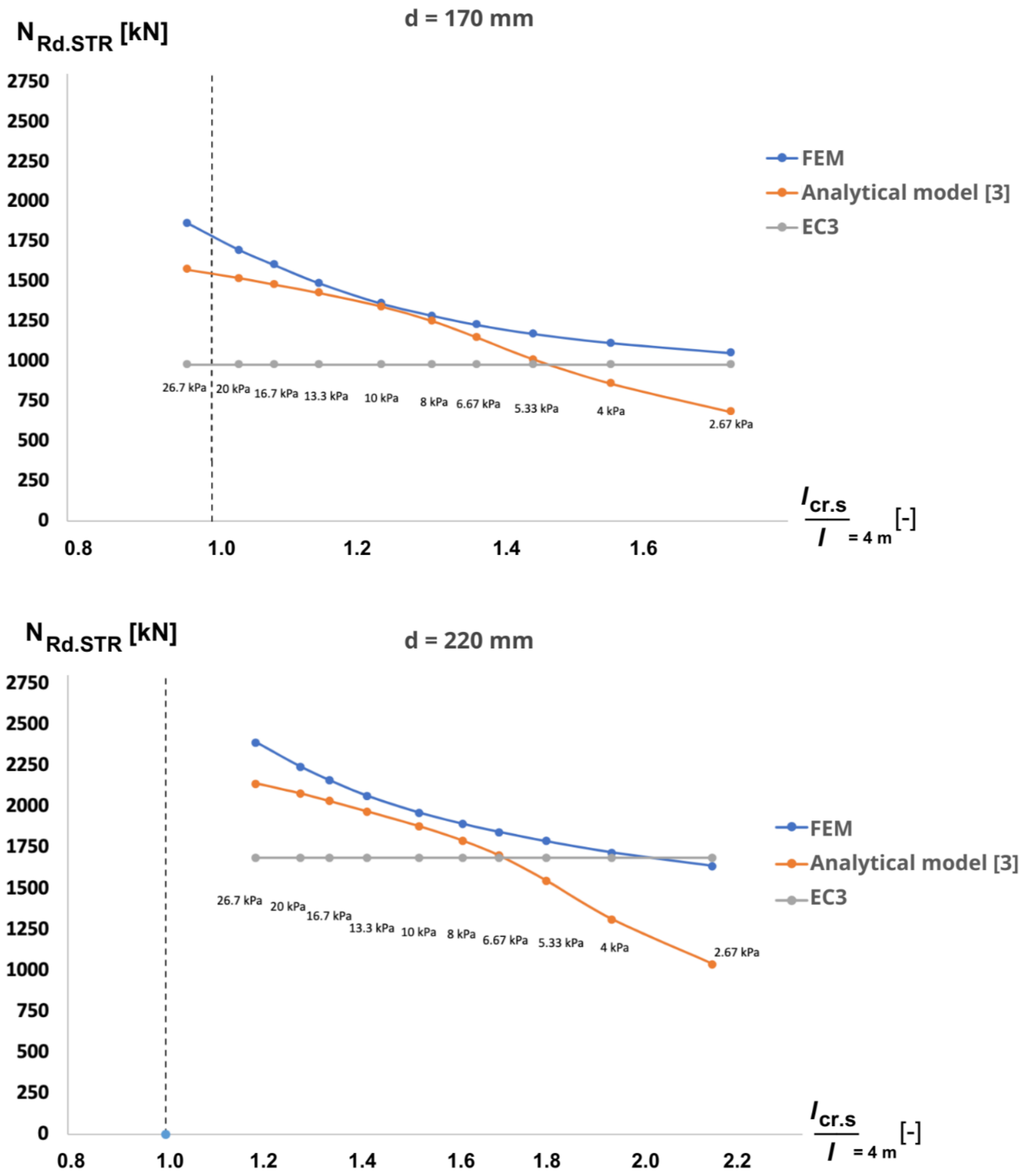

Figure 3. Evaluated design resistance compared to the ratio between the elastic buckling length, $l_{cr.s}$, and the physical length of the pile, l, for test cases where d = 170/220 mm.

Overall, the design resistance, $N_{Rd.STR}$, of the different models illustrated in Figure 2 and Figure 3 exhibit expected results. The numerical model, which mirrors the prerequisites of the analytical model in [3], yields $N_{Rd.STR}$ similar to those of the analytical model. The Eurocode 3 model for steel piles yields lower $N_{Rd.STR}$ compared to others, as it treats the pile as a freestanding column without the confining pressure exerted by the surrounding soil. Based on the presented diagrams, it is evident that with a pile length of 4 m, $l_{cr.s}$ frequently exceeds l given the input parameters. Consequently, the pile design is assumed to be based on an alternative method. The adoption of a comprehensive numerical model in the practical design of a pile is often economically disadvantageous, while analytical alternatives remain as available choices for the designer.

Given an appropriate representation of the pile behavior in the numerical model presented in [7], it becomes evident that $N_{Rd.STR}$ as per reference [3] yields non-conservative outcomes in certain examined cases. This is observed within intervals where the analytical model [3] is considered both valid and invalid for application. It is notable that this phenomenon seems to be limited to slender piles (d <140 mm) in clay soils with c_u below approximately 15 kPa. However, examining the bearing capacity of the numerical model in isolation ($\frac{l_{cr.s}}{l} > 1$) may not yield accurate design resistances. This is because, as discussed in [9], it is stated that*: "If the determination of the actual initial curvature is conducted on a section that deviates from the theoretical buckling length, a conversion to the buckling length must be performed."* A method for conducting this conversion is not provided, and selecting the actual pile length, l, may not be appropriate as $l_{cr.s}$ is determined based on both c_u and the cross-sectional properties. However, the numerical approach can be utilized for comparison with the analytical model outlined in [3] since the preconditions are identical.

By studying Figure 2 and Figure 3 it is evident that the calculation model incorporating a steel pile according to Eurocode 3 in some cases results in design resistances exceeding those obtained from the analytical and numerical models that consider a confining pressure along the pile. This discrepancy prompts further investigation of the models and their preconditions. Given the non-fulfillment of the prerequisite conditions for the analytical method as outlined in reference [3], this may serve as a plausible explanation for the observed discrepancy. In Figure 2 and Figure 3 it is observed from ratios of 1.2 and above. By studying scenarios involving larger pile diameters, the model in accordance with Eurocode 3 also appears to yield design resistances that exceed those generated by the numerical model considering a confining pressure. Through an examination of the foundational assumptions inherent in both methodologies, it becomes apparent that they address imperfections in different ways. Both the analytical model in [3] and the numerical model treat pile imperfections similarly, expressing the initial deflection of the element as a function of the elastic buckling length, $l_{cr.s}$. Furthermore, residual stresses are addressed within these models by adding an increase of the initial deflection (which is also dependent on $l_{cr.s}$) and by reducing the Young's Modulus, E. However, in the model according to Eurocode 3 these aspects cannot be addressed in the same manner as imperfections are integrated into the buckling curves, which are derived from empirical experiments in in [8]. In [8], it is evident that the δ_0 taken into account during the formulation of the buckling curves outlined in Eurocode 3 is the element length divided by 1000 ($\frac{l}{1000}$), equating to 4 mm in the present context. Conversely, when utilizing the analytical model described in [3] accounting for initial imperfections, residual stresses and pile driving, this value ranges from 13 to 39 mm. Nevertheless, according to the chapter of piles (EN 1993-5), the recommendation is to employ buckling curve d to accommodate the effect of pile-driving, thereby addressing imperfections up to $\frac{l}{200}$. This leads to a decrease in $N_{Rd.STR}$ due to the significant reduction in the chi-factor χ. However, it appears that this approach does not adequately consider piles situated in conditions of very soft clay characterized by c_u falling below 10 kPa.

In reference [7], Stener and Ebenhardt introduce an alternative semi-analytical approach designed to incorporate initial deflection, residual stresses and effects of pile driving into buckling curves. The method uses the numerical model, which has proven to accurately replicate the buckling curves of Eurocode, thereby enabling the formulation of new buckling curves that account for all imperfections specified in reference [9]. Subsequently, in Figure 4 and Figure 5, the design resistances obtained through the semi-analytical method are integrated into the previously presented graphs in Figure 2 and Figure 3.

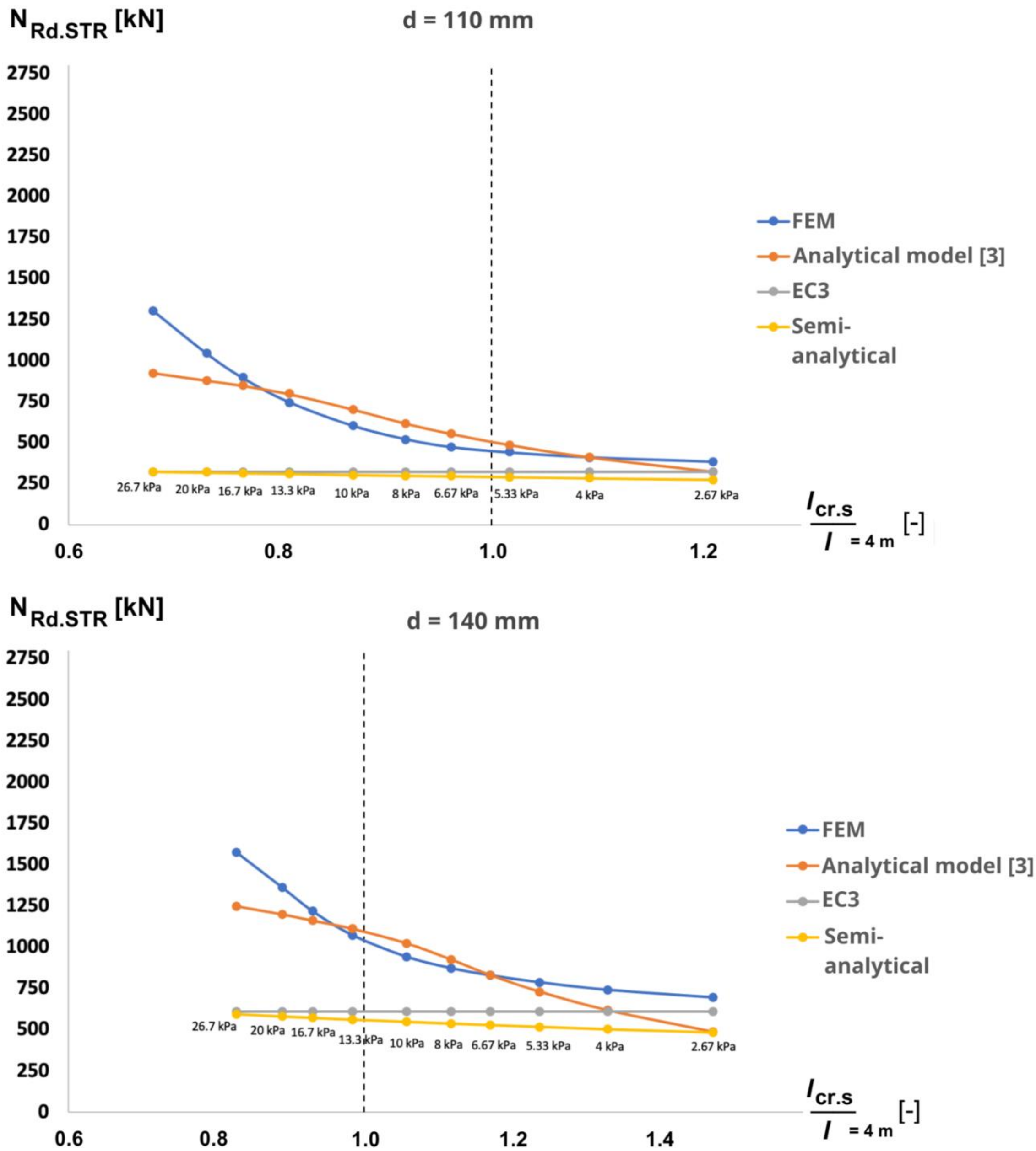

Figure 4. Evaluated design resistance compared to the ratio between the elastic buckling length, $l_{cr.s}$, and the physical length of the pile, l, for test cases where d = 110/140 mm. Semi-analytical represents the model developed in [7].

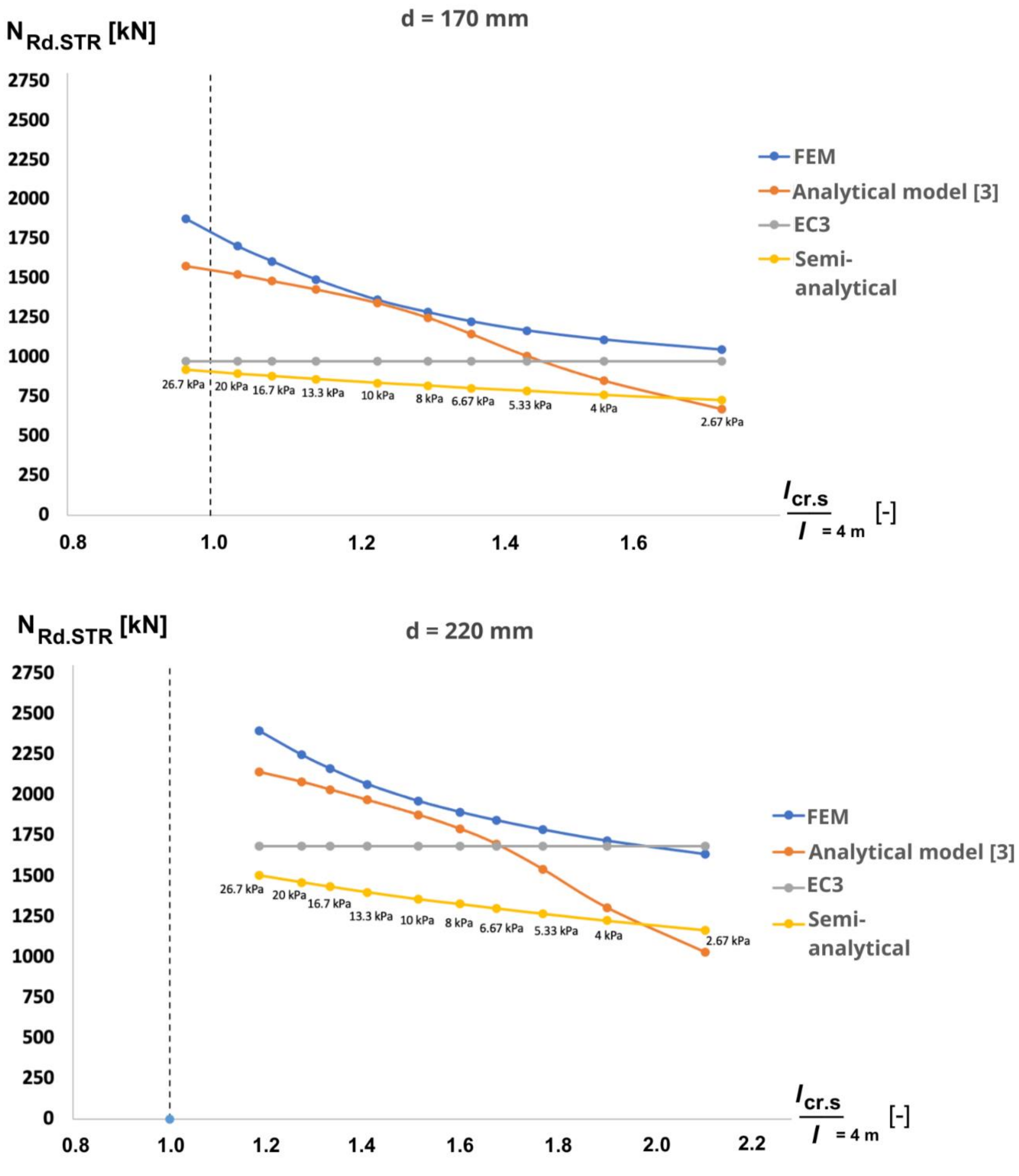

Figure 5. Evaluated design resistance compared to the ratio between the elastic buckling length, $l_{cr.s}$, and the physical length of the pile, l, for test cases where d = 170/220 mm. Semi-analytical represents the model developed in [7].

As expected, $N_{Rd.STR}$ decline in comparison to those obtained by the Eurocode 3 model when employing the semi-analytical approach, as demonstrated in Figure 4 and Figure 5. The illustrated curves consistently demonstrate a decrease in $N_{Rd.STR}$ with the increase of pile imperfections, δ_0. In other words, based on Figure 4 and Figure 5 one can notice that the use of realistic pile imperfections is essential when determining the design resistance of a pile. Through the examination of cases in which the ratio $\frac{l_{cr.s}}{l}$ closely approaches 1, a reasonable assessment of the initial deflection of a 4 m pile is undertaken. Notably, the length over which the pile is presumed to buckle coincides with the actual length of the pile. In Figure 6 and Figure 7, two cases where this occurs are presented, with initial deflections of 19 mm in both cases.

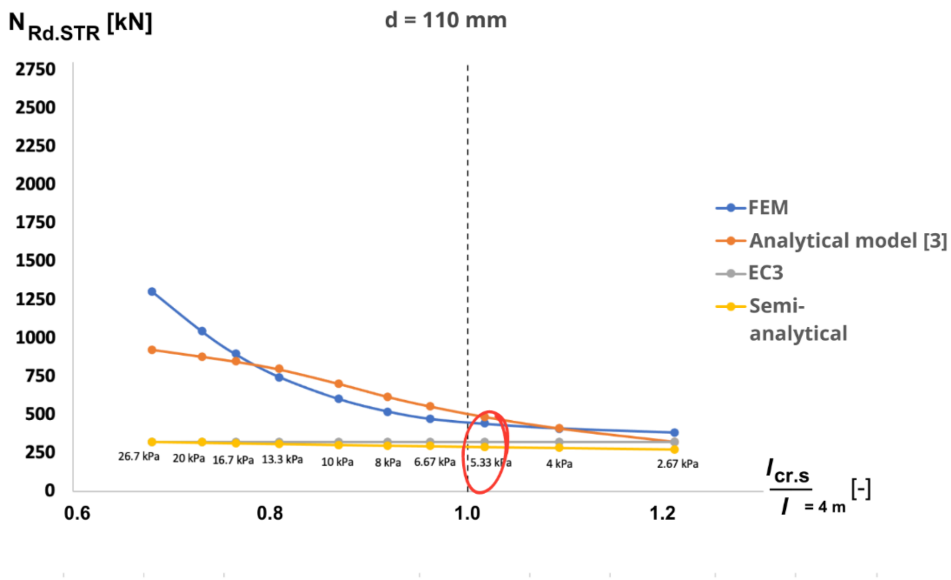

Figure 6. Evaluated design resistance compared to the ratio between the elastic buckling length, $l_{cr.s}$, and the physical length of the pile, l, for the test case considering a diameter of 110 mm. Highlighted case where the initial deflection is equal to 19 mm.

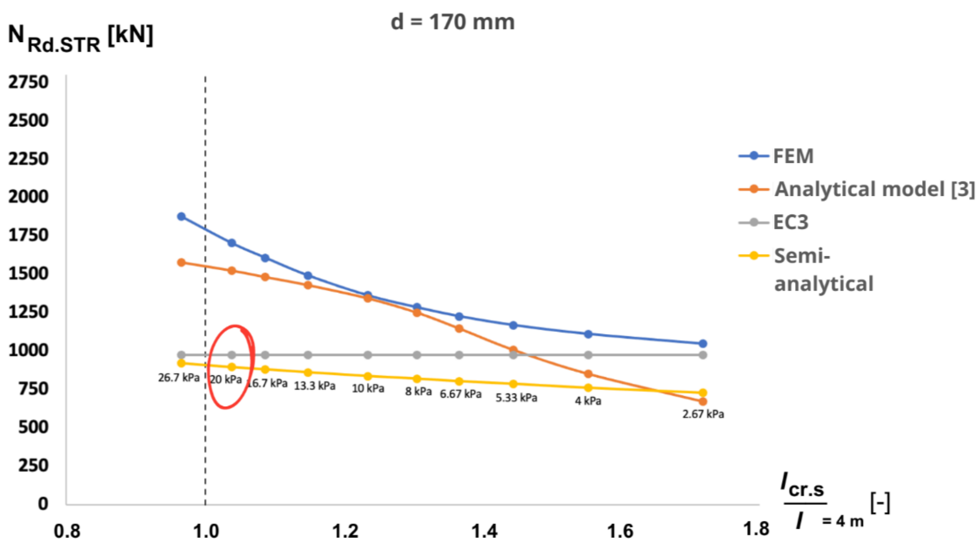

Figure 7. Evaluated design resistance compared to the ratio between the elastic buckling length, $l_{cr.s}$, and the physical length of the pile, l, for the test case considering a diameter of 170 mm. Highlighted case where the initial deflection is equal to 19 mm.

Based on the information presented above, it is evident that the Eurocode 3 model may not accurately account for all pile imperfections with sufficient precision solely by selecting buckling curve d, as stipulated in the code. In such instances, the design resistance, $N_{\mathrm{Rd.STR}}$, can be overestimated by the designer of the pile.

4. CONCLUSIONS

The following conclusions can be stated from this study.

- The analytical model presented in [3] tends to be conservative in the majority of the investigated cases, even when the pile's length, l, is shorter than the elastic buckling threshold, $l_{\mathrm{cr.s}}$.
- Modelling a pile as a freestanding column according to Eurocode 3 may underestimate the effect of imperfections, potentially leading to a non-conservative design of the pile.
- Utilizing a numerical model for pile design when the pile's length, l, is shorter than the elastic buckling length, $l_{\mathrm{cr.s}}$, is deemed impractical from both a time requirement and economic perspective.
- In practice it is often observed that the same pile diameters are utilized for both shorter and longer piles, resulting in a naturally lower utilization ratio for the former.

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