Weight reduction of concrete poles for the Swedish power line grid

*Using a Finite Element Model to optimize geometry in relation to load requirements*

**Author:** Jenny Bülow Angeling

**Supervisor (LNU):** Michael Dorn

**Supervisor company:** Rikard Bolmsvik (Abetong)

**Examinar:** Björn Johannesson

**Course Code:** 4BY35E

**Semester:** Spring 2017, 15 credits

**Linnaeus University, Faculty of Technology**

**Department of Building Technology**
Abstract

Because of an eventual ban of creosote-impregnated products, alternative materials for poles used in the electrical grid are needed. Concrete is one alternative and spun concrete poles have been manufactured for the Swedish grid before. These poles are still in use since the high strength and good functioning. However, they weigh too much in terms of the way that poles are assembled on the grid today. Therefore, a study comparing the capacity of different geometries, resulting in lower weight, is of interest.

In this Master’s Thesis, crack initiation and compressive failure in concrete poles are examined by creating FE-models in the software BRIGADE/Plus, using concrete damage plasticity. Thus, guidance is provided about how thin the concrete walls can be made without risking failure – which also means how low the weight of such a pole can be.

The failure most likely to occur is a compressive failure in the concrete with a ductile behavior. The result shows that a geometry change, which implies a thinner concrete wall, is possible. This means a weight reduction between 30-75 % or even more, depending on which network the poles are designed for.

Keywords: FE-model, Finite Element model, Concrete damage plasticity, Fracture mechanics, Fracture energy, Pole
Acknowledgement

This Master’s Thesis is the final step when finishing the Master Program in Structural Engineering at the Department of Building Technology at Linnaeus University in Växjö. The report and analysis has been conducted at Abetong AB in Växjö between Mars and May 2017.

I would like to thank my supervisors, senior lecturer Michael Dorn at the Department of Building Technology at Linnaeus University, and Rikard Bolmsvik, at Abetong AB in Växjö, for guidance during my work.

Jenny Bülow Angeling

Växjö 23rd of May 2017
# Table of contents

SYMBOL DESCRIPTION........................................................................................................... IX

1. INTRODUCTION.................................................................................................................. 1
   1.1 Background and problem description............................................................................. 2
   1.2 Aim and purpose............................................................................................................. 4
   1.3 Hypothesis and limitations............................................................................................ 4
      1.3.1 Hypothesis............................................................................................................. 4
      1.3.2 Limitations........................................................................................................... 4
   1.4 Reliability, validity and objectivity................................................................................. 5

2. LITERATURE REVIEW........................................................................................................ 7
   2.1 Concrete poles and columns reinforced by high-strength steel..................................... 7
      2.1.1 Ductile or brittle failure of reinforced concrete poles........................................... 7
   2.2 Finite element modeling of poles.................................................................................. 8
      2.2.1 Finite element analysis of reinforced concrete poles.......................................... 8
      2.2.2 Finite element analysis of other types of poles.................................................... 9
   2.3 Fracture energy and how to handle it in FE-modelling software.................................... 9
      2.3.1 Measure fracture energy by testing...................................................................... 9
      2.3.2 Fracture energy in ATENA and ABAQUS......................................................... 10

3. THEORY................................................................................................................................ 13
   3.1 Properties and behavior of concrete............................................................................. 13
      3.1.1 Strength classes..................................................................................................... 13
      3.1.2 Non-linear behavior for compressed concrete according to Eurocode............... 14
      3.1.3 Behavior of concrete in tension........................................................................... 15
      3.1.4 Reinforced concrete............................................................................................. 16
   3.2 Fracture mechanics of concrete.................................................................................... 18
      3.2.1 Fracture energy.................................................................................................... 18
      3.2.2 Stress-strain relations for short term compression............................................. 20
      3.2.3 Stress-strain and stress-crack relations for short term tension......................... 22
   3.3 Properties and behavior of steel................................................................................... 24
      3.3.1 Prestressed steel................................................................................................. 24
   3.4 Designing poles and columns....................................................................................... 25
      3.4.1 Mode of actions of slender poles and columns.................................................... 25
      3.4.2 Elementary cases for beams with a fixed attachment........................................... 26
      3.4.3 Prevented transverse expansion......................................................................... 26
      3.4.4 Design loads for poles to the power line grid..................................................... 27
      3.4.5 Bending test on poles........................................................................................ 28
   3.5 The Finite Element Method........................................................................................... 28
      3.5.1 The modelling software BRIGADE/Plus.......................................................... 29
      3.5.2 Fracture mechanics in BRIGADE/Plus.............................................................. 29

4. METHODS AND IMPLEMENTATION................................................................................. 33
   4.1 Field test...................................................................................................................... 33
      4.1.1 Test model......................................................................................................... 33
      4.1.2 Test set up.......................................................................................................... 33
   4.2 Analysis in BRIGADE/Plus......................................................................................... 35
      4.2.1 Geometrical modeling in the part-module......................................................... 35
      4.2.2 Material model for concrete.............................................................................. 36
      4.2.3 Material model for steel..................................................................................... 38
      4.2.4 Element types and sizes.................................................................................... 39
      4.2.5 Load steps......................................................................................................... 39
      4.2.6 Loads and boundary conditions......................................................................... 40
Symbol description

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A$</td>
<td>Area</td>
</tr>
<tr>
<td>$A_c$</td>
<td>Cross section area concrete</td>
</tr>
<tr>
<td>$A_s$</td>
<td>Cross section area of steel bar</td>
</tr>
<tr>
<td>$A_{s, tot}$</td>
<td>Total cross section area of steel bars</td>
</tr>
<tr>
<td>$E$</td>
<td>Modulus of elasticity</td>
</tr>
<tr>
<td>$E_{cm}$</td>
<td>Mean value for the modulus of elasticity</td>
</tr>
<tr>
<td>$E_d$</td>
<td>Design load</td>
</tr>
<tr>
<td>$E_p$</td>
<td>Design value for modulus of elasticity for steel</td>
</tr>
<tr>
<td>$F_{ini}$</td>
<td>Initial force in tendons</td>
</tr>
<tr>
<td>$F_n$</td>
<td>Load applied at bending test</td>
</tr>
<tr>
<td>$F_u$</td>
<td>Maximum load, failure load</td>
</tr>
<tr>
<td>$F_x$</td>
<td>Initial stress in tendons, prior to anchoring at distance $x$ from cable end</td>
</tr>
<tr>
<td>$G_f$</td>
<td>Fracture energy, absorbed per unit crack area</td>
</tr>
<tr>
<td>$G_{f0}$</td>
<td>Base value of fracture energy</td>
</tr>
<tr>
<td>$G_K$</td>
<td>Characteristic value for permanent loads</td>
</tr>
<tr>
<td>$H_i$</td>
<td>Vertical load caused by imperfections</td>
</tr>
<tr>
<td>$I$</td>
<td>Increment size</td>
</tr>
<tr>
<td>$I_l$</td>
<td>Moment of inertia</td>
</tr>
<tr>
<td>$I_{max}$</td>
<td>Maximum increment size</td>
</tr>
<tr>
<td>$I_{min}$</td>
<td>Minimum increment size</td>
</tr>
<tr>
<td>$K$</td>
<td>Yield surface in the deviatory plane</td>
</tr>
<tr>
<td>$L$</td>
<td>Length</td>
</tr>
<tr>
<td>$L^d$</td>
<td>Length of the damage zone</td>
</tr>
<tr>
<td>$N$</td>
<td>Normal force, axial force</td>
</tr>
<tr>
<td>$N_{cr}$</td>
<td>Crack load</td>
</tr>
<tr>
<td>$N_u$</td>
<td>Ultimate load</td>
</tr>
<tr>
<td>$M$</td>
<td>Bending moment</td>
</tr>
<tr>
<td>$P$</td>
<td>Point load</td>
</tr>
<tr>
<td>$P_{tot}$</td>
<td>Total pressure</td>
</tr>
<tr>
<td>$T_s$</td>
<td>Time period for one step</td>
</tr>
</tbody>
</table>
\( d_i \) Diameter
\( e_i \) Unintentional eccentricity
\( f_{cd} \) Design value for the compressive strength for concrete
\( f_{ck} \) Characteristic compressive strength for concrete cylinder
\( f_{ck,cube} \) Characteristic compressive strength for concrete cube
\( f_{cm} \) Mean value for the cylindrical compressive strength
\( f_{ctd} \) Design value for the concrete tensile strength
\( f_{ctk0.05} \) Tensile strength, lower characteristic value
\( f_{ctk0.95} \) Tensile strength, upper characteristic value
\( f_{tm} \) Mean value for the cylindrical tensile strength
\( f_{ctm,fi} \) Mean value for tensile strength, increased by \( k \)
\( f_n \) Displacement, corresponding to the load \( F_n \)
\( f_{pk} \) Characteristic tensile strength for prestressing steel
\( f_{p0.1k} \) Characteristic strength at the strain 0.1%
\( f_{yk} \) Yield stress for steel
\( h \) Height
\( h_a \) Distance from the top of a pole to the applied load
\( k \) Coefficient
\( l \) Length
\( l_0 \) Effective length or buckling length
\( r \) Radius
\( t \) Thickness
\( w \) Deformation, crack width
\( \alpha_{cc} \) Coefficient for long term effects on compression strength
\( \alpha_{ct} \) Coefficient for long term effects on the tensile strength
\( \gamma_C \) Partial factor for concrete
\( \gamma_G \) Partial factor for permanent actions
\( \gamma_S \) Partial factor for the prestressed steel
\( \gamma_Q \) Partial factor for variable actions
\( \varepsilon_c \) Compressive strain
\( \varepsilon_{c,\text{in}} \) Compressive inelastic strain
\( \varepsilon_{c,p} \) Plastic strain
\( \varepsilon_{c,\text{lim}} \) Strain at \( \sigma_{c,\text{lim}} = 0.5f_{cm} \).
\( \varepsilon_{c1} \) Strain at maximum stress \( f_{cm} \).
\( \varepsilon_{\text{cu1}} \) Strain at ultimate limit state.
\( \varepsilon_d \) Average additional strain.
\( \varepsilon_m \) Average strain.
\( \varepsilon_s \) Strain in steel.
\( \mu \) Friction coefficient.
\( \nu \) Poisson’s ratio.
\( \rho \) Density.
\( \varepsilon_{uk} \) Characteristic strain for prestressing steel.
\( \sigma_c \) Stress in concrete.
\( \sigma_{cn} \) Stress in concrete from moment.
\( \sigma_{cm} \) Stress in concrete from normal force.
\( \varnothing_{\text{top}} \) Top diameter.
\( \varnothing_{\text{bottom}} \) Bottom diameter.
\( \psi \) Dilation angle.
1. Introduction

There are over 10 million km of power lines in Europe [1]. The electrical network grid consists of overhead lines and underground cables. When choosing material for the poles, used to carry overhead lines, there are several factors taken into account, including the age-dependency for the materials, the life cycle cost (LCC) and the environmental impact [2].

There are several different materials used for poles, carrying overhead lines. Steel, wood and concrete are three common materials used in Europe. According to Tanifener [3] concrete is to prefer of those three materials because of its durability and low maintenance require and also because of environmental and economic reasons. According to Bolin and Smith [4] wood poles impregnated with pentachlorophenol (penta), which is an organochlorine, is a better alternative than concrete and steel in case of fossil fuel, acid rain and carbon dioxide (CO₂) emissions, but that penta-treated poles result in more smog than both concrete and steel poles. When comparing different materials for poles in a report from the Swedish Environmental Research Institute [5], concrete is a better alternative than steel when comparing both energy consumption and environmental impact, such as climate change, acidification and eutrophication. However, steel poles are often very high and are able to carry overhead lines with a great span between the poles, which make them a good alternative to the parts of the electrical network with extra high voltage.

When comparing concrete with wood impregnated with creosote, which is a carbonaceous chemical, CO₂-emissions are larger for concrete than the creosote-treated wood [6]. However, in this study the life span for the product made of concrete is 50 years but for concrete poles it is often determined to minimum 80 years, which will affect the results in a LCA. In another study, comparing alternative pole materials for the Swedish market, and the lifespan is equal to 80 years, the concrete pole is considered to be a very good alternative [7]. Over its total lifetime the concrete pole still has a larger carbon footprint than the wooden pole, but the fact that concrete is a material that could be recycled and also that it is not containing hazardous materials is making it a recommended alternative. Steel, however, is the least recommended material according to all mentioned studies when taking economy, environment and also the weight that affects mounting into account.

To sum up, the main problem with both penta and creosote is not the emissions, but the containment of hazardous materials, which has negative effects of human exposure and at the sites where these products are used or stored. These effects are considered sufficiently problematic for other options to be interesting, even if it means a greater contribution to emissions. Efforts to reduce material consumption in these alternative products are therefore of great importance.
1.1 Background and problem description

In Sweden, the electrical grid consists of almost 900 000 km lines, of which approximately 300 000 km are overhead lines [8]. The grid is separated into three different network levels, depending on voltage; the local network, mostly consisting of lines with 0.2 and 0.4 kV, called low and medium voltage; regional network, with voltage between 40-130 kV, called high voltage; and the national grid, mostly with voltage 220 and 400 kV, called extra high voltage. Today, the Swedish local and regional power line grid consists largely of creosote-impregnated power poles.

As mentioned before, creosote is a carbonaceous chemical, used to impregnate wood. This is made to protect the wood against infestation of wood decaying fungi. Creosote is classified as carcinogenic [9]. Several substances in creosote have hazardous properties and some of those are persistent, poisonous and can accumulate in living tissue. In areas where creosote-treated poles are used, polycyclic aromatic hydrocarbons (PAHs) occur in the groundwater [10].

The European Union is undergoing a review that could lead to a ban of creosote from year 2018 [9]. Therefore, there is a need for alternative material for poles on the Swedish market. Suppliers on the market are actively working to find alternative pole materials [11]. Materials considered competitive are steel, concrete, wood veneer and glass-fiber composite [7] [11]. Pole materials that are used today, except for creosote impregnated wood, are spruce veneer, glass fiber reinforced polyester (called composite pole) and steel. Veneer and composite poles are mainly used in the local electricity network and steel poles in the national grid. Poles made of concrete have been manufactured in the past and is still considered to be an interesting option if it would go into production [12]. Moreover, in comparison with, for example, the veneer pole, it can be an alternative to the creosote impregnated pole both in the local and regional electricity network. Right now, in the beginning of 2017, there are no manufacturing of concrete poles in Sweden.

Abetong manufactured poles made of spun concrete for the Swedish grid in the 1990’s [12]. These poles, shaped as a coned hollow cylinder with prestressed reinforcement, are still in use due the high strength and good functioning. However, they weigh too much in terms of the way that poles are assembled on the grid today. According to Vattenfall and E.ON [12], electricity distribution companies in Sweden, a weight reduction would make the concrete pole a competitive alternative on the Swedish market. Furthermore, a weight reduction resulting in lower production costs, leading to a more economic competitive alternative, and, not least, less emissions of CO₂.

Poles to the Swedish power line grid should be designed for different load cases, taking into account permanent loads, wind load, ice load, construction
and maintenance loads as well as safety loads, which requires the capacity to withstand torque due to e.g. the uneven load due to cable breakage (according to the Swedish Standards for electrical powerlines, [13] [14] [15]). When testing poles according to the European Standard for masts and poles [16], the load in relation to deflection under bending, the maximum load capacity and the torque capacity should be measured. Following the classification of wood poles, often used as basis in Sweden, the poles must obtain a given load capacity for a load applied as a vertical point load 0.2 m from the top [17]. For poles in the local network this capacity varies between 3.5-7.8 kN, where the most common pole, called model G (as in the Swedish word grov, meaning coarse), must have the capacity of 4.5 kN. The Research Institute of Sweden (RISE) has conducted buckling, bending and torsional tests on wood poles of the model G and, except for the requirements mentioned above, the result gives a picture of the capacity needed when designing poles of different materials [18]. Capacity requirements increase for poles used in the regional network and the size of it depends on several things, such as where the poles will be placed geographically, what distance there should be between them and which type of line they shall bear.

In the beginning of this study, an old concrete pole was drawn to failure out in the field, where it had stand for about 20 years. The outcome of the test gave a perception about the behavior of the pole. The test included loading to a certain load level and thereafter unloading before loading it again until failure. The result of the field test showed that this type of concrete pole has a higher strength than required, which means that it has an excessive size. Because of the need of new alternative on the market in combination with the requirement to limit the weight, it is of great interest to examine the strength of poles of this type in more ways than by hand calculations. This provides a picture of realistic dimensions of concrete poles for the Swedish power line grid.

Finite Element Modelling is a numerical simulation tool, useful for understanding the behavior of a model at different loads. By creating a Finite Element Model (FE-model) that captures the behavior of the concrete pole that was drawn to failure in the field, it will be possible to examine the expected behavior of models with different geometries. By stressing the materials to the limit, a good picture of the possible size reduction will be obtained, while still fulfilling the requirements for poles. The results of the simulations will also be compared to the capacity of wood poles, used in the local network today.

The purpose with this Master’s Thesis is to examine the compressive or tensile failure and crack initiation by creating a FE-model in the software BRIGADE/Plus. Thus, guidance about how thin wall of concrete, which also means how low weight, that can be achieved without risking failure will be provided. As mentioned above, this is important when designing poles to the
Swedish electricity grid, to be able to assemble them with the same methods used for wood poles today.

1.2 Aim and purpose

The aim with this Master’s Thesis is to

- establish a FE-model that is able to capture the behavior of a tested pole in full scale,

- optimize the FE-model, with the behavior as above, by using different geometries reducing overall weight, but still fulfilling load requirements, and

- deliver an investigation on the failure most likely to occur when the pole is loaded to the ultimate limit (crack initiation in serviceability state will also be included in the model).

The purpose is to examine the possibility of making a concrete pole to the Swedish power line grid, with as low weight as possible, to facilitate transport and assembling on the electricity grid, but still meeting the load requirement.

1.3 Hypothesis and limitations

1.3.1 Hypothesis

By changing the geometry of a concrete pole the weight will be reduced.

A pole with a reduced geometry and lower weight, compared with concrete poles manufactured at Abetong before, is still going to fulfill load requirements for the pole.

By creating a FE-model and examining compression and/or tension failure of a concrete section, it will be possible to obtain a good prediction about which type of failure that will occur for a pole when loading it to the ultimate limit.

1.3.2 Limitations

In this Thesis concrete with the strength 105 MPa, for a 100 mm cube, will be examined.
In the model, created in BRIGADE/Plus, the reinforcement will be embedded. This means that it will be full interaction between the reinforcement and the concrete.

The amount of prestressed reinforcement will not be varied but is limited to 40 prestressed lines, which is the number used in the pole tested in the field.

1.4 Reliability, validity and objectivity

By creating a FE-model in the software BRIGADE/Plus, to examine the behavior of the pole and the concrete, the reliability of this work will increase, because, if the same parameters are used as in this project, the outcome will be the same for repeated attempts.

The concrete pole was drawn to failure out in the field, not in a laboratory, where the test set-up and measurement of the load could be carried out with full control. Because of that, this test provides an approximate value for the strength of the pole and its behavior. Therefore, the result would very likely vary if the test was repeated.
2. Literature Review

This chapter provides a review over previous reports and articles written on topics and studies of interest for this thesis. Articles examining reinforced concrete poles in programs similar to the software BRIGADE/Plus is of particular interest, but other studies examining the behavior of reinforced and prestressed concrete will be mentioned. This is made to define which type of failures of the poles that can be expected. How to take fracture energy into account will also be addressed.

2.1 Concrete poles and columns reinforced by high-strength steel

In this section, a literature review is done about reinforced columns and poles made of spun or regular (vibrated) concrete.

2.1.1 Ductile or brittle failure of reinforced concrete poles

According to Kudzys and Kliukas [19], investigations show that failure of compressed spun concrete members, reinforced with high-strength steel, is most likely to be ductile. In their study, hollow cubical specimens of spun concrete, with outer diameters of 500 and 260 mm were used. The specimens were reinforced with ribbed high-strength steel bars, uniformly distributed over the cross section. Only when the geometrical reinforcement ratio, which is the relation between the total cross section area of the reinforcement, $A_{s,tot}$, and the cross section area of the concrete, $A_c$, calculated by

$$\rho = \frac{A_{s,tot}}{A_c}$$

was less than 3% there was a risk for a relative brittle failure. According to this study, a recommended value for the reinforcement ratio in spun concrete members is 3-6%, which means that the total area of the reinforcement should be between $0.03 \cdot A_c$ and $0.06 \cdot A_c$. This can be compared to the minimum cross section area of the reinforcement according to the Eurocode for design of concrete structures [20], which is

$$A_{s,\text{min}} = 0.002 \cdot A_c.$$  \hspace{1cm} (2)

In the Eurocode there is no limiting maximum value for the cross section area of the reinforcement. In the study made by Kudzys and Kliukas only longitudinal reinforcement was used.

In a study by Kuebler and Polak [21], helical reinforcement was used when examining torsion failure. Helical reinforcement, which is a type of lateral reinforcement, counteracts post cracking before failure occurs. When this
was used, the torsion failure of the concrete poles was brittle. The advantage with helical reinforcement is the prevention of cracks in the concrete during the release of prestressed reinforcement.

According to the European Standard for overhead electrical lines, EN 50341-1-1, [14], lateral reinforcement, which consists of helical spirals or lateral ties, is used to control longitudinal cracking from concrete shrinkage, transversal forces, wedging effects due to prestressed reinforcement or other sources. Regarding the design of concrete poles, the standard for overhead electrical lines refers to EN 12843, the Swedish standard for precast masts and poles [16], in which there are no requirements for lateral reinforcement if prestressed reinforcement is used. Nevertheless, when lateral reinforcement is not used the choice should be supported by experience or testing.

2.2 Finite Element modeling of poles

In this section, a review of literature about Finite Element modeling (FE-modeling) in programs similar to BRIGADE/Plus, will be provided. Firstly, articles about reinforced concrete poles will be treated. Thereafter, articles about other materials, but within the same area of interest when it comes to behavior investigated and which type of models that have been created, will be treated.

2.2.1 Finite Element Analysis of reinforced concrete poles

An article, written by Shalaby [22], examining the flexural behavior of spun concrete poles by using finite element (FE) analysis in the software ANSYS. Two poles with identical geometry but with different amount of reinforcement, made by glass fiber reinforced polymer (GFRP) bars, were used in the study and the result were compared to experimental data. As the poles in this thesis, the poles examined by Shalaby [22] are hollow of a conical shape and tapered from the top and down.

When creating the model in ANSYS boundary conditions were applied to model support conditions and to prevent out-of-plane displacements [22]. Load was applied as point loads in a line over the nodes on one outer half of the pole, at a distance of 305 mm from the top of the pole. When comparing the models made in ANSYS with experimental values, conclusions were drawn that the FE-model overestimated the compressive strain values, but failure loads of the FE-model agreed well with loads obtained from experimental data.

Another article, written by Kenna and Basu [23], describes how the effects of pre-stressed reinforcement can be taken into account when making FE-models. It is also including material and geometrical non-linearity, which is
important to take into account when doing concrete models. This study is about wind turbine towers, which are much larger constructions than the poles this thesis will cover. Still, the methods used to model the prestressed concrete towers are of interest and applicable.

Instead of solid elements, Kenna and Basu used shell elements, with six DOF on each node, and bar elements [23]. The bar elements, with three DOF per node, where mapped onto the shell elements. The non-linear behavior of concrete was modelled according to a modified Hognestad model, described in a report by Kwak and Filippou [24]. This model captures the uniaxial strain-stress behavior of concrete quite well.

2.2.2 Finite Element Analysis of other types of poles

Masmoudi et al. [25] studied deflections and bending response in GFRP poles by using a non-linear FE analysis. In this analysis, the non-linear behavior of tapered hollow poles with lengths between 6 up to around 12 meters under lateral loads were examined. The 3D FE-model was developed by using the finite element software ADINA, which is a commercial engineering simulation software program. The type of element used was an eight-node quadrilateral multilayered shell element with six DOF at every node (three displacements and three rotations). The models were divided into three zones from the top to the bottom. As in the study made by Shalaby [22], boundary conditions were applied to simulate the support conditions described in standards for poles. A load was applied 300 mm from the top of the pole and the load was varied from zero up to the ultimate load capacity for each pole. In this case, the load was not applied as point loads, but as a load distributed over half of the circumference, which is similar to the load application in this thesis.

2.3 Fracture energy and how to handle it in FE-modelling software

This chapter is treating fracture energy. An explanation of the criterion is followed by a review of literature written about how to take the fracture energy into account when doing FE-modelling and analysis.

2.3.1 Measure fracture energy by testing

Fracture energy is, according to Kazemi et al. [26], an evidenced material parameter important to include in a fracture analysis of concrete. In their study the fracture energy per unit crack area, $G_f$, is measured by doing three-point-bend (3PB) tests on cylindrical specimens made of plain concrete and steel-fiber-reinforced concrete. Conclusions were drawn, that reinforcement had a significant effect on the fracture energy and on the post peak region, where the ductility was increased.
When calculating the fracture energy, according to Kazemi et al. [26], the work of fracture, which is the total work needed to break a concrete beam, must be known. This is a method developed by Arne Hillerborg and recommended by the RILEM Technical Committee. The method is based on the theory that cracks are accompanied by energy absorption. In an article describing the influence of fracture energy, written by Markeset and Hillerborg [27], conclusions were drawn that the slope of the declining branch of the stress-strain curve will increase with increasing compressive strength, increase with decreasing fracture energy, increase with increasing length of the specimen and also increase with increasing slenderness of the specimen.

In another article, written by Fernández-Canteli et al. [28], a method called the modified compact tension (MCT) test is used, instead of 3PB. Fernández-Canteli et al. describes 3PB to be the most extended procedure to obtain the fracture energy, but because the method is both complicated and requires material and machines to do it they suggesting the MCT as an alternative. In their article, the way to do the MCT experimentally is described and also how to simulate it in ABAQUS, which is of interest for this thesis, since BRIGADE/Plus and ABAQUS are very similar programs.

2.3.2 Fracture energy in ATENA and ABAQUS

Fernández-Canteli et al. [28] describe two different codes used to perform numerical simulations of the MCT. The first one is made in the software ATENA and the second in ABAQUS. In both programs, characteristic input parameters for young modulus $E$, poisons ratio $\nu$, compressive strength $f_c$ and tensile strength $f_t$ for concrete were used. A total number of four models were created in ATENA and ABAQUS; of which one 2D model in ATENA, using ATENA code; one 2D model in ABAQUS, using ABAQUS code with rigid bars; and two different 3D models in ABAQUS, using free elastic lineal bars and ABAQUS intermediate. When studying the results, it is very clear that results of the 3D models are more consistent with the results from experiment, compared with results of the 2D models.

In ABAQUS/Standard there is a concrete damage-plasticity model including compression failure in the concrete, tension-stiffening and dilation angle, which is a term defining inelastic volumetric change in granular materials, such as concrete. Mercan et al. [29] describe how to simulate the tension-stiffening, which is the tensile behavior of concrete after cracking, in ABAQUS by using cracking criterion based on fracture energy. In their study, precast prestressed concrete was examined. The concrete was modelled by using eight-node brick elements. Maximum brick size was around 100 mm. Prestressed reinforcement was modelled by using two-node linear 3D truss element, in which initial stresses were introduced to
representing pre-stressing forces. This method is an available alternative in ABAQUS/Standard.
3. Theory

In this chapter, mathematical models and analysis methods that are relevant for this study will be described.

3.1 Properties and behavior of concrete

3.1.1 Strength classes

Concrete is a material that is, mainly, made of cement, sand, aggregate and water. An important property to take into account when dimensioning concrete is that the tensile strength is much lower than the compressive strength [30]. The compressive strength of concrete is given by different strength classes, which are related to the characteristic (5%) cylinder strength, $f_{ck}$, or the characteristic cube strength, $f_{ck,cube}$ [20]. Concrete with higher strength class than C50/60 is referred to as high-strength concrete [31]. The highest recommended value for the compressive strength is C90/105. The higher the compressive strength is, the lower is the water-cement ratio ($wcr$) [32], as can be seen in Figure 1a). This is affecting failure, which becomes more brittle when the compressive strength is getting higher [30], as shown in Figure 1b).

![Figure 1: a) Relationship between water-cement ratio and compressive strength, taken from [32]. b) Relationship between the strength class of the concrete and the type of failure. The failure is getting more brittle when the strength class is getting higher, taken from [30].]

Strength and strain properties for high strength concrete classes C55/67-C90/105, according to EN 1992, can be found in Table 1. The cylindrical compression strength is called $f_{ck}$ and the cubical compression strength $f_{ck,cube}$. Other values given in the table are the mean value for cylindrical compression strength, $f_{cm}$; the characteristic tensile strength, which is given with a lower characteristic value, $f_{ctk0.05}$, and an upper
characteristic value, \( f_{ctk0.95} \); the mean tensile strength, \( f_{tm} \); the strain, \( \varepsilon_{cu1} \), at maximum stress \( f_{cm} \); and at last the mean value for the modulus of elasticity, \( E_{cm} \).

Table 1: Strength and strain properties for concrete C90/105, according to EN 1992 [20].

<table>
<thead>
<tr>
<th>Concrete class</th>
<th>( f_{ck} ) [MPa]</th>
<th>( f_{ck, cube} ) [MPa]</th>
<th>( f_{cm} ) [MPa]</th>
<th>( f_{ctk0.05} ) [MPa]</th>
<th>( f_{ctk0.95} ) [MPa]</th>
<th>( \varepsilon_{cu1} ) [%]</th>
<th>( E_{cm} ) [GPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>55/67</td>
<td>55</td>
<td>67</td>
<td>63</td>
<td>3.0</td>
<td>4.2</td>
<td>5.5</td>
<td>3.2</td>
</tr>
<tr>
<td>C60/75</td>
<td>60</td>
<td>75</td>
<td>68</td>
<td>3.1</td>
<td>4.4</td>
<td>5.7</td>
<td>3.0</td>
</tr>
<tr>
<td>C70/85</td>
<td>70</td>
<td>85</td>
<td>78</td>
<td>3.2</td>
<td>4.6</td>
<td>6.0</td>
<td>2.8</td>
</tr>
<tr>
<td>C80/95</td>
<td>80</td>
<td>95</td>
<td>88</td>
<td>3.4</td>
<td>4.8</td>
<td>6.3</td>
<td>2.8</td>
</tr>
<tr>
<td>C90/105</td>
<td>90</td>
<td>105</td>
<td>98</td>
<td>3.5</td>
<td>5.0</td>
<td>6.6</td>
<td>2.8</td>
</tr>
</tbody>
</table>

According to EN 1992 exact values for Young’s modulus, \( E_{cm} \), can be calculated by

\[
E_{cm} = 22 \cdot \left( \frac{f_{cm}}{10} \right)^{0.3},
\]

and the mean value for the tensile strength, \( f_{ctm} \), for concrete classes above C50/60 are calculated by

\[
f_{ctm} = 2.12 \cdot \ln \left( 1 + \frac{f_{cm}}{10} \right),
\]

which are useful equations when using properties for concrete decided by testing, instead of properties from Eurocode.

3.1.2 Non-linear behavior for compressed concrete according to Eurocode

The stress-strain curve of concrete in compression is non-linear from the beginning [20]. This is shown in Figure 2, where \( f_{cm} \) is the mean value of the compressive strength. In the figure, it is described how to calculate a value for the modulus of elasticity. Because of the non-linearity, the modulus of elasticity, \( E \), is changing with the stress level, \( \sigma_c \). The higher the stress, the lower is the value of the modulus of elasticity. The compressive strain, \( \varepsilon_c \), of the concrete at maximum stress is called \( \varepsilon_{c1} \) and the strain at ultimate limit state, when failure occurs, \( \varepsilon_{cu1} \).
However, for stress less than $0.6 f_{cm}$, the stress-strain relationship is almost linear [30]. Therefore, it is appropriate to apply the theory of linear elasticity when doing calculations in serviceability limit state.

The stress-strain relationship for the non-linear analysis is described by

$$\sigma_c = \frac{k \eta - \eta^2}{1 + (k - 2) \eta}$$

(5)

where $\eta = \varepsilon_c / \varepsilon_{c1}$ [20]. The coefficient $k$ is calculated by

$$k = 1.05E_{cm} \cdot \frac{|\varepsilon_{c1}|}{f_{cm}}$$

(6)

Values of $f_{cm}$ and $\varepsilon_c$ are taken from Table 1 or calculated by the equations in Section 3.1.1.

3.1.3 Behavior of concrete in tension

The behavior of concrete in tension is assumed to be linear-elastic for the uncracked section, which is illustrated in Figure 3a [33]. For a cracked section the tension can be modelled by using the relation between the stress, $\sigma$, and the crack opening, $w$, together with the fracture energy, $G_f$, illustrated in Figure 3b.
The tensile strength of the concrete is much lower than the compressive strength. The tensile strength increases with increasing compression strength, but not proportionally. For higher concrete classes the tensile strength increases less than for lower concrete classes.

There are different test methods used to decide the tensile strength: splitting test and flexure test. The characteristic values for the tensile strength in EN 1992 are determined by splitting tests. However, it is hard to determine the tensile strength by testing and therefore it is often determined by calculations based on the compression strength [30]. According to EN 1992, the tensile strength for concrete classes higher than C50/60 can be determined by

\[ f_{ctm} = 2.12 \ln \left(1 + \frac{f_{ck} + 8}{10}\right), \]  
(7)

and from that the 5%-fractile is determined by

\[ f_{c0.05} = 0.70 \cdot f_{ctm}, \]  
(8)

and the 95%-fractile by

\[ f_{c0.95} = 1.3 \cdot f_{ctm}, \]  
(9)

### 3.1.4 Reinforced concrete

In order to overcome the low tensile strength of concrete, reinforcement is used. The reinforcement is transferring tensile forces after cracking in the tensile areas. The type of failure that occurs, if the structure reaches its ultimate limit state, depends on if the concrete or the reinforcement is reaching its ultimate limit first [30]. For both compressive failures in the concrete and tensile failure in the reinforcement, there are two kinds of
failure modes that can occur at bending, which gives a total of four failure modes for reinforced concrete, described by Engström [30] as:

- Compressive failure of the concrete with ductile behavior, which occurs when the steel is reaching its yield stress before the concrete reaches its ultimate compressive strain. In this case the pressure zone is collapsing before the reinforcement breaks.

- Compressive failure of the concrete with brittle behavior, which occurs when the steel is not reaching its yield stress before the concrete reaches its ultimate compressive strain.

- Tensile failure of the steel with ductile behavior, which occurs when the steel reaches its yield stress and tears off before the concrete reaches its ultimate compressive strain.

- Tensile failure of the steel with brittle behavior, which occurs when the steel tears off as soon as the cross section cracks. This happens when there is a too small amount of reinforcement.

There are different kinds of reinforcement; untensioned slack reinforcement, partly prestressed reinforcement and completely prestressed reinforcement. Prestressed reinforcement, often made of high strength steel, can be pre-tensioned or post-tensioned. Prestressed reinforcement is adding compressive forces into the construction, allowing for high loads before critical tensile stresses arise. Therefore, the use of prestressed reinforcement is a way to avoid cracks in the serviceability limit state.

When using prestressed reinforcement and when the relation between the cross section area of the concrete and the steel, according to Equation (1), is bigger than 3%, the failure is most likely ductile [17]. Using prestressed reinforcement instead of untensioned slack reinforcement, increases the load required for cracks considerably [30]. Furthermore, prestressed reinforcement is having a positive effect on the shear capacity, but almost no improving effect on the strength in ultimate limit state.

The load-displacement relation for plain concrete, reinforced concrete and prestressed concrete is shown in Figure 4. For a cross section with plain concrete, the crack load, $N_{cr}$, is equal to the ultimate load, $N_u$. When cracks occur in reinforced concrete, the stiffness reduces and deformations increase. The use of prestressed reinforcement makes it possible to limit the cracks in serviceability limit state, which improves the mode of action.
3.2 Fracture mechanics of concrete

3.2.1 Fracture energy

The tension stiffening behavior of concrete, after cracks have occurred, can be described by fracture energy [29]. The fracture energy is a material property, but because it depends on if the concrete is reinforced and the amount and type of the reinforcement there is no fixed value to use. Plain concrete has very low fracture energy, which increases for reinforced concrete and increases even more for prestressed concrete.

Longitudinal micro cracks developed before the compressive strength, \( f_c \) (in this thesis also called \( f_{cm} \)), is reached are, as mentioned in Section 3.1.4 above, assumed to create an inelastic strain in the stress-strain curve of concrete. Markeset and Hillerborg [27] presenting a mechanical model for concrete under compression, called the Compressive Damage Zone (CDZ). The model is based on the assumption that compressive failure occurs in a zone of a limited length. This is called localization, which means that the decreasing branch of the stress-strain curve is dependent on the size of the specimen. Therefore, the stress-strain curve cannot be treated as a material property, which is illustrated in Figure 5, where the influence of the specimen lengths 50 mm, 100 mm and 200 mm on a constant cross section of 100×100 mm² is shown.

Figure 5: Influence of the specimen length on the uniaxial stress-strain curve for a constant cross-section, according to [34].
According to Markeset and Hillerborg [27], there are three curves that can describe the behavior of a concrete specimen under centric pressure. These are shown in Figure 6 together with the complete stress-strain curve, when fracture energy is taken into account. The first curve in the figure is showing the stress-strain curve for concrete that is first loaded up to the compressive strength, $f_{cm}$, and then unloaded. In the second curve the relation between the stress and the average additional strain, $\varepsilon_d$, is showed. This behavior is occurring within the damage zone and is related to the formation of longitudinal cracks and the corresponding lateral strain within this zone. The third curve is showing the stress in relation to the deformation for localized deformations. These three curves together can be summarized in one curve showing stress in relation to both strain and deformation.

As illustrated in Figure 6, the average strain is calculated by

$$\varepsilon_m = \varepsilon + \varepsilon_d \frac{L^d}{L} + \frac{w}{L},$$

(10)

where $\varepsilon$ is the value of $\varepsilon_{c1}$ for the actual concrete type, $L^d$ is the length of the damage zone, $w$ is the deformation and $L$ is the total length of the specimen. In Figure 6, the effect of $\varepsilon_d \cdot L^d / L$ is shown by number 1 and the effect of $w / L$ by number 2.

The opening of a longitudinal crack and a pure tensile crack can be assumed to absorb the same amount of energy [27]. The fracture energy, absorbed per unit crack area, is denoted $G_f$. It can also be explained as the energy needed to form a unit area of crack. According to Kazemi et al. [26], the fracture energy is calculated by

$$G_f = \frac{W_F}{A},$$

(11)
where \( A \) is the area of the fracture surface and \( W_F \) is the total work needed to fracture the concrete. \( W_F \) is also called the fracture strength, which is calculated by

\[
W_F = W_0 + P_0 \cdot u_0, \tag{12}
\]

where \( W_0 \) is the area under the load-displacement curve, \( u_0 \) is the maximum measured deflection and \( P_0 \) is a point load equivalent to the weight of the beam. The CEB-FIP Model Code 1990 (CEB) [34] also deals with the fracture energy and define it as

\[
G_f = G_{f0} \left( \frac{f_{cm}}{f_{cm0}} \right)^{0.7}, \tag{13}
\]

where \( G_{f0} \) is the base value for the fracture energy as a function of the aggregate size, given in Table 2, and \( f_{cm0} \) is a reference value for the concrete compressive strength, equal to 10 MPa [34].

<table>
<thead>
<tr>
<th>( d_{\text{max}} ) [mm]</th>
<th>( G_{f0} ) [Nmm/mm(^2)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>0.025</td>
</tr>
<tr>
<td>16</td>
<td>0.030</td>
</tr>
<tr>
<td>32</td>
<td>0.038</td>
</tr>
</tbody>
</table>

The fracture energy for different strength classes, calculated by Equation (13) and with values for \( G_{f0} \) from Table 2, is reported in Table 3.

<table>
<thead>
<tr>
<th>( d_{\text{max}} ) [mm]</th>
<th>( G_f ) [Nm/m(^2)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\times 10^{3})</td>
<td>(\times 10^{3})</td>
</tr>
<tr>
<td>8</td>
<td>40</td>
</tr>
<tr>
<td>16</td>
<td>50</td>
</tr>
<tr>
<td>32</td>
<td>60</td>
</tr>
</tbody>
</table>

When a constant value for the fracture energy is used, the loss of the tensile strength is linear with the displacement after cracking. Thus, the fracture energy is equal to the area under the stress-crack opening.

### 3.2.2 Stress-strain relations for short term compression

The stress-strain relation for compressed concrete is schematically illustrated in Figure 7, where \( E_{c1} \) is the tangent modulus; \( E_{c1} \) the secant modulus from
the origin to the peak compressive stress; \( f_{cm} \), the strain at maximum stress \( \varepsilon_{c1} = -0.0022 \); and \( \varepsilon_{c,lim} \) is the strain at \( \sigma_{c,lim} = 0.5f_{cm} \).

\[ \text{Figure 7: Stress-strain diagram for uniaxial compression, according to [34].} \]

When the strength of concrete at an age of 28 days, \( f_{cm} \), is known, the tangent modulus is estimated by

\[ E_{ci} = E_{c0} \left( \frac{f_{cm}}{f_{cm0}} \right)^{1/3}, \]  

where \( E_{c0} = 2.15 \cdot 10^4 \) MPa and \( f_{cm0} = 10 \) MPa. Thus, the secant modulus of elasticity becomes \( E_{c1} = f_{cm}/0.0022 \) [34].

The strain \( \varepsilon_{c,lim} \) is calculated from the expression

\[ \frac{\varepsilon_{c,lim}}{\varepsilon_{c1}} = \frac{1}{2} \left( \frac{1}{E_{ci}} + \frac{1}{E_{c1}} \right) \left( \frac{1}{4} \left( \frac{1}{2} \frac{E_{ci}}{E_{c1}} + 1 \right) - 1 \right)^{1/2}. \]  

The approximately stress-strain relation is calculated by

\[ \sigma = \begin{cases} \frac{E_{ci} \varepsilon_{c}}{E_{c1}} \frac{\varepsilon_{c} - (\varepsilon_{c1})^2}{1 + \left( \frac{E_{ci}}{E_{c1}} - 2 \right) \varepsilon_{c} \varepsilon_{c1}} \cdot f_{cm} & \text{for } |\varepsilon_{c}| < |\varepsilon_{c,lim}| \\ \left[- \left( \frac{1}{E_{c,lim}/E_{c1}} \xi - 2 \right) \left( \frac{\varepsilon_{c}}{\varepsilon_{c1}} \right)^2 + \left( \frac{4}{E_{c,lim}/E_{c1} - \xi} \right) \left( \frac{\varepsilon_{c}}{\varepsilon_{c1}} \right)^{-1} \right] & \text{for } |\varepsilon_{c}| > |\varepsilon_{c,lim}| \end{cases} \]  

where

\[ \xi = \frac{4 \left( \varepsilon_{c,lim}^2 \left( \frac{E_{ci}}{E_{c1}} - 2 \right) + 2 \varepsilon_{c,lim} \frac{E_{ci}}{E_{c1}} - \frac{E_{ci}}{E_{c1}} \right)}{\left( \frac{\varepsilon_{c,lim}}{E_{c1}} \frac{E_{ci}}{E_{c1} - 2} + 1 \right)^2}. \]
The stress-strain diagram for various concrete classes, calculated by Equations (16)-(18), is illustrated in Figure 8, which implies that the curve for concrete classes above C50/60, meaning high strength concrete, should have a steep slope.

![Stress-strain diagram for concrete in compression calculated by Equations (16)-(18), according to [34].](image)

3.2.3 Stress-strain and stress-crack relations for short term tension

When using the design code CEB, the stress-strain relation is calculated for uncracked concrete and for cracked concrete subjected to tension. For uncracked concrete, the behavior when subjected to tension is described by Figure 9.

![Stress-strain diagram for uncracked cross section subjected to uniaxial tension, according to [34].](image)

For the bilinear stress-strain relation, when \( \varepsilon_{ct} \leq 0.00015 \), the stress is calculated by
For a cracked section, the bilinear stress-crack opening relation, illustrated in Figure 10, may be used.

\[
\sigma_{ct} = \begin{cases} 
\frac{E_{ct} \varepsilon_{ct}}{f_{ctm}} & \text{for } \sigma_{ct} \leq 0.9 f_{ctm} \\
0.1 f_{ctm} - \frac{f_{ctm}}{0.00015 - 0.9 f_{ctm}/E_{ct}} (0.00015 - \varepsilon_{ct}) & \text{for } 0.9 f_{ctm} \leq \sigma_{ct} \leq f_{ctm}
\end{cases}
\]

For a cracked section, the bilinear stress-crack opening relation, illustrated in Figure 10, may be used.

![Stress-crack opening diagram for uniaxial tension](image)

The stress for \( \varepsilon_{ct} > 0.00015 \) is calculated by

\[
\sigma_{ct} = \begin{cases} 
0.15 f_{ctm} (w_e - w) & \text{for } 0 \leq \sigma_{ct} \leq 0.15 f_{ctm} \\
f_{ctm} \left(1 - 0.85 \frac{w}{w_1}\right) & \text{for } 0.15 f_{ctm} \leq \sigma_{ct} \leq f_{ctm}
\end{cases}
\]

where \( w_1 \) is the crack opening, given in mm, for \( \sigma_{ct} = 0 \), calculated by

\[
w_1 = \alpha_F \frac{G_f}{f_{ctm}},
\]

and the crack opening, given in mm, for \( \sigma_{ct} = 0.15 f_{ctm} \) is calculated by

\[
w_c = 2 \frac{G_f}{f_{ctm}} - 0.15 w_c.
\]

The value of the coefficient \( \alpha_F \) for different maximum aggregate size is given in Table 4 and the fracture energy, \( G_f \) is described in Section 3.2.1 above.

<table>
<thead>
<tr>
<th>( d_{max} )</th>
<th>8</th>
<th>16</th>
<th>32</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \alpha_F )</td>
<td>8</td>
<td>7</td>
<td>5</td>
</tr>
</tbody>
</table>
3.3 Properties and behavior of steel

The steel response in tensile strain is showed in Figure 11. The behavior is linear up to the yield stress, $f_{yk}$. When the tensile strength, $f_t$, of the steel is reached, the corresponding strain is the ultimate strain at maximum load, $\varepsilon_{uk}$. The value of the factor $k = f_t / f_{yk}$ depends on the steel class and it is given in Appendix C in EN 1992 [20].

![Figure 11: The stress-strain relation of hot rolled reinforcing steel, according to EN 1992 [20].](image)

3.3.1 Prestressed steel

The stress-strain relation for prestressed steel is illustrated in Figure 12, where the characteristic value of the tensile strength is called $f_{pk}$ and $f_{p0.1k}$ is the 0.1%-limit, which means the characteristic strength at the strain 0.1% [20]. The characteristic value for the strain at maximum load is given by $\varepsilon_{uk}$ and $E_p$ is the design value for the modulus of elasticity.

![Figure 12: The stress-strain relation for prestressing steel, according to [20].](image)
The values for the stress-strain relation for the steel used in this study are reported in Table 5.

<table>
<thead>
<tr>
<th>Stress $\sigma_s$ [MPa]</th>
<th>Strain $\varepsilon_S$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1265</td>
<td>0.006</td>
</tr>
<tr>
<td>1600</td>
<td>0.008</td>
</tr>
<tr>
<td>1740</td>
<td>0.012</td>
</tr>
<tr>
<td>1860</td>
<td>0.035</td>
</tr>
</tbody>
</table>

3.4 Designing poles and columns

3.4.1 Mode of actions of slender poles and columns

There are two types of columns; braced or non-braced columns. A braced column has support in both ends, while a non-braced column is only fastened in the bottom, which gives them different behavior when they are loaded (shown in Figure 13). While the braced column is bending almost without any movement of the top, the non-braced column is bending out to the side. Depending on the type of column, the effective length, $l_0$, will be different.

![Figure 13: Different types of columns, a) is showing the behavior of non-braced column and b) the behavior of a braced column [20].](image)

From now, this chapter will handle non-braced columns, because poles carrying the lines on the grid are, like non-braced columns, only fastened at the bottom.
3.4.2 Elementary cases for beams with a fixed attachment

The test setup for a bending test according to the European standard for masts and poles, EN 12843, [20], described in Section 3.4.4 below, can be compared with the elementary case for a beam with a fixed attachment, illustrated in Figure 14a). The bending moment at the support, $M_A$, is calculated by

$$M_A = PL,$$  \hfill (25)

where $P$ is a point load and $L$ is the total length from the ground to the point where the force is applied. For a beam with a constant geometric section the second moment of inertia is depending on the outer and inner diameter, $d_1$ and $d_2$, illustrated in Figure 14b).

The bending moment and the moment of inertia are known, the compression stresses in the concrete on distance $z$ from the center of the beam can be calculated by

$$\sigma_c = \frac{M}{I} z.$$  \hfill (27)

3.4.3 Prevented transverse expansion

When a pressure is applied from the side of a, for example, pole or column it counteracts the axial pressure, which increases the rigidity and the compressive strength of the concrete because of the prevented transverse expansion, see Figure 15, where $\sigma_2$ and $\sigma_3$ are the compressive stresses in the cross section [20]. The increased value on the compressive strength, $f_{ck,e}$, is calculated by
where the compressive stress $\sigma_2 = \sigma_3$.

\[
f_{ck,c} = \begin{cases} 
    f_{ck} \left( 1.000 + 5.0 \frac{\sigma_2}{f_{ck}} \right) & \text{for } \sigma_2 \leq 0.05 f_{ck} \\
    f_{ck} \left( 1.125 + 2.5 \frac{\sigma_2}{f_{ck}} \right) & \text{for } \sigma_2 > 0.05 f_{ck}
\end{cases}
\]  

(28) 

3.4.4 Design loads for poles to the power line grid

Loads to consider are permanent loads, wind loads, ice loads, assembly loads, maintenance loads and security loads. The basic equation for design loads is

\[
E_d = \sum \gamma_G \cdot G_K + \psi \cdot \sum \gamma_Q \cdot Q_{nk}.
\]  

(30)

where the reduction factor $\psi$ is equal to 1.0, $\gamma_G$ is a partial factor for permanent actions and $\gamma_Q$ a partial factor for variable loads [15]. $G_K$ is the characteristic value for permanent loads and $Q_{nk}$ the characteristic value for all variable actions, such as wind and ice loads. Partial factors for supports for the Swedish grid, which in this study means for the poles, are reported in Table 6.

\begin{table}[h]
\centering
\caption{Partial factors for design loads, taken from [15].}
\begin{tabular}{|l|c|c|c|c|c|}
\hline
Action & Symbol & Load combination  \\
& & 1 & 2 & 3 \\
\hline
Permanent actions & $\gamma_G$ & 1.1 & 0.94 & 1.0 \\
Permanent actions from soil and ground water & $\gamma_G$ & 1.1 & 1.1 & 1.0 \\
Variable actions & $\gamma_Q$ & 1.43 & 1.43 & 1.0 \\
Dynamic construction and maintenance loads & $\gamma_G$ & 1.8 & 1.8 & 1.3 \\
\hline
\end{tabular}
\end{table}

Figure 15: Stress-strain relation for concrete when the transverse expansion is prevented, where A is without prevented expansion, according to [20].
Load combination number 1 is determinant for poles and load combination number 2 for foundations [13]. Load combination number 3 should be used when checking deformations and cracks in the concrete in the serviceability limit state.

3.4.5 Bending test on poles

Requirements for the performance of bending tests on poles are given in the European standard for masts and poles, EN 12843, [20]. Tests carried out according to the standard should be done on poles placed horizontal, fastened in one end and resting on supports preventing the pole from bending due to its own weight. A principle picture of the bending test according to the standard is shown in Figure 16a, where $F_n$ is the force applied and $f_n$ is the corresponding deflection.

The test set up when RISE conducted bending tests on poles made of wood as mentioned in Section 1.1, is illustrated in Figure 16b. The total height of the pole tested was 12 m and the distance from the top to the place where the load was applied, $h_a$, was 0.16 m.

![Figure 16: Test set up for bending test a) according to the standard for masts and poles [20] and b) according to RISE [36].](image)

The mean failure load of five creosote impregnated wood poles tested was 9.2 kN, which gives a bending moment capacity at the support of 90.53 kNm. Maximum displacement of the top measured was 3.1 m and the mean value of the displacement from the test was 2.6 m.

3.5 The Finite Element Method

The finite element method is a numerical approximation method, where a structure is divided into a finite number of parts, called finite elements. These elements create a finite element mesh [37]. The behavior of each element can be determined by applying different material properties and
boundary conditions. By defining the behavior for all elements and then assembling them to one body, an approximate solution for the whole structure can be obtained.

3.5.1 The modelling software BRIGADE/Plus

BRIGADE/Plus is a tool box for modelling and analysis of structures [38]. The software includes predefined loads and load combinations in accordance with Eurocode. The results from analyses made in the program can be visualized in 3D plots and 2D graphs. BRIGADE/Plus includes a prestress-module, which is of great benefit for this study. When using the prestress-module in BRIGADE/Plus, short term losses, due to friction and anchorage, are calculated by atomization.

3.5.2 Fracture mechanics in BRIGADE/Plus

The concrete behavior is represented in BRIGADE/Plus by using the concrete damage plasticity model, which includes concrete plasticity, compressive behavior, described by the stress-strain relation, and tensile behavior, described by the stress-crack-opening behavior. The concrete damage plasticity model assumes that the failure mechanisms are tensile cracking or compressive crushing of the concrete [39].

3.5.2.1 Defining compressive behavior

As described in Section 3.2.2, the compressive behavior is described by a stress-strain relation. The definition of the compressive inelastic strain, or crushing strain, $\varepsilon_c^{\text{in}}$, according to the Abaqus manual is illustrated in Figure 17 [38]. In this definition, hardening data are given in terms of the inelastic strain instead of the plastic strain, $\varepsilon_c^{\text{pl}}$.

![Figure 17: Definition of the compressive inelastic strain $\varepsilon_c^{\text{in}}$, according to [38].](image)

In the definition above, the compressive inelastic strain is calculated by
where \( \varepsilon_{0c} = \sigma_c/E_0 \), as illustrated in Figure 17. When unloading, the behavior is described by compressive damage curves, \( d_c - \tilde{\varepsilon}_{c}^{ln} \), where \( d_c \) is a variable between 0 and 1, defining the size of the decreasing stiffness [40]. The inelastic strain is automatically converted to plastic strain by the expression

\[
\tilde{\varepsilon}_{c}^{pl} = \tilde{\varepsilon}_{c}^{ln} - \frac{d_c}{(1 - d_c) E_0} \sigma_c
\]  

(32)

3.5.2.2 Defining tension stiffening

The tension stiffening behavior can be specified by using a stress-strain relation or by applying a fracture energy cracking criterion [38]. In this study, the later alternative is used. The energy cracking criterion is based on Hillerborg’s fracture energy proposal, mentioned in Section 2.3 and 3.2. With this approach the fracture energy, \( G_f \), is assumed to be a material parameter and the brittle behavior of the concrete is characterized by a stress-crack opening relation. In BRIGDE/Plus, this fracture energy cracking model can be used by specifying the post failure stress as a tabular function of the cracking displacement, which is illustrated in Figure 18a, or by specifying the fracture energy, \( G_f \), as a material property, where the fracture energy is the area under the linear strength loss between \( \sigma_{t0} \) and the displacement when the stress is zero, \( u_0 = 2G_f/\sigma_{t0} \), illustrated in Figure 18b.

![Figure 18: The fracture energy cracking model described in a) by a tabular function of the cracking displacement and in b) by using the fracture energy as a material property, according to [38].](image)

3.5.2.3 Concrete plasticity

Concrete plasticity is described by the dilation angle, \( \psi \); the eccentricity; the relation between the initial equibiaxial compressive yield stress, \( f_{b0} \), and the initial uniaxial compressive yield stress, \( f_{c0} \); the yield surface in the deviatory plane, \( K \); and the viscous parameter.
The dilation angle, $\psi$, describes the relation between hydrostatic pressure and the concrete strength and should be in the interval $32^\circ \leq \psi \leq 38^\circ$ [40]. Default values for the rest of the terms, according to the ABAQUS manual [38], are reported in Table 7.

<table>
<thead>
<tr>
<th>Eccentricity</th>
<th>$f_{bd}/f_{co}$</th>
<th>$K$</th>
<th>Viscosity parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>1.16</td>
<td>0.67</td>
<td>0</td>
</tr>
</tbody>
</table>

### 3.5.1.3 Prestressed reinforcement in BRIGADE/Plus

In BRIGADE/Plus, there is also a prestress-module. Instead of creating tendons in the part-module, the prestressed reinforcement is created directly in this module, where material, cross section area and location are assigned.

There are two different functions to use in BRIGADE/Plus, adding prestressed tendons and defining the prestress forces in the tendons [37]. The force in each tendon is calculated by

$$ F_x = F_{\text{ini}} \cdot e^{-(\mu \alpha + kx)} $$

where $F_x$ is the initial stress prior to anchoring at the distance $x$ from the cable end; $F_{\text{ini}}$ the initial stress prior to anchoring at the cable end; $\mu$ the friction coefficient; $\alpha$ the initially intended, cumulative angle change from the positions where the tendon force is applied, given in radians; $x$ the distance from the position where the force is applied; and $k$ the wobble friction loss, given per meter, due to unintended deviation of the duct [37]. The maximum cable stress after anchor set, $F_{\text{max}}$, and the initial stress prior to anchoring, $F_{\text{ini}}$, are defined by the user, while the maximum distance $x_{\text{max}}$, $F_x$ and the force after anchor set is calculated by the software BRIGADE/Plus. The relation between the force and the maximum distance, $x_{\text{max}}$, after anchor set, $F_x$ and $x_{\text{max}}$ are illustrated in Figure 19.

![Figure 19: Stress after anchor set, according to [37]. $F_{\text{ini}}$ is the initial stress prior to anchoring and $F_x$ is the initial stress prior to anchoring at the distance $x$ from the cable end.](image-url)
4. Methods and implementation

In this chapter, the methods used in this project are described, as well as the aim and purpose of their choice and how they are implemented. In this project, quantitative methods are used. Empirical data is collected from a test field and numerical methods are used to make analyses of models created in the software BRIGADE/Plus.

4.1 Field test

A field test, when a concrete pole was drawn to failure, was conducted. The aim with this test was to apply different loads and observe deflection of the pole and the failure when the ultimate limit state was reached. The purpose with the test was to capture the behavior, to get a perception about how the pole deflects and the maximum bending strength. The result was then used for calibrating the FE-model, created in BRIGADE/Plus.

4.1.1 Test model

Measurements of the pole tested and its prestressed reinforcement are reported in Table 8. Some of the values are approximated, because exact measurements were not possible to perform at the testing site and. Other values are picked from drawings and calculations of the actual pole.

<table>
<thead>
<tr>
<th>Property</th>
<th>Measurement [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height above ground (approximately)</td>
<td>19 000</td>
</tr>
<tr>
<td>Support length under ground (approximately)</td>
<td>2000</td>
</tr>
<tr>
<td>Diameter at the top</td>
<td>444</td>
</tr>
<tr>
<td>Diameter at the bottom</td>
<td>822</td>
</tr>
<tr>
<td>Thickness concrete (top/bottom)</td>
<td>100/110.5</td>
</tr>
<tr>
<td>Diameter of steel lines</td>
<td>11.3</td>
</tr>
</tbody>
</table>

Properties for the concrete class and steel type used in this study are presented in Section 3.1 and 3.3 respectively.

4.1.2 Test set up

A wire was fastened around the pole, 3 m down from the top, as shown in the simplified sketch in Figure 20. Pole diameter, $\varnothing$, and pole thickness, $t$, are marked in the figure. At one side of the wire a dial indicator, measuring half of the applied force, $F$, was attached.
Figure 20: In a) the whole pole is shown, with the load $F$ attached 3 m from the top and in b) the test set up is seen from above with the placement for the dial indicator, measuring the force, market.

Figure 21: Test set up in the field, with a wire fastened in the top.

The test set up out in the field is shown in Figure 21, with the wire fastened. The wire was also attached to a tow truck with measurement equipment, showing the applied force. A camera was placed out on the road to be able to capture the procedure and the failure.
The procedure of the test was to pull the wire in by the tow truck and measure the force needed to do that. The test was conducted in two parts. The first part was to increase the load slowly to be able to observe the loading and the corresponding displacement. After reaching the total load of around 100 kN, the behavior of the pole was observed when it was unloaded. The second part of the test was to increase the load until failure.

4.2 Analysis in BRIGADE/Plus

A model of the concrete pole, drawn to failure in the field test, is created in the software BRIGADE/Plus 6.1. Known material properties and dimensions are used and the behavior of the pole is captured by adding loads and boundary conditions. This model is the basic template when poles with different geometries are evaluated. Crack initiation and its effect on the pole are also evaluated according to the sections below.

In the sections below, different modules used in BRIGADE/Plus are described, as well as how they are used when creating and examining the models.

4.2.1 Geometrical modeling in the part-module

A total number of 6 models were created in BRIGADE/Plus. Their geometry is reported in Table 9. The height, \( h \), of the poles is counted from the bottom to the top. When the pole is in use, about two meters of the pole are under ground, embedded in a foundation. The top diameter, \( \Phi_{\text{top}} \), and the bottom diameter, \( \Phi_{\text{bottom}} \), given in the table are the outer diameters. The thickness, \( t \), illustrated together with the diameter in Figure 20, is indicating the concrete wall thickness at top/bottom.

<table>
<thead>
<tr>
<th>Model no.</th>
<th>Height, ( h ) [m]</th>
<th>( \Phi_{\text{top}} ) [mm]</th>
<th>( \Phi_{\text{bottom}} ) [mm]</th>
<th>Thickness, ( t ) [mm]</th>
<th>Weight [tonne]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>21</td>
<td>444</td>
<td>822</td>
<td>100/110.5</td>
<td>9.24</td>
</tr>
<tr>
<td>2</td>
<td>21</td>
<td>444</td>
<td>822</td>
<td>80/90</td>
<td>7.20</td>
</tr>
<tr>
<td>3</td>
<td>21</td>
<td>444</td>
<td>822</td>
<td>60/70</td>
<td>5.85</td>
</tr>
<tr>
<td>4</td>
<td>21</td>
<td>424</td>
<td>802</td>
<td>50/50</td>
<td>4.46</td>
</tr>
<tr>
<td>5</td>
<td>21</td>
<td>292</td>
<td>670</td>
<td>50/50</td>
<td>3.44</td>
</tr>
<tr>
<td>6</td>
<td>12</td>
<td>454</td>
<td>670</td>
<td>50/50</td>
<td>2.32</td>
</tr>
</tbody>
</table>

Table 9: Geometries of models created and analysed in BRIGADE/Plus.
4.2.2 Material model for concrete

The material model for concrete, created in the property-module, was based on the concrete strength 105 MPa for 100 mm cubes. Material parameters for this quality are found in Section 3.1. Both elastic and concrete damage plasticity models were created. The latter alternative is able to describe the behavior of prestressed concrete in the cracked state. Young’s modulus of elasticity, $E_{cm}$, and the mean value for the tensile strength, $f_{ctm}$, are calculated by Equations (3) and (4).

The elastic material model included density, $\rho$, Young’s modulus, $E_{cm}$, and Poisson’s ratio, $\nu$. These values are presented in Table 10.

<table>
<thead>
<tr>
<th>Material name</th>
<th>$\rho$ [kg/m$^3$]</th>
<th>$E_{cm}$ [GPa]</th>
<th>$\nu$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>2400</td>
<td>39.70</td>
<td>0.2</td>
</tr>
</tbody>
</table>

The concrete damage plasticity model includes plasticity under compression load, described by the stress-strain relation, and stress-crack-opening behavior for tension loading.

Plasticity for the concrete is described by the dilation angle, eccentricity, $f_{b0}/f_{c0}$, the yield surface in the deviatory plane, $K$, and the viscous parameter, reported in Table 11.

<table>
<thead>
<tr>
<th>Dilation Angle, $\psi$</th>
<th>Eccentricity</th>
<th>$f_{b0}/f_{c0}$</th>
<th>$K$</th>
<th>Viscosity parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>36.31</td>
<td>0.1</td>
<td>1.16</td>
<td>0.67</td>
<td>0</td>
</tr>
</tbody>
</table>

The stress-strain relation, describing the compressive behavior, was calculated according to Section 3.2.2. Values used are reported in Table 12 and the stress-strain relation is illustrated in Figure 22.
**Table 12: Stress-strain relation for compressed concrete.**

<table>
<thead>
<tr>
<th>Stress, $\sigma_c$ [MPa]</th>
<th>Strain, $\varepsilon_c$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0.000</td>
</tr>
<tr>
<td>22.90</td>
<td>0.0005</td>
</tr>
<tr>
<td>36.30</td>
<td>0.0010</td>
</tr>
<tr>
<td>51.50</td>
<td>0.0015</td>
</tr>
<tr>
<td>63.60</td>
<td>0.0020</td>
</tr>
<tr>
<td>70.75</td>
<td>0.0025</td>
</tr>
<tr>
<td>71.60</td>
<td>0.0027</td>
</tr>
<tr>
<td>69.60</td>
<td>0.0030</td>
</tr>
<tr>
<td>53.10</td>
<td>0.0035</td>
</tr>
<tr>
<td>38.80</td>
<td>0.0037</td>
</tr>
<tr>
<td>34.30</td>
<td>0.00375</td>
</tr>
<tr>
<td>14.20</td>
<td>0.00425</td>
</tr>
<tr>
<td>8.28</td>
<td>0.00475</td>
</tr>
<tr>
<td>5.56</td>
<td>0.00525</td>
</tr>
<tr>
<td>4.04</td>
<td>0.00575</td>
</tr>
<tr>
<td>3.08</td>
<td>0.00625</td>
</tr>
<tr>
<td>2.44</td>
<td>0.00675</td>
</tr>
<tr>
<td>1.98</td>
<td>0.00725</td>
</tr>
<tr>
<td>1.65</td>
<td>0.00775</td>
</tr>
</tbody>
</table>

**Figure 22: Stress-strain relation for compressed concrete.**

The tensile behavior was described by the stress-crack opening relation and calculated according to Section 3.2.3. The relation between the stress and the width of the crack opening was adjusted to capture the behavior of the pole tested in the field. Values for the stress-crack opening relation are reported in Table 13 and illustrated in Figure 23.
Table 13: Stress-crack opening behavior for concrete in tension.

<table>
<thead>
<tr>
<th>Stress, $\sigma_r$ [MPa]</th>
<th>Crack widht, $w$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.450</td>
<td>0.0</td>
</tr>
<tr>
<td>0.675</td>
<td>0.3</td>
</tr>
<tr>
<td>0.450</td>
<td>0.5</td>
</tr>
<tr>
<td>0.000</td>
<td>3.0</td>
</tr>
</tbody>
</table>

Figure 23: Stress-crack opening relation for concrete in tension in the cracked state.

4.2.3 Material model for steel

The material model for steel is based on the high-strength steel quality Y860S3-6.8. Material parameters for this quality are found in Section 3.3.

The elastic material model included density, $\rho$, Young’s modulus, $E_{cm}$, and Poisson’s ratio, $\nu$. Values for these properties are presented in Table 14.

Table 14: Properties for elastic material models.

<table>
<thead>
<tr>
<th>Material name</th>
<th>$\rho$ [kg/m$^3$]</th>
<th>$E_{cm}$ [GPa]</th>
<th>$\nu$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>7800</td>
<td>200.00</td>
<td>0.3</td>
</tr>
</tbody>
</table>

The plastic material model for steel is based on the stress-strain relation, described in Section 3.3.1. The plastic strain is equal to zero when the steel is reaching the yield stress. Using the relation given for the steel, with the strain for the yield stress equal to zero, results in the plastic stress-strain relation presented in Table 15 and illustrated in Figure 24.
Table 15: Stress-plastic strain relation for the steel.

<table>
<thead>
<tr>
<th>Stress $\sigma_s$ [MPa]</th>
<th>Plastic strain $\varepsilon_s$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1265</td>
<td>0.000</td>
</tr>
<tr>
<td>1600</td>
<td>0.002</td>
</tr>
<tr>
<td>1740</td>
<td>0.006</td>
</tr>
<tr>
<td>1860</td>
<td>0.029</td>
</tr>
</tbody>
</table>

Figure 24: Stress-plastic strain relation for the steel.

4.2.4 Element types and sizes

The elements used for the concrete pole were 3D tetrahedral elements. The global element size varied between 0.065-0.045 m for the different models, depending on the concrete thickness. For the tendons, the element size was chosen in the prestress-module, where it was set to 0.01 m.

4.2.5 Load steps

Three load steps were created: Gravity, Prestress and Pressure. All features chosen for these steps are reported in Table 16, where $T_s$ is the time period for the step; $I$ is the increment size; $I_{min}$ is the minimum increment size; and $I_{max}$ is the maximum increment size. For all three steps default values for dissipated energy fraction and adaptive stabilization are used. For both the Gravity-step and the Prestress-step, the matrix storage under the tab “Other” in the step editor is unchanged, which means that solver default is used. In the Pressure-step, the matrix storage is chosen to be unsymmetrical.
Table 16: Features for the three different steps

<table>
<thead>
<tr>
<th>Step name</th>
<th>Basic</th>
<th>Incrementation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$T_s$</td>
<td>Ngeom</td>
</tr>
<tr>
<td>Gravity</td>
<td>1</td>
<td>Off</td>
</tr>
<tr>
<td>Prestress</td>
<td>1</td>
<td>Off</td>
</tr>
<tr>
<td>Pressure</td>
<td>250</td>
<td>On</td>
</tr>
</tbody>
</table>

4.2.6 Loads and boundary conditions

Two different loads were applied, acting on the pole: gravity and pressure. Gravity was applied on the whole model downwards in the length direction. Pressure was applied on an area, illustrated in Figure 25b. For Models 1 to 5, that area had a height, $h$, of 0.25 m and the length of the half circumference, placed on a distance, $d$, 3 m down from the top. For Model 6, the height, $h$, was changed to 0.04 m and distance, $d$, to 0.16 m. Displacements presented in the results are measured at two different points. The point $D_1$ is placed in the area where the pressure is applied and $D_2$ is the top of the pole. Both points are marked in Figure 25b.

![Figure 25](image)

Figure 25: a) The whole pole, where b) showing the area where the pressure is applied and c) the boundary condition simulating support conditions. In b) the points $D_1$ and $D_2$ where displacement are measured, are pointed out.

Boundary conditions, illustrated in Figure 25c, were applied, simulating the support conditions. Displacements in X-, Y- and Z-direction were set equal to zero for the 2 m long section on the lower part of the pole.
4.2.7 Prestressed tendons and prestressing force

The amount and size of the reinforcement were the same as in the pole tested in the field, which means 40 prestressed lines with a diameter of 11.28 mm. The tendons were created, with the concrete pole as a solid host, in the prestress manager. They were evenly distributed over the cross section, all with the same distance to the outside of the concrete. Their placement was defined by using coordinates.

The number of lines, \( N_s \), the cross-section area of one tendon, \( A_s \), the total cross section area of all tendons, \( A_{stot} \), and the distance from the center of a tendon to the outside of the concrete, \( a \), are reported in Table 17. In the Prestress Load manager the force applied in the tendons, \( F_s \), were defined together with the friction coefficient, \( \mu \), the friction loss, \( \Delta \mu \), and the element size for the tendons, \( e_{ten} \). The force was applied in both ends of the tendons.

<table>
<thead>
<tr>
<th>Model no</th>
<th>( N_s ) [pcs]</th>
<th>( A_s ) [mm²]</th>
<th>( A_{stot} ) [mm²]</th>
<th>( a ) [mm]</th>
<th>( \mu ) [-]</th>
<th>( \Delta \mu ) [-]</th>
<th>( F_s ) [kN]</th>
<th>( e_{ten} ) [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1-4</td>
<td>40</td>
<td>100</td>
<td>4000</td>
<td>36</td>
<td>0.20</td>
<td>0.30</td>
<td>108</td>
<td>0.01</td>
</tr>
<tr>
<td>Model 5</td>
<td>40</td>
<td>100</td>
<td>4000</td>
<td>25</td>
<td>0.20</td>
<td>0.30</td>
<td>54</td>
<td>0.01</td>
</tr>
<tr>
<td>Model 6</td>
<td>40</td>
<td>100</td>
<td>4000</td>
<td>25</td>
<td>0.20</td>
<td>0.03</td>
<td>2.5</td>
<td>0.01</td>
</tr>
</tbody>
</table>

The size of the prestressing force applied for Model 1 was chosen from descriptions of the pole tested on the field. Because Models 5 and 6 were crushed lengthwise when using the same prestressing force, the tendon forces were lower for these models.

4.3 Analysis of the results

4.3.1 Analysis of the results from the field test

From the outcome of the field test, the bending shape of the pole and the deflection for different loads was analyzed by studying the film that captured the test.

The bending moment capacity at the support and the compressive elastic stress was calculated according to elementary cases for beams, described in Section 3.4.2. The results of these calculations are compared with results of the FE-models.

When performing the field test the force needed to draw the pole to failure was measured in tonnes. Therefore, results are sometimes given in both kN and tonnes.
4.3.2 Analysis of the results in BRIGADE/Plus

In BRIGADE/Plus the results were analyzed in the visualization module. Load-displacement curves were created by using the load together with the time-displacement relation for the point $D_1$, pointed out in Figure 25 in Section 4.2.6.

The displacement, used as a comparison with the field test, was the displacement along a path on the back on the pole. Loads of interest were the loads measured in the field, which means 58.90 kN, 68.74 kN and 102.62 kN.

Stresses and strains were analyzed using probe values in the visualization module. Values reported in the result are the highest values found for the integration points and for the centroid value, which is a mean value for all integration points in one element. Relations between stresses and strains or load and displacement are plotted by using ODB History Output.

When the behavior of the FE-model was consistent with the behavior observed at the test site, stresses and strains for the failure load measured at the field was analyzed in the FE-model. Compressive stresses in the concrete obtained for that load were defined as the maximum admissible compressive strength and used to define the failure load for the optimized FE-models. The maximum tensile strength in the concrete was defined as the maximum value used in the material model.
5. Results

5.1 Field test

The outcome of the field test was in the form of measurements of the force and observations of displacements, also captured on film. The test was two-part procedure, were the first part was to observe displacements while increasing the load, followed by unloading, and the second part was to increase the load until failure. Load and displacement at the location where the load was applied, $D_1$, and displacement at the top, $D_2$, are reported in Table 18. At the test site the load was measured in tonnes and in the table below it is reported in both tonnes and kN.

![Table 18: Force and displacement and failure load from the field test.](image)

The failure load was 103 kN (10.5 tonnes), which corresponds to a bending moment capacity at the support of 1650 kNm. The type of failure observed was a compression failure in the concrete. The results are reported in Table 19, together with the elastic compression stress, calculated for the failure load by using elementary cases.

The required capacity according to the calculations for the pole is 917 kNm in serviceability limit state, which is compared with the crack moment capacity, and 1406 kNm in ultimate limit state. The calculated crack moment capacity for the pole is 912 kNm and the bending moment capacity 1602 kNm. All results are reported in Table 19 together with corresponding elastic compression stresses in the concrete.

![Table 19: Maximum bending moment, crack moment and compression stresses in concrete from the field test, calculations and requirements on the pole.](image)

5.2 Base Model 1

Model 1 is used as reference and template to which the other models are compared. Therefore, the results for Model 1 will be presented thoroughly in
a separate section, where it is also compared with the field test model. The rest of the models will be presented more general in following section.

Results of interest from Model 1 are the failure load, the type of failure (compression or tension), crack load, displacement, stresses and strains.

5.2.1 Maximum load, failure load and crack load

The maximum load is the load reached when the FE-model failed in BRIGADE/Plus because no more solutions where found. The failure load for the model is the load reached when stresses between 95-100 MPa on compression side are achieved. If the tensile stress of 4.45 MPa on the tension side is reached, the failure load depends on whether the ultimate compression strength in the concrete or the ultimate tensile strength in the tendons is reached first.

The load-displacement relation, illustrated in Figure 26, shows the behavior of the pole when the load increases. Displacements are measured in the point $D_1$, according to Figure 25 in Section 4.2.6.

![Figure 26: Load-displacement curve for Model 1, with crack load, failure load and maximum load pointed out.](image)

The crack-load is determined to 30.1 kN, which is the load achieved when the curve is changing direction the first time. Maximum load, when the FE-model fails, is 112 kN while failure load, when stresses of magnitude 95-100 MPa are reached, is 102 kN.

Maximum load, failure load and the load when the first crack occurred in the FE-model are reported in Table 20, together with the corresponding moment capacity at the support. Failure load and the corresponding bending moment from the field test are also reported in the table, but the crack load and corresponding moment are not known from the test. The capacity calculated
for the pole that was tested in the field is reported, both as bending moment capacity, crack moment capacity and the corresponding load applied 3 m down from the top.

Table 20: Failure load, crack load and moments for Model 1 and the field model.

<table>
<thead>
<tr>
<th>Model</th>
<th>Maximum load [kN]</th>
<th>Failure load [kN]</th>
<th>Bending moment capacity [kNm]</th>
<th>Crack load [kN]</th>
<th>Crack moment capacity [kNm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1 (FE)</td>
<td>112</td>
<td>102</td>
<td>1630</td>
<td>30.1</td>
<td>482</td>
</tr>
<tr>
<td>Field test</td>
<td>–</td>
<td>103</td>
<td>1650</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Field calculations</td>
<td>–</td>
<td>100</td>
<td>1602</td>
<td>57.0</td>
<td>912</td>
</tr>
</tbody>
</table>

The failure load for the FE-model model agrees with the field test with a factor 0.99.

5.2.2 Displacement

Displacements are compared to displacements observed at the test site for three different loads. The loads and the corresponding displacement from the field and for Model 1 are reported in Table 21 and illustrated in Figure 27. In the table, displacements are given for positions $D_1$ and $D_2$ and in the figure along the entire length of the pole.

Table 21: Load and deflection result from the field test and for Model 1.

<table>
<thead>
<tr>
<th>Load [tonne]</th>
<th>Load [kN]</th>
<th>Field $D_1$ [mm]</th>
<th>Field $D_2$ [mm]</th>
<th>Model 1 $D_1$ [mm]</th>
<th>Model 1 $D_2$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.00</td>
<td>58.90</td>
<td>550</td>
<td>800</td>
<td>592</td>
<td>765</td>
</tr>
<tr>
<td>7.00</td>
<td>68.74</td>
<td>690</td>
<td>980</td>
<td>782</td>
<td>1015</td>
</tr>
<tr>
<td>10.45</td>
<td>102.62</td>
<td>1460</td>
<td>1950</td>
<td>1540</td>
<td>2000</td>
</tr>
</tbody>
</table>
Figure 27: Displacement for the loads 60, 70 and 102 kN for the field test (red line) and the FE-model (blue line).

The load-displacement relations from the field test and for Model 1 are illustrated in Figure 28. The load and displacement are both measured in point $D_1$, where the pressure is applied. There was no information about the displacements for loads less than 40 kN from the field test. Therefore, results from the field test and the FE-model are not comparable before that load is reached. Loads marked for the field test are the loads measured. For Model 1, loads are marked when the curve is changing direction.

Figure 28: Load-displacement relation from the field test (red line) and for Model 1 (blue line).
5.2.3 Stresses and strains in the concrete

The highest values of compressive stress in the concrete are obtained along the edge of the support. High values of tensile stresses in the concrete are found along the edge of the support, but also on the sides of the pole and higher up, under the area where the pressure is applied. Maximum values of the strain are found along the edge of the support on the tension side.

Stresses and strains at the support, for the failure load 102 kN, are reported in Table 22 for the compression and the tension side. Stresses reported are the axial stresses, S33, acting in vertical direction, and the principal stress. Values reported for plastic strain are PE33, acting in vertical direction, and the principal plastic strain. Two different values of the result are reported. Firstly, obtained by using the centroid position, and secondly by looking at the highest value of the interpolation points. When using the centroid position, a mean value of all the integration points in the element is obtained.

<table>
<thead>
<tr>
<th>Stress or strain</th>
<th>Compression side</th>
<th>Tension side</th>
</tr>
</thead>
<tbody>
<tr>
<td>S33 [MPa]</td>
<td>-99.5</td>
<td>-123.7</td>
</tr>
<tr>
<td>Principal stress [MPa]</td>
<td>-96.0</td>
<td>-138.3</td>
</tr>
<tr>
<td>PE33 [-]</td>
<td>-0.0017</td>
<td>-0.0021</td>
</tr>
<tr>
<td>Principal strain [-]</td>
<td>-0.0019</td>
<td>-0.0023</td>
</tr>
</tbody>
</table>

The axial stresses and the plastic strains along the whole pole are shown in Figure 29. The highest values of stresses on the compression side are found at the edge of the support. On the tension side, the highest values of the axial stresses are found on the sides down at the support, as shown in Figure 30, and further up, under the area where the pressure is applied. The highest values of strains are found along the support on both the compression and the tension side, but in both cases high values are also found further up on the pole. Maximum stresses at the support, reported in Table 22, are illustrated in Figure 30 and the highest values of plastic strain in Figure 31.
Figure 29: The axial stress, $S_{33}$, is shown in a) on the compression side and in b) on the tension side. Plastic strain, $PE_{33}$, is shown in c) on the compression side and d) on the tension side. The areas where the highest values are found are marked out.
The elastic compressive stress, for the pole tested in the field, was estimated by using elementary cases. The elastic compressive stress in the concrete pole was 48.4 MPa, as reported in Section 5.1. When analyzing the compressive stress of the model, when different loads are applied, the
maximum value of the elastic compressive stress is 46.70 MPa, which is read out from the stress-load relation illustrated in Figure 32.

![Figure 32: Compressive stress for different loads applied on the model, added to the prestressing. The maximum value of the elastic stress, pointed out, is 46.70 MPa.]

### 5.2.4 Stresses and strains in the reinforcement

Stresses and strains in the tendons are of interest on the tension side. The maximum values of stresses and strains are obtained at the upper edge of the support. Maximum values for principal stress and plastic strain, for the failure load 102 kN, are reported in Table 23 and illustrated in Figure 33. The values reported for stress and strain in Table 23 are the highest values observed. They are read out by using integration point and centroid, which in this case are the same.

<table>
<thead>
<tr>
<th>Max. principal stresss [MPa]</th>
<th>Max principal plastic strain [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1620</td>
<td>0.0026</td>
</tr>
</tbody>
</table>
a) Max principal stress, on tension side.  b) Max principal plastic strain, on tension side.

Figure 33: Max principal stress and plastic strain in the tendons are found at the edge of the support.

The stress-plastic strain relation in the tendons, for the maximum load of 112 kN, is illustrated in Figure 34, together with the theoretical stress-plastic strain relation used in the material model for steel.

![Stress-Plastic Strain Diagram](image)

Figure 34: The stress-plastic strain relation used in the material model and the result for the FE-model at the maximum load 112 kN.

The maximum strength for steel used in the material model is 1860 MPa, together with the maximum value of the plastic strain, which is 0.029. The highest value for stress, obtained in the model, is 1749 MPa together with the maximum plastic strain 0.0077.
5.3 Optimized FE-models

In this section, the results for Models 2-6, the five models optimized to decrease pole weight, are reported. The geometry for these models is changed in accordance with Table 9 in Section 4.2.1.

5.3.1 Failure load and displacement for Models 2-6

The results for Models 2-6 are reported in Table 24. $D_1$ and $D_2$ are, as before, displacements on the height 18 and 21 m, except for Model 6 where they are the displacements at a height of 11.8 m, where the load is applied, and at 12 m.

<table>
<thead>
<tr>
<th>Model no</th>
<th>Failure load [kN]</th>
<th>Bending moment [kNm]</th>
<th>Crack load [kN]</th>
<th>Crack moment [kNm]</th>
<th>Displacement $D_1$ [mm]</th>
<th>Displacement $D_2$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>93.9</td>
<td>1502</td>
<td>30.6</td>
<td>490</td>
<td>1430</td>
<td>1860</td>
</tr>
<tr>
<td>3</td>
<td>80.8</td>
<td>1295</td>
<td>23.2</td>
<td>371</td>
<td>1272</td>
<td>1648</td>
</tr>
<tr>
<td>4</td>
<td>51.7</td>
<td>827</td>
<td>19.4</td>
<td>310</td>
<td>755</td>
<td>996</td>
</tr>
<tr>
<td>5</td>
<td>44.9</td>
<td>718</td>
<td>9.6</td>
<td>154</td>
<td>1402</td>
<td>1831</td>
</tr>
<tr>
<td>6</td>
<td>52.8</td>
<td>520</td>
<td>9.6</td>
<td>94</td>
<td>0.52</td>
<td>0.53</td>
</tr>
</tbody>
</table>

In Figure 35, the load-displacement relations for Models 2-6 are illustrated together with Model 1. Both loads and displacements are measured at point $D_1$, where the pressure is applied.
The displacement decreased for Models 2-4 but increased for Model 5, which is of a more slender shape. The behavior of Model 6, which is shorter, differs from all the other models.

5.3.2 Stresses and strains in Models 2-6

Maximum stresses and strains in Models 2-6 are compared with Model 1 in two aspects. Firstly, the failure load is determined for the stresses of the same size as for the failure of Model 1. This means compression stresses of sizes between 95-100 MPa or tension stresses around 4.45 MPa, when looking at the centroid probe value. Secondly, the location for maximum stresses and strains are examined to analyze differences when the geometry of the pole changes. Maximum centroid values for the failure load, defined for each model, are reported in Table 25.

Table 25: Axial stress, $S_{33}$, principal stress and plastic strain, $PE$, in Models 2-6.

<table>
<thead>
<tr>
<th>Model. no</th>
<th>Compression side</th>
<th>Tension side</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_{33}$ [MPa]</td>
<td>Prin. stress [MPa]</td>
</tr>
<tr>
<td>2</td>
<td>-97.8</td>
<td>-102.5</td>
</tr>
<tr>
<td>3</td>
<td>-97.3</td>
<td>-95.8</td>
</tr>
<tr>
<td>4</td>
<td>-87.8</td>
<td>-87.7</td>
</tr>
<tr>
<td>5</td>
<td>-98.7</td>
<td>-103.5</td>
</tr>
<tr>
<td>6</td>
<td>-91.2</td>
<td>-95.0</td>
</tr>
</tbody>
</table>

Stresses and strains in Model 2 are acting in the same way as in Model 1. In Model 3, strains on the tension side are lower at failure, just reaching values of 0.0053, to be compared with values around 0.015 for Model 1. The strain distribution over the tension side of the pole is also different, as illustrated in Figure 36.

The same behavior, with lower values of the strain, is valid for Model 4, but in this case compression stresses did not reach 95 MPa before failure of the FE-model. For Model 4, stresses around 90 MPa were reached when the model failed.
Figure 36: Plastic strain, PE33, over the pole for Model 4.

Stresses and strains on the compression and the tension side for Model 4 are reported in Table 26. The highest values of tension stress were found on the side of the pole. The highest values of the plastic strain on the tension side were found on the middle of the pole height, but the highest values for the principal strain were found at the support.

Table 26: Stresses and strains obtained from Model 4.

<table>
<thead>
<tr>
<th>Stress or strain</th>
<th>Compression side</th>
<th>Tension side</th>
</tr>
</thead>
<tbody>
<tr>
<td>S33 [MPa]</td>
<td>-87.8</td>
<td>-116.9</td>
</tr>
<tr>
<td>Principal stress [MPa]</td>
<td>-87.7</td>
<td>-115.4</td>
</tr>
<tr>
<td>PE33 [–]</td>
<td>-0.0013</td>
<td>-0.0018</td>
</tr>
<tr>
<td>Principal strain [–]</td>
<td>-0.0015</td>
<td>-0.0019</td>
</tr>
</tbody>
</table>

For Model 5, the behavior is, again, more like the behavior of Model 1. Stresses, strains and displacement are almost the same for the failure load, but the failure load itself is much lower.

For Model 6, the behavior reminded about Model 4, thus the compressive stresses did not reach values higher than 95 MPa, when looking at centroid values, before failure of the model occurred. Tensile stresses on the other hand were higher, which makes it difficult to determine if there is a compressive or tensile failure of the pole.
6. Analysis

6.1 Field test

The outcome of the calculations for the pole is quite consistent with the field test. The force needed to draw the pole to failure was only about 3% higher than the maximum capacity according to the calculations. However, comparing the result with the requirements of the pole, there is a big enough difference to allow for a geometry change.

The elastic stresses calculated by using elementary cases are higher than the elastic strength according to Eurocode, described in Section 3.1.2. This may imply an effect of prevented transverse expansion, described in Section 3.4.3, which then also has an effect on the plastic strength.

6.1 Model 1

The purpose of Model 1 was to capture the behavior for the pole tested in the field. The concrete damage plasticity model was defined according to CEB, with small adjustments to give a reasonable behavior.

The failure load for Model 1 is 102 kN. When analyzing the centroid values for the failure load, compressive stresses of 99.5 MPa are found at the edge of the support. The stress on the tension side is 4.2 MPa at the same time. The highest value for the compression strength, used in the concrete damage plasticity model, is 71.60 MPa and the highest value for the tension strength is 4.45 MPa. Taking the effect of prevented transverse expansion into account, the stresses obtained are reasonable. The elastic compressive stress, read out from the curve showing stress for different loads in Section 5.2.3, is also very close to the elastic stress estimated from the field test.

When comparing the load-displacement relation for different loads from the field test with Model 1, their behavior is very similar. Therefore, with both elastic stresses and the behavior when it comes to bending agreeing between the FE-model and the field test, Model 1 can be used as a template when creating models with different geometries.

6.2 Type of failure

When studying the field test, both on the test site and by watching the film from it, the failure probably is a compression failure in the concrete with a ductile behavior. However, it is hard to tell if the failure did occur on the compressive side or the tension side, because when the failure load is reach, after a slow procedure when the load was increased and the pole was
bending out, the failure goes very fast and is difficult to catch with the naked eye.

When analyzing the FE-model, the assumption which failure most likely will occur, and did occur on the field, was confirmed. While tension stresses in the concrete did not reach the strength limit, compression stresses did. Furthermore, the steel tendons reach the yield stress before failure. According to theory, see Section 3.1.4, this is the requirement for a ductile compressive failure in the concrete.

When optimizing the geometry, the behavior changes. For most of the optimized models the type of failure will be the same as for Model 1, but in some cases the compressive stresses are lower, while tension stresses increase. When the tensile strength in the concrete is reached, cracks occur and the reinforcement is acting on the tension side. If the concrete reaches its ultimate compressive strength before the failure of the steel occurs, the failure will be the same as for Model 1, meaning compressive failure in the concrete with a ductile behavior. Because of the amount of high strength steel used, this is most likely the case. However, if the steel tears off before the compressive strength in the concrete is reached, the behavior of the failure will be ductile if the steel has reached the yield stress, but brittle if it has not. The second case happens when the amount of reinforcement used is too small. Therefore, it will most likely not be the reason for failure in this case.

Thus, the behavior of the failure for most of the models, will be compressive failure in the concrete with a ductile behavior. This may differ for some geometries where the failure could be a tensile failure in the steel with a ductile behavior. However, the behavior in both cases is ductile.

6.3 Comparison of FE-models and test results

As mentioned in the introduction, Section 1.1, and the theory Section 3.4, RISE carried out bending tests on wood poles. The mean values of the bending moment and the corresponding load are compared with the models created in BRIGADE/Plus to illustrate which dimensions of a concrete pole are the most suitable for the Swedish local grid. Results from the bending tests at RISE, as well as results from the field test conducted in this study, are reported in Table 27, together with the failure load, the corresponding bending moment and the maximum deflection at the top.
Table 27: Failure load, corresponding bending moment and displacement for poles tested in the field and at RISE and all FE-models.

<table>
<thead>
<tr>
<th>Model</th>
<th>Failure load [kN]</th>
<th>Bending moment [kNm]</th>
<th>Displacement [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>RISE</td>
<td>9.2</td>
<td>90.5</td>
<td>2.60</td>
</tr>
<tr>
<td>Field</td>
<td>103</td>
<td>1650</td>
<td>1.95</td>
</tr>
<tr>
<td>Model 1</td>
<td>102</td>
<td>1630</td>
<td>2.00</td>
</tr>
<tr>
<td>Model 2</td>
<td>93.9</td>
<td>1502</td>
<td>1.86</td>
</tr>
<tr>
<td>Model 3</td>
<td>80.9</td>
<td>1295</td>
<td>1.65</td>
</tr>
<tr>
<td>Model 4</td>
<td>51.7</td>
<td>827</td>
<td>1.00</td>
</tr>
<tr>
<td>Model 5</td>
<td>44.9</td>
<td>718</td>
<td>1.83</td>
</tr>
<tr>
<td>Model 6</td>
<td>52.8</td>
<td>520</td>
<td>0.53</td>
</tr>
</tbody>
</table>

The weights for the FE-models are reported in Table 28. The ratio between the weight of Models 2-6 and Model 1 are also reported, to provide a picture of the effect of different geometry changes. In Figure 37 the relation between the weight and displacement is illustrated.

Table 28: Weight of all FE-models and the ratio between the weight of Model 1 and the other models in %.

<table>
<thead>
<tr>
<th>Model</th>
<th>Weight [tonne]</th>
<th>Ratio [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>9.24</td>
<td>100.0</td>
</tr>
<tr>
<td>Model 2</td>
<td>7.20</td>
<td>77.9</td>
</tr>
<tr>
<td>Model 3</td>
<td>5.85</td>
<td>63.2</td>
</tr>
<tr>
<td>Model 4</td>
<td>4.46</td>
<td>48.3</td>
</tr>
<tr>
<td>Model 5</td>
<td>3.44</td>
<td>37.2</td>
</tr>
<tr>
<td>Model 6</td>
<td>2.32</td>
<td>25.1</td>
</tr>
</tbody>
</table>

Figure 37: Weight and displacement for the FE-models. The weight for each model in relation to Model 1 is given in percent.
A comparison between the displacement, when the models are loaded from 0 kN up to 45 kN, are illustrated in Figure 38. The maximum displacement, within this load span, are reported in the figure.

Figure 38: Displacement for all FE-models, loaded from 0 to 45 kN. The maximum displacement for this load span are given in the figure.

6.4 Weight reduction of the pole

Making the pole slenderer, but keeping the same concrete thickness, results in almost the same capacity but a significant weight reduction. However, these geometry changes lead to much higher displacements. The loss of capacity, due to such a geometry change, can be achieved by making the pole shorter. With a decreasing length of the pole, but with all other parameters staying the same, displacements decrease.

Comparing the displacements of the FE-models with the wood poles tested by RISE, no deflection as big as for the wood pole is observed for any of the geometries analyzed.

The magnitude of the possible weight reduction depends on several things, such as which electrical grid the pole should be designed for and the geographical location. The comparison between the capacity for the pole tested in the field and the pole tested by RISE provides a picture of the difference between poles for the local and regional grid.

Comparing the results with the requirements for the actual pole, there is a difference big enough to imply a geometry change, resulting in a 30% reduction of the mass. Looking at the requirements for the local network the
change can be even bigger, resulting in weight reductions of 75% or more, depending on the geometry of the pole.
7. Discussion

7.1 Comparison between the field test and Model 1

The maximum load for the FE-model was higher than the failure load from the field test, which implies that the capacity for the FE-model is overestimated in BRIGADE/Plus. When analyzing the results, compressive stresses in the concrete were around 111 MPa, when looking at the centroid probe value, which is too high to be reasonable. When looking at stresses in the FE-model for the load corresponding to the failure load in the field, the size of the stresses seems more reasonable. Therefore, the stresses obtained at that time were determined to be the maximum allowable stresses and used as references when creating new models with different geometries.

The magnitude of the moment, when the first crack occurred, is smaller for the FE-model than the result of the calculations of the pole tested in the field. Changes in the stress-crack opening model may change that, which makes the material model interesting to examine further. This, because of limitations in time, was not considered in this study.

7.2 Reliability of the results for the optimized models

Because the maximum stresses, leading to failure, were determined for Model 1 when the behavior agreed with the field model, the results for Models 2-6 are reliable. Therefore, the FE-models provide a good picture of the possibility of changing the geometry of a concrete pole in order to fulfill requirements and preferences for use in the electrical network in Sweden.

7.3 The choice of methods

The methods used to create material models was based on the theory of previous works and was, particularly, a combination of Hilleborg’s theories about fracture energy [27], and the material behavior of concrete and steel according to CEB [34] and EN 1992 [20]. Because these methods have been used in several studies, some of them mentioned in the literature review and theory section, where the theory has been compared with experimental results, they were considered credible.

The tensile-crack opening relation, used in the material model, was based on the combination of theories from CEB [34], EN 1992 [20] and more, e.g. the way to calculate the fracture energy according to Kazemi et. al [26]. Using different ways to calculate the stress-crack opening relation, which includes calculations on fracture energy and crack width, according to Section 3.2 in the theory chapter, gave very different results. Finally, the result used for the
material model, which is reported in method Section 4.2.2, is a combination adjusted for the FE-model to create a behavior similar to the test model.
8. Conclusions

The failure most likely to occur is a compressive failure in the concrete. Because the tendons reach their plastic behavior before the failure load is reached, the behavior of the failure is most likely to be ductile.

If the type failure is tensile failure in the tendons, which is not likely, the type of failure is most likely to be ductile because of the amount of reinforcement.

The material model used gave a credible result, with the behavior of the FE-model consistent with the behavior of the pole tested in the field.

The failure load and the maximum displacement decrease with a decreasing thickness of the concrete.

When changing the diameter to a smaller one, the strength of the pole decreases, while the displacements of the pole increase. A reduction of the diameter, without changing the thickness of the concrete, gives a weight reduction around 30% for a height of 21 m.

The capacity loss of the pole when making it more slender can be gained by making it shorter. Hence, the combination of a smaller diameter and a reduced length results in the most effective weight reduction.

Depending on the voltage in the lines and the geographical location of the pole, a weight reduction of 30-75%, or more, is possible. Still fulfilling the requirements, a weight reduction around 30% is possible for the type of poles that was tested. It is possible to reduce the weight with more than 75% for poles to the local grid. This means a mass reduction of 2 and 7 tonnes respectively.
References


