Cracks in onshore wind power foundations

Causes and consequences

Elforsk rapport 11:56

Manouchehr Hassanzadeh

January 2012

Vindforsk Energimyndigheten ELFORSK

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Foreword

Some years ago Thomas Stalin at Vattenfall read an article in Neues Energi about 3000 turbines in Germany with damages in the foundations. In order to investigate how frequent crack are, for which type of foundations cracks have been observed and possible causes, this pre-study was therefore started within the Vindforsk programme.

The work has been carried out by the Manouchehr Hassanzadeh at Vattenfall. Thomas Stalin at Vattenfall has contributed with contacts and material for the study and contributed with reviews and discussions.

Vindforsk-III is funded by ABB, Arise windpower, AQ System, E.ON Elnät, E.ON Vind Sverige, EnergiNorge, Falkenberg Energi, Fortum, Fred. Olsen Renewables, Gothia Vind, Göteborg Energi, HS Kraft, Jämtkraft, Karlstads Energi, Luleå Energi, Mälarenergi, o2 Vindkompaniet, Rabbalshede Kraft, Skellefteå Kraft, Statkraft, Sena Renewable, Svenska kraftnät, Tekniska Verken i Linköping, Triventus, Wallenstam, Varberg Energi, Vattenfall Vindkraft, Vestas Northern Europe, Öresundskraft and the Swedish Energy Agency.

Comments on the work and the final report have been given by a reference group with the following members: Sven-Erik Thor, Vattenfall, Thomas Stalin Vattenfall, Henrik Berglund, Statkraft , Dan Sandros, Stena Renewable AB, Tomi Jokiranta, Fortum, Robert Lundström, Skelleftå Kraft, Rune Rönnholm, Triventus and Anders Björck Elforsk

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Sammanfattning

Det finns många välutformade fundament för vindkraftverk, som inte är spruckna och uppfyller samtliga funktionskrav. Men det finns även fundament som har spruckit på ett sådant sätt att de inte uppfyller kraven med avseende på stabilitet och livslängd. Denna undersökning visar att sprickor i fundament för vindkraftverk är en fråga som måste tas på allvar. Rapporten visar att anslutningen mellan torn och fundament med en ingjuten stålring kan ge upphov till sprickbildning i betongen och bilda ett mellanrum mellan betong och stålringen. Trots att skadorna inte medför några omedelbara konsekvenser vad gäller bärförmåga kan de i det på lång sikt äventyra konstruktionens egenskaper och funktionsduglighet.

Det har påpekats att den huvudsakliga orsaken till de observerade skadorna har varit bristfällig utformning, det vill säga att man använt de konstruktionslösningar som har utvecklats för mindre anläggningar har tillämpats på de större vindkraftverken. Dessutom har skadeutredningar inte utförts på rätt sätt och resultaten inte har utnyttjats för att förbättra konstruktionsutformningen.

Förutom den ovan nämnda skadetypen förekommer två andra typer av sprickbildningar. Den ena typen är sprickor som uppkommer injekteringsbruket mellan anslutningsflänsen och betongfundamentet. Den här typen av sprickor uppkommer på grund av dåligt utförande och fel materialval. Den andra typen av sprickor uppkommer i höga socklar, d.v.s. socklar med en höjd mellan 4 och 8 m. Orsaken till sprickbildningen är bristfällig dimensionering av konstruktionen. Det bör noteras att fundament med höga socklar användes för att öka konstruktionens navhöjd. Den här typen av konstruktioner användes, dock, i en begränsad omfattning. Idag har man nästan övergett den här typen av konstruktion. Konstruktionen är inte vanligt förekommande i Sverige.

förekommer sprickor i injekteringsbruket mellan anslutningsflänsen och betongfundamentet samt betongsockeln. Den första skadan antas ha orsakats av dåligt arbetsutförande medan den andra antas ha orsakats av bristfällig utformning. Båda skadetyperna kan undvikas genom bättre arbetsutförande och utformning.

Baserat på undersökningens resultat rekommenderas att nya projekt initieras med följande innehåll:

- Utveckling av anvisningar för planering och produktion av fundament för vindkraftverk.
- Utveckling av anvisningar för produktion och kvalitetskontroll, inspektion och skadeutredning.
- Utveckling av anvisningar och handbok för reparation av skadade betongfundament.
- Utveckling av anvisningar för dimensionering av detaljutformning av fundament för vindkraftverk.
- Numerisk modellering av det strukturella beteende hos vindkraftverk som inkluderar torn, stiftelser, anslutningar och andra detaljer.

Summary

There are many perfectly designed wind power turbine foundations, which are not cracked and fulfil all functional requirements. However, there are foundations that have been cracked in such a way that they do not fulfil the requirements with regard to service life and structural stability. This investigation shows that cracking of the wind power turbine foundations is an issue which must be taken seriously. The report shows that the connection of the tower to the foundation by means of insert ring may lead to cracking of the concrete and formation of gaps between the concrete and the insert ring. Although the damages have no immediate consequences on the bearing capacity of the structure, they compromise the serviceability and long-term behaviour of the structure.

It has been pointed out that the main reason for the observed damages has been poor structural design, i.e. the solutions which were developed for smaller facilities, sometimes without thinking about possible consequences, have been applied to the larger wind turbines. Furthermore, the site investigations were not conducted properly and the findings were not brought back to the design praxis.

Besides the above-mentioned damage type two other types of cracking has been observed. One type is the cracking of the mortar grout between the connection flange and the concrete foundation. This type of cracking is caused by poor workmanship performance and inappropriate material selection. The other type observed is the cracks in the high pedestals, i.e. pedestals with a height of about 4 to 8 m. The cracks are caused by poor structural design. It should be noted that foundations with high pedestals were used in order to gain higher hub heights. This type of design was, however, not commonly used in the past and it is almost abandoned at the present time. This design has not commonly been used in Sweden.

Based on the results of the investigation it is recommended that new projects be initiated with following contents:

- Development of guidelines for planning and production of wind power turbine foundations.
- Development of guidelines for production and quality control, inspection and damage assessment.
- Development of guidelines and handbook for repair of damaged foundations.
- Development of guidelines for design and detailing of the foundations for the wind power turbines.
- Numerical modelling of the structural behaviour of the wind power turbines, towers, foundations, connections and other details.

Content

1 Introduction

1.1 Background

Foundations of the wind power turbines onshore are made of reinforced concrete. The reinforcement is non-stressed and is designed to carry the tensile stresses, and to distribute and to limit the size of the cracks in the concrete. The non-stressed reinforcement starts to function efficiently when concrete is cracked, i.e. cracks in reinforced concrete structures are not avoidable. Although cracks can’t be avoided the structures must be designed in such a way that the cracks become evenly distributed and gain limited length, depth and width. Otherwise, the service life, and in some cases the stability and load bearing capacity, of the structures may be compromised.

Causes of the cracks in the wind power foundations onshore may have different origins such as poor material and structural design, poor execution of work, faulty detailing and lack of knowledge regarding loading and boundary conditions. The above-mentioned causes and faults are manifested as thermal and plastic shrinkage cracks during the production phase or in early age of the structure; drying shrinkage and other environmentally induced cracks later on; structural cracks caused by static and dynamic overloading, stress concentration around the details and creep. For instance, the dynamic loading may induce cracks, gaps, between the anchoring details cast in the concrete and the concrete. The water which penetrates in the cracks may cause leaching and/or frost-damage. In other cases the water may transport loose particles, such as fractured aggregates and cement paste, out of the crack or promote packing of the deeply located particles. The latter phenomenon may cause stress concentrations which may lead to widening of the cracks and may trigger off crack propagation.

At the present time there are many perfectly designed wind power turbine foundations, which are not cracked and fulfil all functional requirements. However, there are foundations that have been cracked in such a way that they do not fulfil the requirements with regard to service life and structural stability. It is important to investigate whether the cracking of the foundations is a rare phenomenon or not. If not what are the causes, in which way they may influence the structural behaviour of the wind power turbine construction and how the cracks can be prevented. Furthermore, how a cracked foundation can be repaired. Lack of knowledge in above-mentioned issues may have large economical consequences.

Wind power turbine foundations onshore engage several competencies such as structural design, material design, production, etc. Therefore, this project involves participants with different competencies. Furthermore, in order to ensure the steering of the project, to facilitate the engagement of the appropriate competencies and to save economical resources the project is subdivided in two phases, namely preliminary study (Phase 1) and detailed study (Phase 2).

The preliminary study (Phase 1) constitutes a limited part of the project and aims to show whether the detailed study (Phase 2) is necessary or not, and to prepare a project proposal if the Phase 2 is necessary.

1.2 Aims

The final aims of the project, both phases included, are to:

- document the failure process of cracking wind turbine foundations from examples in Germany and Denmark,
- determine types and causes of the cracks,
- develop guidelines for inspection, assessment and analyses in order to determine the causes of the cracks and
- develop guidelines for design and detailing of the foundations for the land based wind power turbines in Sweden in order to avoid cracks.

In order to achieve the final aims the project is planed to be conducted in two phases. The aims of the Phase 1 are given below, while the aims of the Phase 2 are described in general terms in section 6.2 of this report.

The aims of the Phase 1 are as followed:

- State-of-the-art report which includes type of foundations used for different types of towers and ground conditions, types of connections between towers and foundations used for different types of tower solutions, loading conditions, design basis, etc.
- Site visit and documentation of the observed cracks and the structural information.
- Preliminary analysis in order to describe the cause of the cracks
- Suggestions for a detailed study.

Only phase 1 is considered in this report.

1.3 Limitations

This report is based on information provided by a number of Swedish owners of wind power turbines and contractors, a German consultant and repair contactor. Furthermore a number of journals, books and internet sites have been used to complete this report. This report has no claim to have covered all types of problems in all countries regarding cracks in foundations of wind power turbines.

The author has received many valuable detailed inputs from individuals and companies regarding production techniques, repair methods, design methods, etc. Since, these subjects are somehow out of the range of the current phase of the investigation the inputs are not reflected or treated in this report.

1.4 Execution

This project has been conducted under the direction of a steering committee consisting of representatives from:

- Fortum AB
- Skelleftå kraft AB
- Statkraft AB
- Stena Renewable AB
- Triventus AB
- Vattenfall AB

with help of contractors Mobjer Entreprenad AB, SOLIDO (-Steinfurt.de) and Prof. Bellmer Ingenieurgruppe GMBH.

The author of this report visited Mobjer on February 28th, 2011; Prof Bellmer and Solido on March 28th and 29th, 2011.

2 Wind power onshore

2.1 Wind power structures

Figure 1 schematically shows the structure of an onshore wind power turbine. A wind power turbine is generically specified by the following technical specifications:

- Hub height i.e. the distance between the hub and the ground, H_{hub}, [m]
- The rotor's diameter, D_{rot}, [m]
- The total height, H_{tot}, [m] which is the sum of the hub height and the half of the rotor diameter
- Wind speed [m/s]
- Rotational speed of the rotor [r/min]
- Power [MW]

The three last items on the list are not single parameters, but they include a set of parameters which define conditions such as starting, operating and shutting down of the wind turbine. Below some of the parameters as defined by Germanischer Lloyd, [1], are presented.

Wind speeds are defined for four different conditions:

1. The "*cut-in wind speed*" is the lowest mean wind speed at hub height at which the wind turbine starts to produce power. Typical speeds 3 - 5 m/s.
2. The "*rated wind speed*" is the lowest mean wind speed at hub height at which the wind turbine produces the rated power. Typical speeds 10 - 14 m/s.
3. The "*cut-off wind speed*" is defined as the maximum wind speed at hub height at which the wind turbine must be shut down. Typical speeds over 25 m/s.
4. The "*short-term cut-out wind speed*" is the instantaneous wind speed at hub height above which the wind turbine must be shut down immediately.

The rotational speeds are defined for six different conditions. Three rotational speeds are defined within the operating range, where as the remaining three concerns with the shutting down conditions. Three of definitions are given below:

1. The "*rated speed*" is the rotational speed at the rated wind speed.
2. The "*set value of the speed controller*" is used for variable-speed plants in the operating state above the rated wind speed. In this operating state, the rotational speed will derivate upwards or downwards

from the set value only by the standard tolerance. Typical speed 20 – 35 rpm.

3. The "*cut-out speed*" is the rotational speed at which an immediate shutdown of the wind turbine must be effected by the control system.

Three different definitions are given for the power where as only the "*rated power*" concerns with the operating conditions. The "*rated power*" is defined as the maximum continuous electrical power at the output terminals of the wind turbine.

The load bearing structures of a wind power turbine may coarsely divided into six groups namely: rotor blades, machinery structures, nacelle covers and spinners, bolted connections, tower and foundation, Figure 1. It should be noted that a wind turbine can be divided into finer groups such as the structures which connect the foundation to the tower and the structures which connect the concrete part of a hybrid tower to the conical steel tube part of the tower, so called adapter. Since, this report deals only with the cracking of the foundations of the onshore wind power turbines, the presented subdivision is adequate for scope the report.

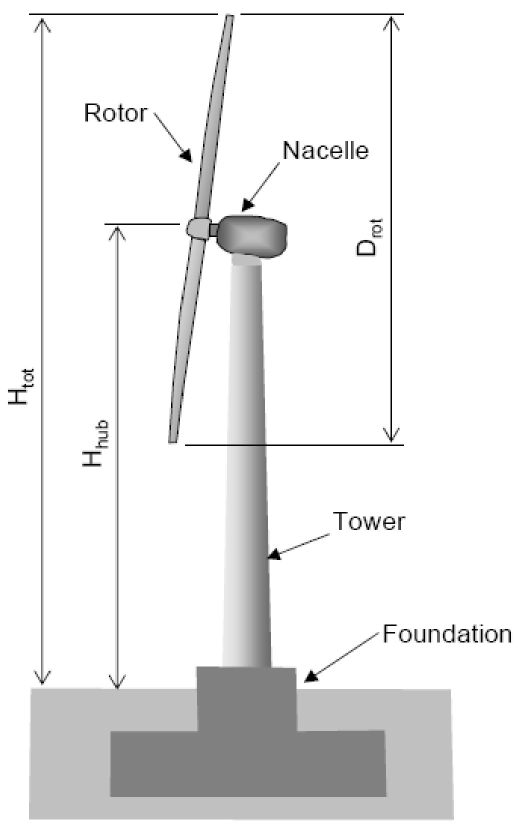

Figure 1 - Structures of a wind power turbine.

2.2 Wind energy, loads and load effects

The wind's kinetic energy is calculated by means of the following equation, [2]:

$$P_{kin} = \frac{1}{2}\rho A v^3 \tag{1}$$

where, P_{kin} [W] is the kinetic energy, ρ [kg/m^3] is the density of the air, A [m^2] is the area of the cross section, plane, perpendicular to the direction of the air flow and v [m/s] is the velocity of the air, wind speed, passing the cross section.

The kinetic energy of the air is converted to the electricity by the rotors and the generator of the wind power turbine. As a result the air flow is slowed down and loads are introduced to the structure. The decelerating of the air flow starts at a distance approximately one rotor diameter in front of the turbine and is completed at the same distance behind the turbine. The amount of energy which is absorbed by the structure is governed by the degree of the deceleration, the rotor diameter, rotor structure and the rotational speed.

In the most efficient case the speed of the wind is reduced at the rotor to 2/3 of the speed corresponding to the undisturbed conditions, and to 1/3 after the rotor, [2]. This deceleration corresponds to 59 % usage of the wind power which is theoretically most optimal utilization of the wind power. However, it should be noted that because of losses such as frictions, vibrations, etc all energy can't be converted to the electricity, i.e. the obtained electric power is less than 59 % of the wind power.

The loads which are imposed on the structure when the turbine is producing power are also governed by the degree of the deceleration, the rotor diameter, rotor structure and the rotational speed. The imposed load is determined by the operational properties of the turbine.

The oscillation of the wind may also cause swinging of the structure which increases the load effect. Furthermore, the variation/distribution of the wind velocity in the vertical direction and turbulences my impose torque on the wind power turbine. As mentioned above even these load effects are governed by the properties of the turbine.

Although determination of the loads and load effects are not within the scope of this report, some issues will still be highlighted by means of the guidelines presented in [1].

A wind turbine is designed for so called external conditions. External conditions are dependent on the intended site or site type for a wind turbine installation. The most important design parameters are wind speed and the turbulence. Therefore the wind turbines are subdivided into wind turbine classes, which are defined in terms of wind speed, V_{ref} and V_{ave}, and turbulence categories *A* (category for higher turbulence intensity value) and *B* (category for lower turbulence intensity value). In addition to the wind turbine classes other external conditions such as normal and extreme temperature ranges, humidity, air density, solar radiation, etc are defined.

According to [1] a plant shall be designed with regard to a reference wind speed V_{ref}, which is specified for a certain wind turbine class. A designed turbine shall withstand the environmental conditions in which the 10-min mean of the extreme wind speed with a recurrence period of 50 years at hub height is equal to or less than V_{ref}. Five turbine classes are defined, namely classes I – IV and S. The V_{ref} [m/s] for classes I – IV are 50, 42.5, 37.5 and 30. The wind speed for class S is specified by the manufacturer.

The mean wind speed V_{ave} is the annual average of the wind speed over many years. The V_{ave} corresponding to classes I – IV are 10, 8.5, 7.5 and 6.

The loads are calculated for design load cases, which are defined by ***Design situation***, ***Wind conditions***, ***Other conditions***, **Type of analysis** (ultimate limit state and fatigue) and ***Partial safety factor*** (Abnormal, normal and extreme).

34 design load cases are defined which are grouped in 8 principal groups as followed:

1. Power production
2. Power production plus occurrence of fault
3. Start-up
4. Normal shut-down
5. Emergency shut-down
6. Parked (standstill or idling)
7. Parked plus fault conditions
8. Transport, erection, maintenance and repair

The analysis is carried out for ultimate limit state and fatigue.

The resulting load effect on the each load bearing structural parts of a wind power turbine is governed by the properties/design of the rotor, the machinery, the tower and the ground structure. Especially, the properties/design of the rotor and the generator are decisive which the manufacturer has thorough knowledge about. Therefore, the loads imposed on the structure are normally delivered by the manufacturer. The loads imposed on the foundations are also normally delivered by the manufacturer.

2.3 Wind power turbine tower structures

The development of larger turbines and the development of higher towers to be able to capture the higher winds at larger heights has led to the development of different tower concepts as described in [3].

In the following some of the tower concepts are described.

2.3.1 Tubular steel tower

Most of the wind power towers are tubular steel structures. The reasons are: 1) the tubular steel structure is relatively light and due to its circular cross-section has the same bending stiffness in all direction; 2) it has god torsional stiffness; 3) the required natural frequency can easily be achieved for certain types of turbines and hub heights; 4) it is relatively easy to install and has low maintenance costs.

Development of turbines with higher maximum power, increased hub height and increased steel price has made the steel tower less economical. Increased hub height decreases the natural frequency of the structure. Furthermore, increased wind power, i.e. turbine power, increases the loads, bending and torsional moments acting on the structure. In order to withstand the increased loadings the dimensions of the tower must be increased, i.e. both diameter of the tube and the thickness of the plate, tube wall, must be increased, which lead to further implications.

One implication is the transportation, i.e. a diameter of 4 to 4.5 meter is usually the upper limit that can be transported to the most locations in land. The other implication is the weight of the segments. The weight of a 12 m long segment with 6 m diameter made of 30 mm plate is 53 tons, which can be difficult to transport in mountainous areas.

Apart from the difficulties associated with the transportation there are also production problems. Rolling of the plates thicker than 50 mm is difficult and the welding becomes labour-intensive. Tubular steel towers have been manufactured with hub heights up to 120 m.

Figure 2 – Tubular steel tower on reinforced concrete pedestal.

2.3.2 Lattice towers

A lattice structure is made of struts which are assembled in a specific order to obtain a prescribed structural strength and stiffness with as little material as possible. In comparison with other tower concepts the lattice towers are less material consuming. The steel is used most effectively and the weight of the tower is less than the other comparable concepts. Quite tall towers can be built by means of lattice system. Since, the towers can be assembled on site no transport difficulties will be encountered. As far as the traditional hub heights are concerned the lattice towers are the cheapest tower solution, provided that the maintenance costs are disregarded.

Maintenance is one of the major drawbacks of the lattice system. There are many joint bolts in a lattice tower, 10,000 in some case. Each bolt must be inspected three times during its service life, which is 20 years. The additional drawbacks of the lattice towers are associated with the towers stiffness and natural frequency. Although the problems can be remedied by increasing the number and the thickness of the elements, profiles, etc the problem can't be solved without increasing the size of the tower's cross-section. However, increased cross-sectional area leads to wider area in the base of the tower, i.e. wider foundation is needed. Furthermore, there are limits for the size of the cross-section at the higher levels. At the higher levels the position of the rotor is a decisive factor, i.e. the distance between the rotor and the tower must be sufficiently large in order to avoid a collision between the rotor and the tower.

Figure 3 - Lattice tower.

Figure 4 - A tubular and a lattice steel tower side by side.

2.3.3 Concrete tower

Pure concrete towers have been used for wind power plants in some cases. The number of towers made of concrete is much lower than the number of tubular steel tower. However, development of new efficient production techniques, increased number of sites, increased number of turbines per site and increased steel price has led to increased number towers made of concrete. For instance cast-in-place by slipform and prefabricated elements has frequently been used to build towers.

All concrete towers are made of reinforced concrete. Functionally two types of reinforcements are used, namely non-stressed (ordinary cast in reinforcement) and post-tensioned reinforcement. The first one is used to provide bearing capacity during production and erection of the tower while the latter is used to place the concrete cross-section in a state of compression before application of service loads. The advantages of the concrete towers are their stiffness, robustness and maintenance properties. A properly designed and produced concrete tower does not need any maintenance during the designed service life. Furthermore, concrete towers in a distant location may be easier and cheaper to produce than the steel towers.

Figure 5 – Towers made of prestressed cast-in-place and slipform cast concrete.

Figure 6 – Production of prestressed cast-in-place and slipform cast concrete.

2.3.4 Hybrid concrete and tubular steel towers

Large turbines and high hub heights require tower structures with high strength, stiffness and natural frequency. High base diameter and wall thickness are demanded in order to fulfil the requirements. It has been shown that it is possible to meet above-mentioned requirements by means of concrete towers either alone or as a part of a hybrid structure.

A hybrid concrete and steel tower consists of concrete sections in the lower part and tubular steel sections on the upper part. For a given hub height the dimension - i.e. diameter, wall thickness and total length of the sections – of the concrete and the steel parts depend on many different factors, such as turbine capacity, wind conditions, logistic, etc. Different manufacturer provide different solution. Approximately 2/3 of a hybrid tower, with hub height 120 - 140 m, may be made of concrete and the remaining part made of tubular steel.

The structural concepts and the production techniques which are used for the concrete part of the hybrid concrete and steel towers are with a few exceptions similar to those of the concrete towers.

Figure 7 – Hybrid concrete and tubular steel tower.

3 Foundations for wind power on-shore

3.1 Principal types of foundations

The type and the size of the foundations used for the wind power turbines are governed by the geotechnical conditions of the site, the maximum power of the turbine and the type of the tower.

When the ground consists of soil with sufficient bearing capacity the loads from the wind power turbine are transferred to the ground by spread footing, i.e. slab foundation. The spread footings rely on soil bearing, and the weight of the foundation itself and the soil backfill on top of the foundation to resist tilting under wind loads.

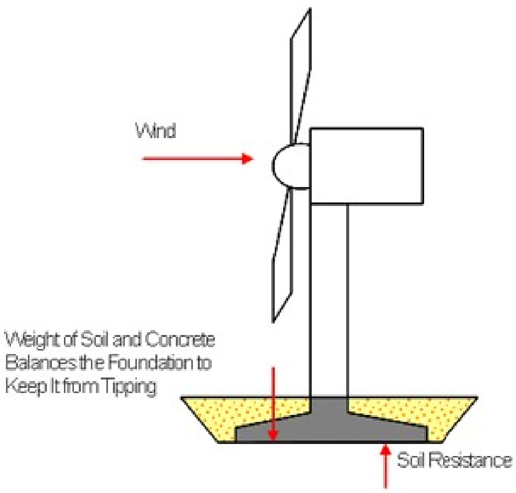

Figure 8 – Spread footing, from [4].

If the bearing capacity of the soil is not sufficient piling is required in order to increase the load bearing capacity of the foundation. The function of the piles are to support the footing and to transfer the loads to a more rigid ground, for instance to the bedrock. The tilting resistance of the foundation comes from the tension and compression resistance of the piles.

Spread footings on the bedrock can be anchored to the bedrock in order to eliminate the need for soil cover and take advantage of high strength rock at the surface.

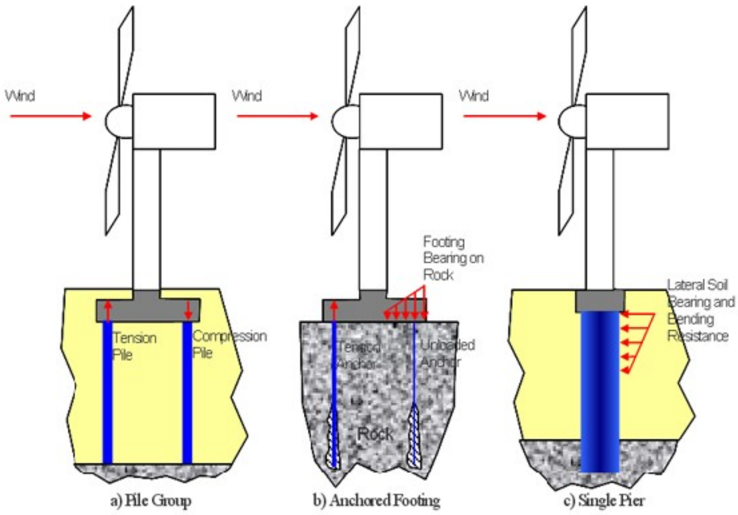

Figure 9 – Different foundations, a) spread footing supported by piles, b) spread footing anchored to the bedrock, c) Footing supported by a single, from [4].

Besides the lattice towers all towers are erected on a single footing. Since, the base of a lattice tower can be wide it is not structurally and economically efficient to erect the tower on a single footing. In practice each legs of a lattice tower is founded on a separate footing.

Recently many wind power turbines have been founded on the bedrock, Figure 9b. Although the number of the turbines founded on the bedrock is increasing still the major part of the Swedish wind power turbines have been founded on the spread footings on grounds/soils with sufficient bearing capacity. The number of the footings which are supported by the piles are not known for the author, but it is supposed that their number is limited.

3.2 Principles for foundations design

As it was mentioned in section 2.2 the loads imposed on the foundations are normally delivered by the manufacturer. Figure 10 shows the coordinate system of the tower bottom at the intersection of the tower axis and the upper edge of the foundation. The section forces which are considered in design are bending moments M_{XF} and M_{YF}, torsional moment M_{ZF}, horizontal forces F_{XF} and F_{YF}, and the vertical force F_{ZF}.

The structure of the foundation and the ground which supports the foundation is designed according to the national, or corresponding international, design codes.

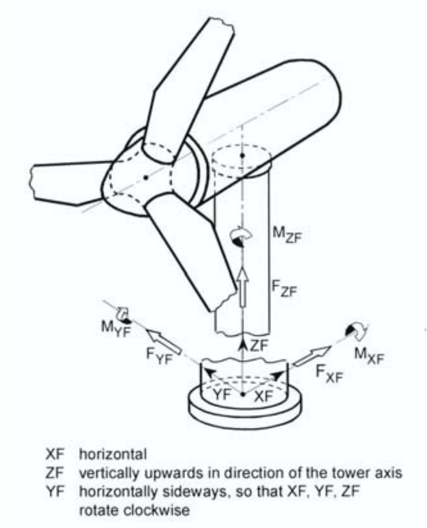

Figure 10 – Tower bottom coordinate system, from [1].

3.3 Structure of the foundation

3.3.1 General

As it is presented in section 2.3 there are different types of towers. The type of the tower may influence the design and the details of the foundation. For instance the transfer of the section forces from the tower to the foundation, i.e. the connection between the tower and the foundation, may influence the design and the shape of the foundation. There are great differences between the connection of a prestressed concrete tower to a concrete foundation and the connection of a steel tower to a concrete foundation. Since, this report deals with the damages observed in foundations for wind power turbines containing steel towers, only foundations supporting steel towers will be considered in the following parts.

Figure 11 shows a principle drawing of a spread footing foundation, which is used for 2 – 3 MW turbines. Nine major details are distinguished in the figure:

1. Soil
2. Concrete layer, representative concrete grade C25/30, thickness ~ 100 mm.
3. Spread footing, slab foundation, made of concrete (representative grade = C35/45) – representative values for L_1, L_2 and L_3 are 18 m, 1,6 m and 1,8 m respectively.
4. Pedestal made of concrete (representative grade = C35/45 or higher) - representative values for L_4 and L_5 are 0,6 m and 5,5 m respectively. This type of the pedestal should not be confused with that presented in section 4.3. The high pedestal, 4 to 8 m high, which is presented in section 4.3 is used in order to gain a significant increase in the hub

height. This type of design was, however, not commonly used in the past and it is almost abandoned at the present time. This design has not commonly been used in Sweden.

5. Top layer reinforcement, two layers of reinforcement in both X and Y directions, representative material Ø 25 mm B500B, Figure 18 and Figure 19.
6. Bottom layer reinforcement, two layers of reinforcement in both X and Y directions, representative material Ø 25 mm B500B, Figure 18 and Figure 19.
7. Shear reinforcement, Ø 25, Figure 18 and Figure 19 .
8. Steel adapter for connection of steel tower - representative value for L_6 is 4,8 m.
9. Thickening of footing for placement of flange for stud bolts. There also footings without this type thickening. In those footings the flange is placed above the bottom layer reinforcement, Figure 19 .

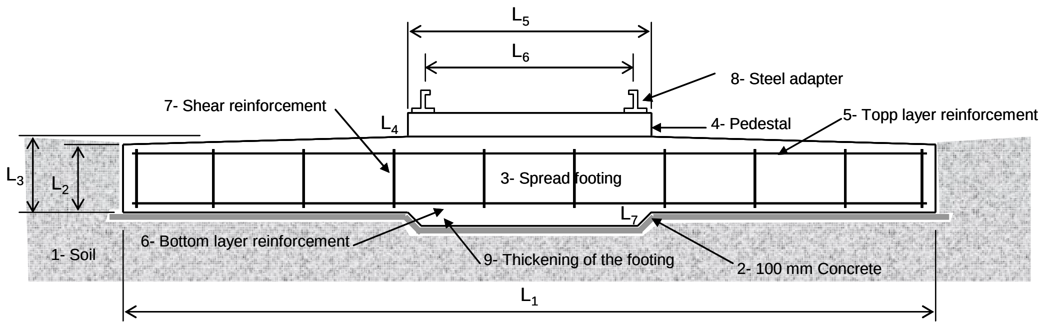

Figure 11 – Principle drawing of a spread footing foundation, vertical cross section.

The spread footings are made square, circular or octagonal. In square footings the bending reinforcements are placed in X and Y directions while in circular and octagonal footings the reinforcement are placed in radial direction.

One of the important parts of the wind power turbine is the connection of the tubular steel tower to the foundation. There are two different principal arrangements to connect the tower to the foundation, namely *insert ring* cast in the foundation and *steel adapter* fixed by stud bolts,

Figure 12. In this context should be noted that principally the same connection methods are applied for footings founded on the bedrocks.

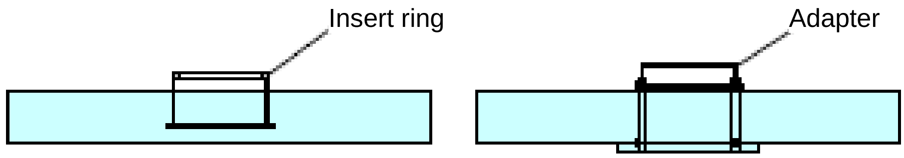

Figure 12 – Different types of arrangements for connection of the tower to the foundation, [5].

3.3.2 Insert rings

In the beginning of the wind power era a great number of the towers were connected to the foundation by means of the insert ring. Because the insert ring was easy to put in place and to adjust Since, at that time the maximum power and the height of the towers were small the loads which were transferred to the foundation did not introduced stresses in such a magnitude to damage the foundation structure. However, development of larger wind power turbines has led to higher stresses in connection devices and the concrete structure of the foundation which have damaged the structure. In order to avoid the damages new types of connections and detailing have been developed. The author has no access to the design of the different foundations with insert rings, which have been used since the early ages of the wind power turbines. Neither has the author access to the designs which have damaged to the foundation result. Therefore, the author will not try to describe the development of the design of the foundations with the insert rings and to point out the designs which have damaged the foundations. However, a few designs which have led to damaged foundations are described in section 4.4.

The design of the foundations with insert rings has been improved since the short comings of the original design were revealed. At the present there are different designs for foundations with insert rings. Although it is out of the range of this report to present different designs, some designs will be presented in order to illuminate the subject and show different detailing.

Figure 13 shows an insert ring embedded in concrete foundation and the detailing around it. As can be observed there is a flange in the upper part of the ring where the tower will be mounted. There is another flange in the lower part of the ring for fixation of the ring and to transfer the tensile and compressive forces from the tower to the foundation. It should be noted that the author is not aware of any damages concerning the foundations build according to the design shown in Figure 13.

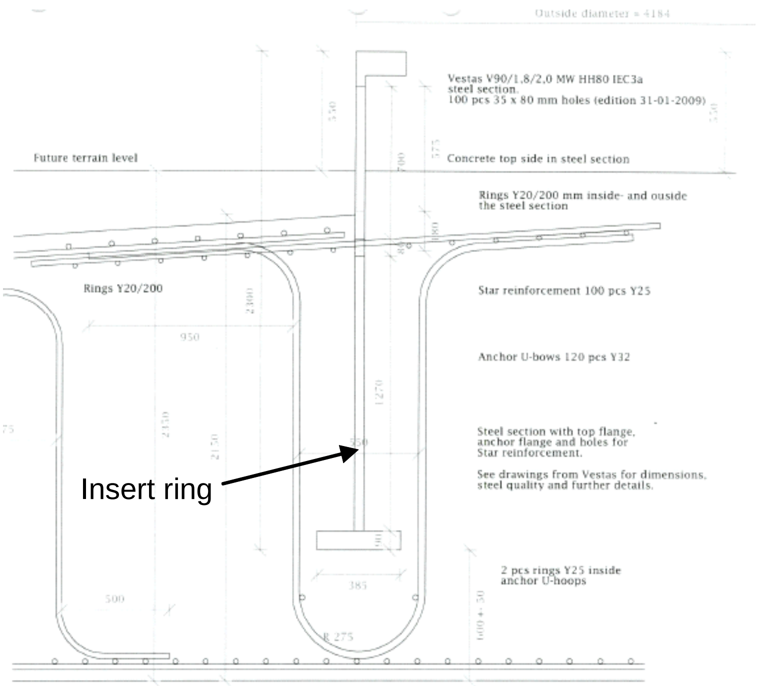

Figure 13 – Insert ring embedded in concrete foundation, [6]. The original drawing is shown in Appendix 1.

Between the flanges the ring contains holes through which the reinforcement bars pass, Figure 14. Please not that the insert ring in the Figure 13 doesn't refer to the ring shown in the Figure 14. According to the design requirements the reinforcement bars shall not be in contact with the ring. Therefore the part of the reinforcement bar which passes through the hole is covered by sealant or other type of material to prevent connection/contact between the bar and the ring.

As mentioned above, connection by insert ring occurs in different designs. The insert ring shown in Figure 13 differs from the designs from the early age of the wind power turbines. In the design shown in Figure 13 the concrete is protected against cracking by means of the U-hoops, reinforcement rings inside and outside of the steel section and the reinforcement rings inside anchor U-hops.

Figure 15 shows another design in which the cracking of the concrete is prevented by means of prestressing arranged by the stud bolts mounted around the ring.

Figure 14 – Insert ring with holes through which the reinforcement bars pass.

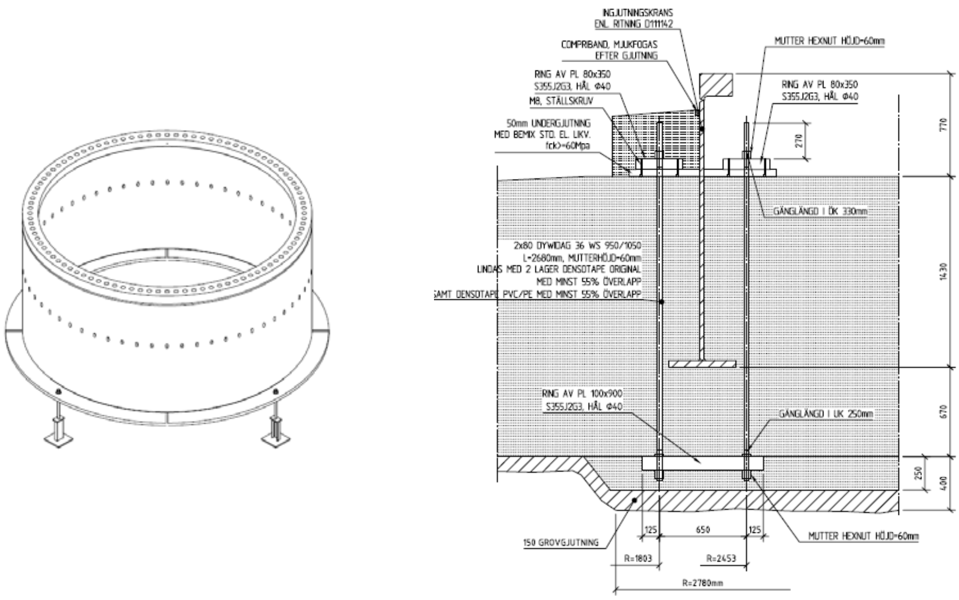

Figure 15 – Insert ring (left) and detailing around insert ring in concrete foundation (right), [8]. The original drawing is shown in Appendix 2.

Repair of concrete around insert ring. The reinforcements, which pass through the insert ring shall be not in contact with the insert ring. The reinforcement is isolated from the ring by sealant.

Figure 16 – Repair of concrete around insert ring, [9].

3.3.3 Connection by means of adapter

Figure 17 and Figure 18 show another type of connection arrangement. As can be observed the connection consists of an adapter which is fixed to the foundation by means of the stud bolts. The stud bolts are in turn fixed in the structure by means of a flange in the bottom of the foundation. As mentioned above in some cases the bottom flange is placed under the reinforcement and in some other over the reinforcement, Figure 19. The stud bolts are not in contact with concrete. The studs are covered when cast in concrete.

Figure 20 shows another design regarding connection by means of anchor bolts. In this design the bottom flange is not placed in the bottom of the foundation but in the upper part, i.e. within the pedestal.

In some designs the adapter is replaced by a load spreading plate. The approximate dimensions of the plate are thickness = 75 mm, width = 700 mm, outer ring diameter = 5000 mm. The plate is placed approximately 70 mm above the planed concrete surface before concrete is cast, Figure 21 and Figure 22. When the concrete is hardened the plate is levelled and fixed and the gap between plate and the hardened concrete is grouted by a high strength mortar. The tower is mounted by placing the tower flange on the load spreading plate and anchoring the flange to the foundation by the cast in stud bolts.

Appendix 5 shows an example of a foundation on bedrock.

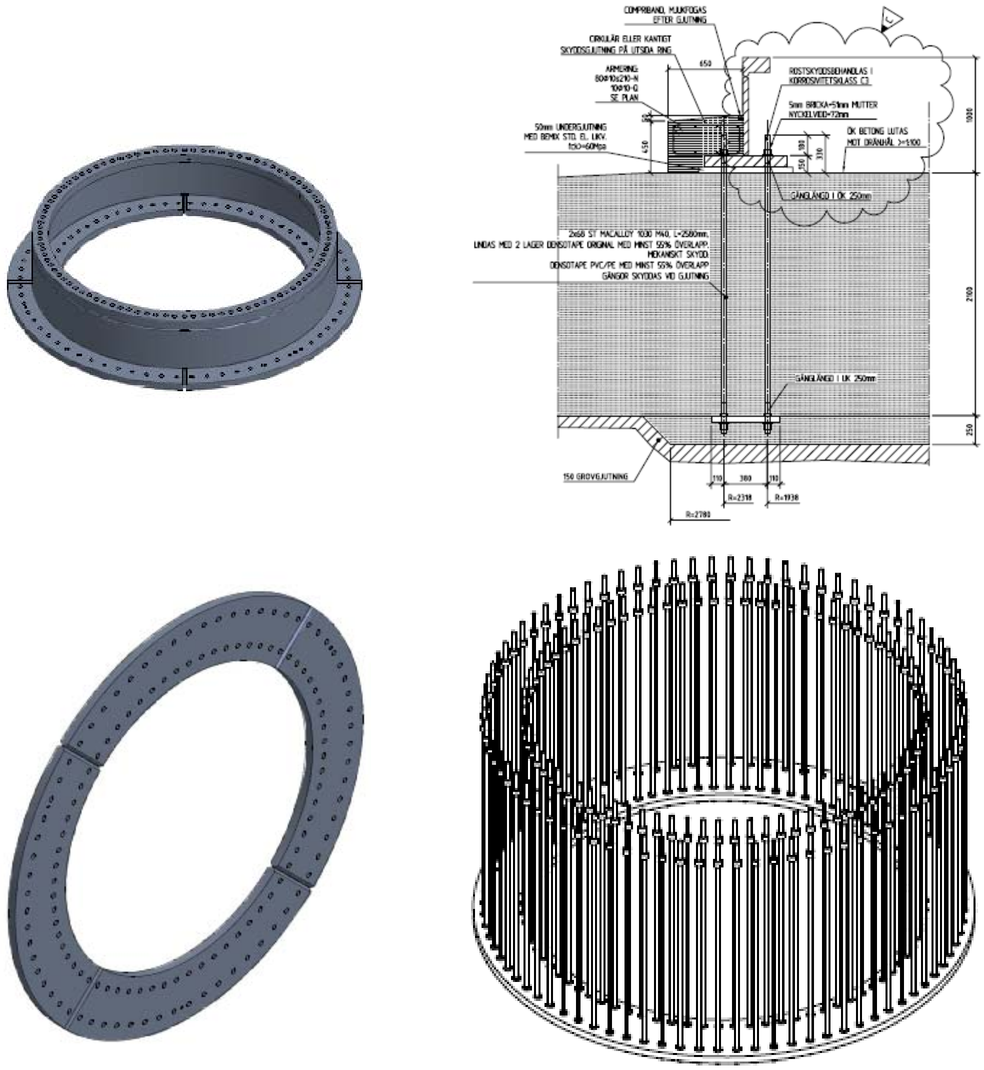

Figure 17 – Stud bolt foundation steel adapter (upper left), detailing around insert ring in concrete foundation (upper right), fixing flange and stud bolt, [8]. The original drawing is shown in Appendix 3.

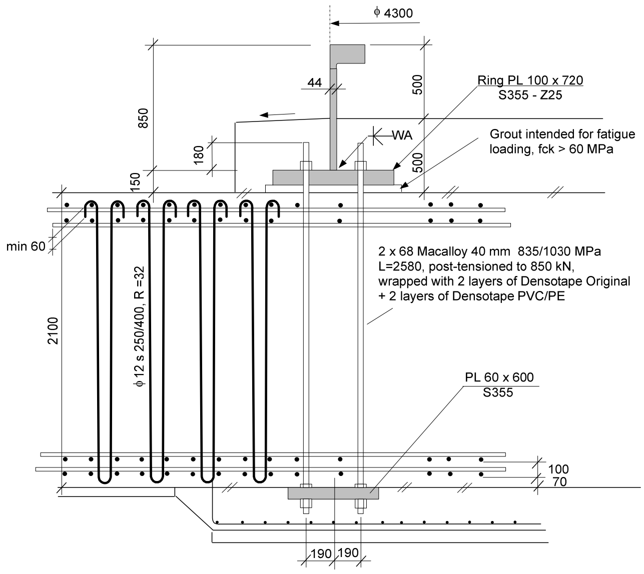

Figure 18 – Adapter and prestressed anchor bolts, [8].

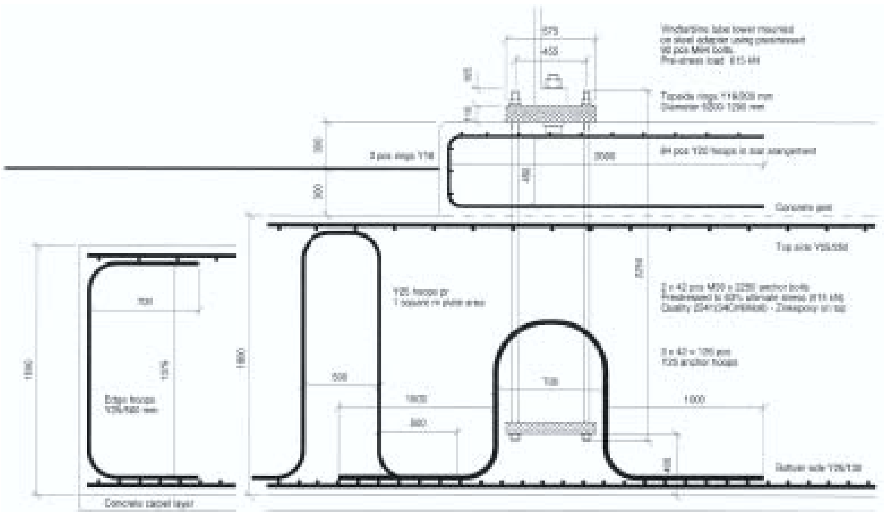

Figure 19 – Steel adapter, anchor bolts with lower flange placed over the re-inforcement, [10].

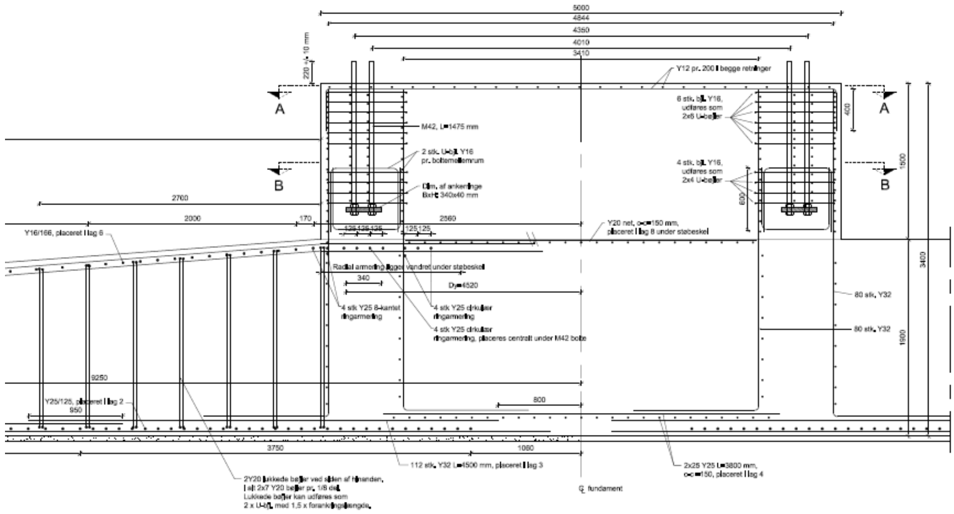

Figure 20 – Connection by anchor bolts with a lower flange in the pedestal, [11]. The original drawing is shown in Appendix 4.

Figure 21 – Flange on the top of the pedestal before casting.

Figure 22 – Flange on the top of the pedestal after casting.

3.4 Construction

The construction of the foundation is briefly described below. It should be noted that the sequential description below is generic and applies for both foundations containing insert ring and foundations containing stud bolts. The principal differences are however pointed out. Furthermore, it should be noted that the details presented in the following figures are not associated with the designs presented in sections 3.3.23.3.3 and 3.3.3.

After excavation the bottom concrete layer is cast. After that the bottom layer and edge reinforcement is placed. The insert ring or the stud bolts are installed before upper layer reinforcement and shear reinforcement are mounted, Figure 23. When all installations are down the concrete, is poured into the form, Figure 24.

Foundation surface is sloped to drain the water from the surface. The slope is achieved by pouring more concrete in the central part of the foundation and then smooth down the surface towards the edges, Figure 25.

1- Bottom concrete layer

2- Bottom reinforcement

3- Bottom and edge reinforcement

4- Insert ring

5- Stud bolts and bottom ring

6- Stud bolts and upper ring

Figure 23 – Different production steps, 1-4 [12] and 5-6 [10].

Figure 24 – Pouring of concrete in a prepared spread footing formwork, [13].

Figure 25 – Trowelling of the surface and covering the surface by plastic foil to prevent drying during curing, [13].

There are great stresses transmitted from the tower to the foundation through a small surface of the flange of the steel adapter or load spreading plate. Therefore, a high flatness of the concrete surface where the adapter flange or load spreading plate is in contact with the concrete is required. An uneven and non-flat surface results in large stress concentrations, which may crush the concrete.

There are two ways to achieve a flat contact between the concrete and the flange or the plate. The first way is that the adapter is mounted a bit above the concrete surface and then the gap between the flange and the concrete is injected with a high strength mortar grout, see Figure 32. The other way is to press the adapter or the plate into the concrete while the concrete is fresh. In this way produces a footprint in the concrete which provides a flat contact between the concrete and the flange.

4 Cracks and damages in foundations for onshore wind power turbines

4.1 General

A comprehensive family tree of crack types is shown in Figure 26, [14]. All types of cracks presented in the figure are possible to occur in wind power turbine foundations as well as any other structure made of concrete. However, most of the cracks can be avoided by means of careful design, choice of material, workmanship, construction, etc.

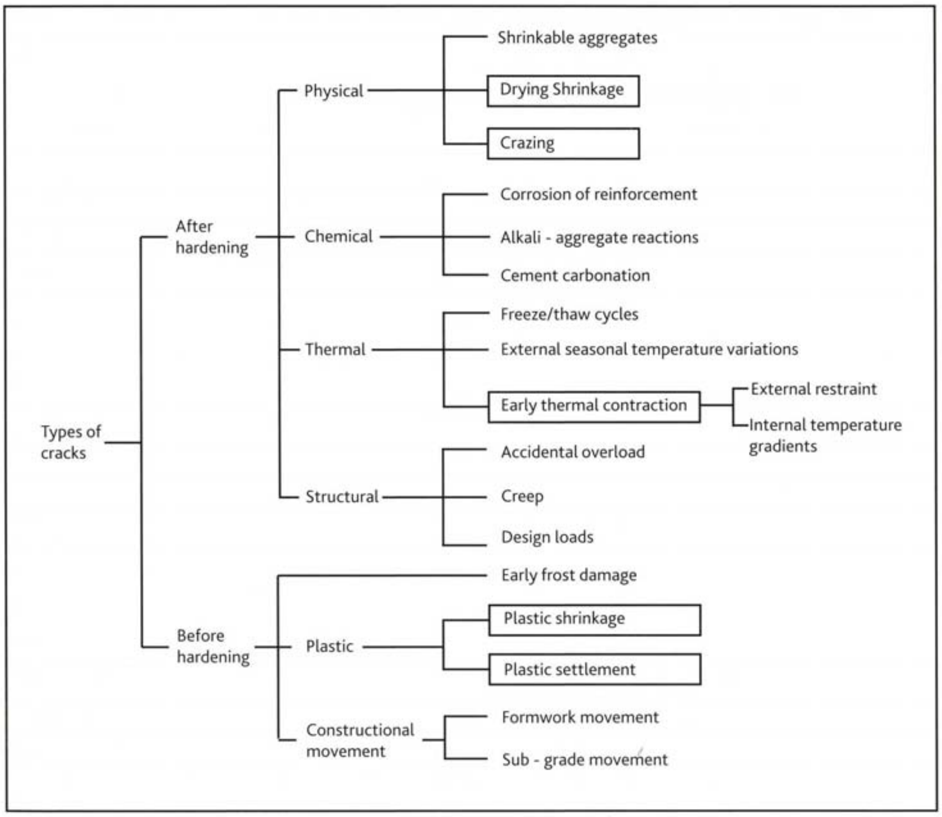

Figure 26 – Types of cracks, [14].

Different types of cracks in concrete foundations for wind power turbines have been reported. Most of the damages that the author has knowledge of have been caused by design faults, please refer to [15], [16], [17], [18], [19] and [20]. Other non-structural cracks are according to [15] caused by:

- Insufficient concrete cover
- Improperly installed reinforcement
- Inadequate concrete curing
- Concreting at low temperatures
- Concrete mix is not correct, water is mixed at the construction site
- Casting joint show cracks or defects

Some cases which have been addressed in [15] will be described next.

4.2 Mortar grout between the steel flange and the foundation

The steel tower is connected to the foundation by means of pre-stalled anchor bolts.

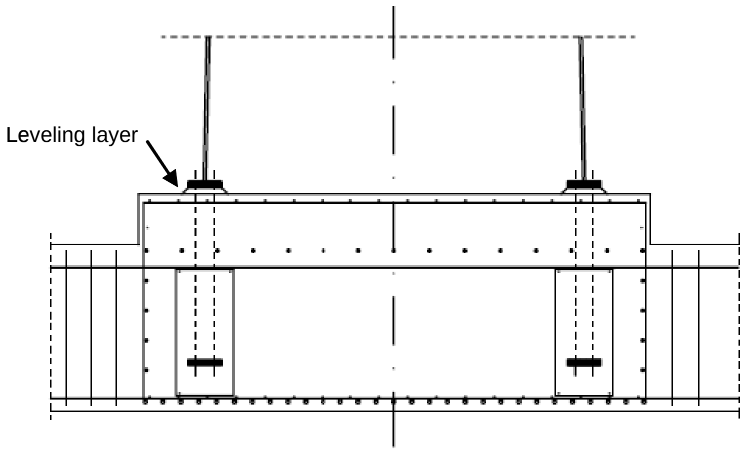

Figure 27 - Connection between the steel tower and the foundation, [15].

As it can be observed in Figure 27 there is a levelling layer of high-strength mortar grout in the transition zone between the steel plate of the tower and the concrete surface, see also Figure 32. The function of the mortar grout is to withstand the compressive forces from the tower and distribute it on the concrete surface. Therefore, flatness of the concrete surface, contact area and the bonding between the concrete and the mortar, the strength of the mortar and the contact area between the mortar and the flange are important factors in this context.

When mounting, the tower segment is first placed on three contact elements followed by the alignment of the segment. After alignment the mortar is poured inside the space between the concrete surface and the flange of the segment. The anchor bolts are tightened when the mortar has been hardened.

There are sometimes problems such as:

- Shrinkage cracks
- Excess material at the edge
- Lack of strength due to processing at low temperatures
- Voids between the concrete and the tower segments due to insufficient injection of mortar grout.

The first two problems are actually harmless. However, the two other problems require some remedial actions to be taken, which may necessitate the replacement of the mortar grout.

Figure 28 - Vertical shrinkage cracks, [15].

Figure 29 - Side excess material, [15].

Some other damages associated with the defect mortar grout are reported by [9], for instance: soft layer due to separation in mortar, air inclusion inside the mortar and below the flange and non-uniform grouting material.

Figure 30 – Weak mortar grout caused by separation of the mortar, [9].

Figure 31 – Inclusion of air voids inside the mortar grout under the flange, [9].

Figure 32 – Two different mortar grouts are used, [9].

4.3 Cracks in concrete foundation pedestal

The foundation pedestals may be divided into two groups, namely “short pedestals” and “high pedestals”. This subdivision is not commonly used, but it is used in this report in order to distinguish between to types of structures. The author has no data about the height of different pedestals used in praxis. The height of the short pedestals which the author has been observed in different drawings is normally lower than 1 m. The height of the high pedestals, which are considered in this report, is about 4 to 8 m. It should be noted that the type of cracks which are described below refers to high pedestals. The author has no information about the similar cracks in short pedestals.

A foundation type which is used, mainly in Germany, to achieve a significant increase in the hub height, and energy production, was steel towers mounted on the foundation pedestal [15]. The solution was a circular pedestal of about 4 to 8 m height on the existing foundation. The wall thickness of the pedestal was approximately 1 m in order to have enough space for the bolts or wires for mounting of the steel tower. The steel tower can be mounted either by wires or by the pre-stressed anchor bolts. This type of structure shows many cracks, which generally are horizontal. The crack widths are not negligible, where as many of them are even larger than 0,4 mm.

Figure 33 - Cracks in the reinforced door area, [15].

The occurrence of the cracks may be explained by the fact that the concrete foundation is designed with regard to the ultimate limit state and not for the serviceability state. It is possible that the stresses caused by the loads at serviceability state in combination with the thermal stresses have caused the cracks. It should be noted that during the winter, the inside of the structure is warmer than the outside, since the operating system inside the tower produce heat. In the summer, however, the outside of the structure is warmer due to the solar radiation. Please consider the stress/strength analysis provided in [15].

It has been observed that the cracks go through the structure. As the Figure 34 shows the crack leads in water and dirt from the outside to the inside of the structure.

Figure 34 – Cracks on the concrete pedestal inside the structure, photo Hassanzadeh 2011.

4.4 Cracks in foundations with insert ring

The tower of the wind turbine is connected to the foundation by means of a ring/tube partially inserted, cast-in, in the foundation. The inserted ring contains a T-flange at the bottom, the inserted part, and an L-flange at the top. The T-flange anchors the tube to the concrete foundation while the L-flange is for mounting the tower. There are also insert rings with two anchoring flanges. The second flange is placed on the upper part of the ring close to the surface of the concrete.

Both types of construction have caused cracking in concrete foundation. The mechanisms causing the cracks have been discussed in [15], [16], [17], [18], [19] and [20]. The mechanisms that cause the cracks will be presented in sections 4.4.1 and 4.4.2, which are entirely based on [15].

4.4.1 Insert ring with single anchoring flange

Figure 35 shows an insert ring with single anchoring flange. As it can be observed the inserted ring contains a T-flange at the bottom and a L-flange at the top. The T-flange anchors the tube to the concrete foundation while the L-flange is for mounting the tower.

A fundamental difficulty concerns the reinforcement on the upper part of the foundation. The reinforcement must be threaded through the holes in the wall of the insert ring, and shall not be in contact with the steel.

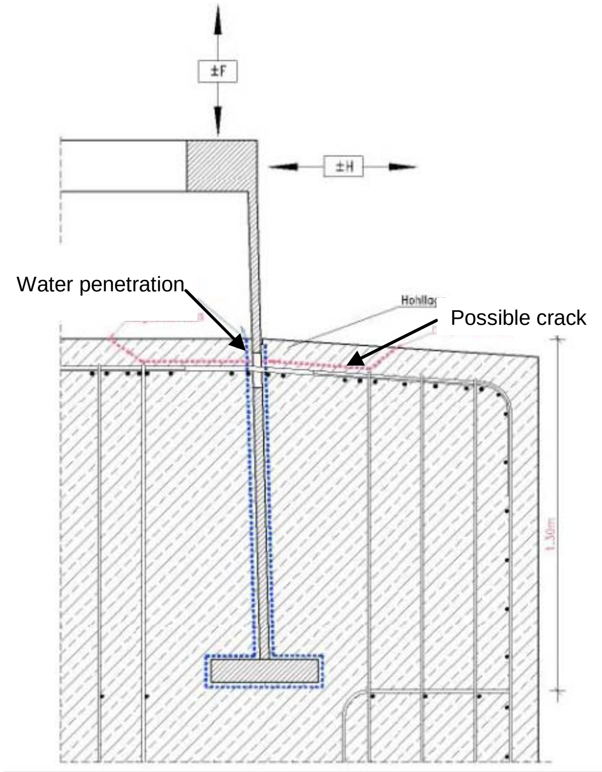

Figure 35 – Damages within the transition areas, [15].

In smaller wind power turbines this type of connection has functioned satisfactorily. In large wind power turbines, however, this solution has caused some concerns regarding cracking and de-bonding. The mechanisms behind the damages, Figure 35, are described below:

- Strains in the steel tube due to the loading lead to differential displacements between the tube and the concrete. The result is formation of a gap between the tube and the concrete.
- The horizontal displacements of the tube lead to stresses in the concrete cover which can't be resisted by the reinforcement. The stresses may lead to cracks in concrete cover and spalling of the cover.
- The horizontal forces also lead to small horizontal displacements/gaps which may increase displacements/movements of the upper part of the tube.
- The gaps and the cracks between the concrete and steel tube lead water. A flow of water around the insert ring and through it's holes will be induced due to the pumping effect caused by the alternating loads and displacements. The flow of the water may lead to leaching of the concrete and the mortar in the transition zones between the insert ring and the concrete.

Some damaged wind power turbines were inspected by means of the endoscopes. The results showed that the wash outs as wide as 3 mm had occurred between the concrete and the insert ring, and between the concrete and the anchoring flange. As a result the tight fit of the inserted ring and the concrete had been loosened. The movement of the ring relative to the concrete was clearly visible.

4.4.2 Insert ring with double anchoring flanges

Figure 36 shows an insert ring with two anchoring flanges cast in the concrete. The assumption made for the load sharing for the design considers the actions of the anchoring flanges separately, i.e. the upper flange transfers the compressive forces while the lower flange transfers the tensile forces to the concrete foundation, Figure 37.

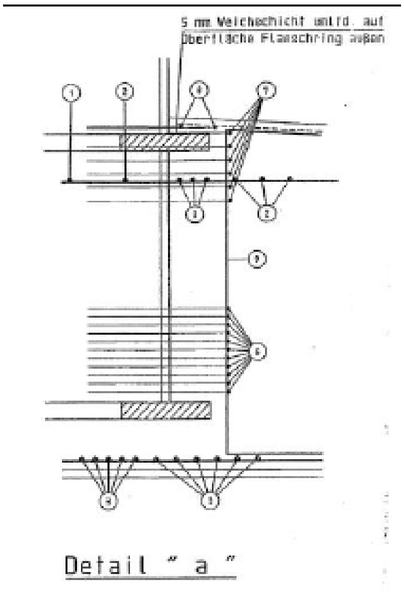

Figure 36 – Insert ring with double anchoring flanges, original details of an existing wind power turbine, [15].

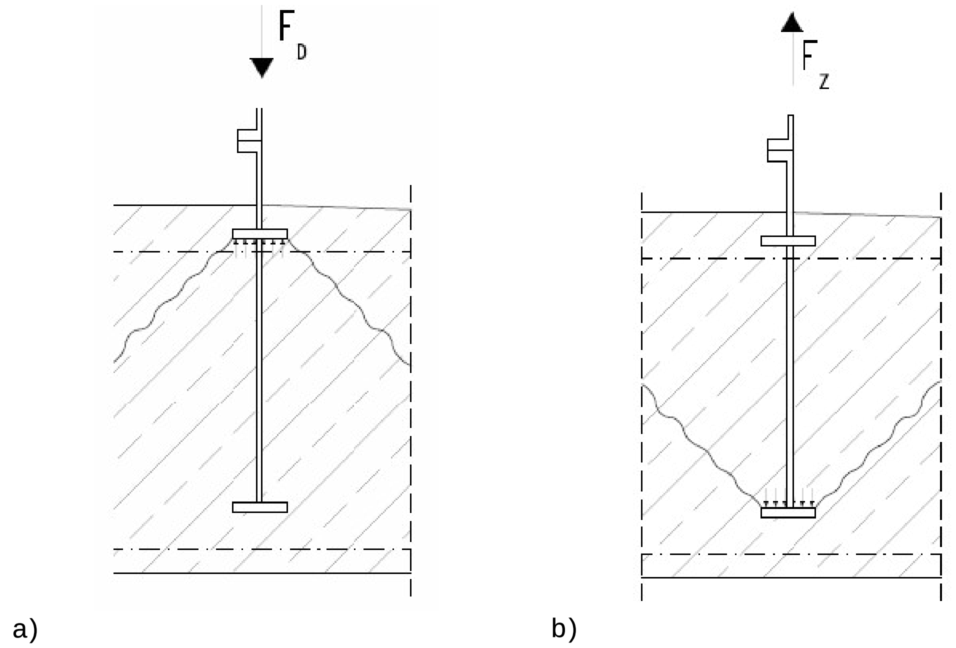

Figure 37 – Assumed model for transfer of the compressive (a) and tensile (b) forces, [15].

The assumptions made in the design model consider only the limit state bearing capacity and doesn’t regard the structural behaviour in the serviceability state. The model doesn’t account for the strains, extension and shortening, of the ring.

The change of the length can be determined as follows:

With the assumption that the stress, σ, and the modulus of the elasticity, E, are *105* N/mm² and *210000* N/mm² respectively the strain can be calculated as follows:

$\varepsilon = \sigma / E = 105/210000 = 0{,}5 \cdot 10^{-3}$

With the assumption that the length of the inserted part of the steel tube is *1000* mm, the length change of the ring will be *0,5* mm.

Due to the strains in the steel, the upper steel plate is pressed against the concrete cover. The concrete cover can't withstand the pressure and cracks as a result, Figure 38. The cracks are initiated on the upper edge of the anchor plate with 45 - 60 ° inclination and continue towards the surface of the foundation to emerge on the surface as concentric rings parallel to the tower wall, Figure 39. The pulling force thus leads to the movement of the upper plate, and cause the cracking at the upper edge of the foundation.

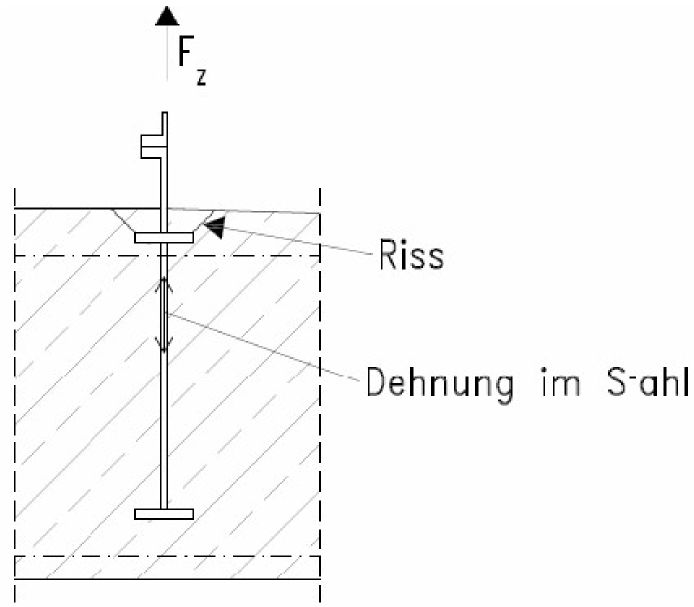

Figure 38 – Actual structural behaviour (Riss = Cracks, Dehung im stahl = strains in steel), [15].

Figure 39 – Significant crack approximately 35 cm from the tower, [15].

Figure 40 - Spalling (s) due to the cracks, with out-flowing water, [15].

The question is what happens in the case of the compressive forces? In general, the compressive forces are transferred by the shell of the ring to the lower flange and the concrete beneath the flange. The question is whether the upper flange carries any load or not. There is evidence suggesting that the compressive forces are in fact transferred to the flange at the bottom of the ring. Since the thickness of the foundation, concrete, under the flange is small it can't withstand the compression induced by the flange. It will crack in the same way as the upper part of the foundation above the upper flange as it is shown in Figure 41.

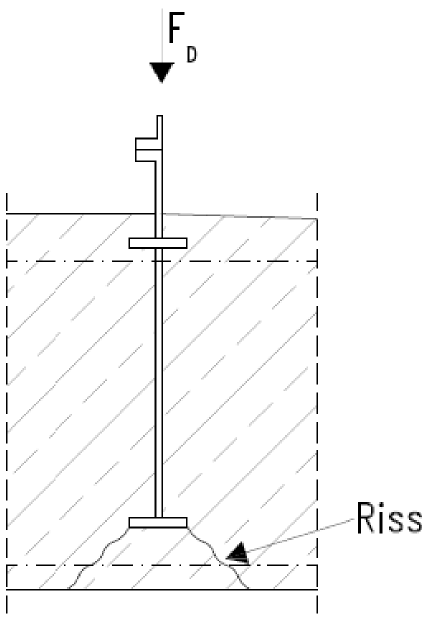

Figure 41 – Cracking caused by punching, [15].

The foundations of two wind power turbines were core-drilled. The drilling was carried out through the height of the foundations. As it was feared the punching cracks, as described above, were observed at the bottom of the foundation, Figure 42.

Figure 42 - Crack approximately 30 cm from the bottom of the foundation, [15].

Since the described damage type has frequently been observed, more detailed investigations were initiated.

Theoretically three mechanisms may cause vertical sliding between concrete and the ring. The mechanisms are as flowed:

- Extension of the steel wall due to tensile stresses
- Compression of the concrete between the steel flanges
- Shrinkage of the concrete between the steel flanges

The above-mentioned mechanisms may together cause 2 mm vertical sliding.

The sliding may be increased by other factors such as:

- Insufficient placing and compaction of concrete may cause that voids be formed under the flanges as well as between the steel ring and the concrete.
- Bleeding and separation of the components of the fresh concrete may create weakness zones below the upper flange.
- Cracks under the bottom flange caused by the compressive forces may disable the concrete to fully support the inserted ring.

Extensive assessments of various wind power turbines have been carried out. The sliding between the inserted tube and the concrete has been measured. The estimated sliding was within the range of 1 to 2 mm, which was also generally observed. However, in some cases much higher sliding were observed. In some cases 3 – 5 mm sliding was measured between the inserted ring and the upper part of the foundation. Sliding greater than 10 mm was also measured in some individual cases.

The observations reported by others are briefly outlined below:

- Cracks go from the upper anchor flange to the surface of the foundation as well as from the lower anchor flange to the bottom surface of the foundation.
- The insert ring de-bonds from the concrete and gaps are formed between the steel and the concrete.
- The insert ring is due to the above-mentioned mechanisms detached from the rest of the foundation. The forces are transferred through the dowel action of the reinforcements located in the top and the bottom of the foundation. Consequently it worsens the conditions.
- Due to the detaching there will be less weight available inside the foundation to balance the tensile forces.
- The anchorage is mainly brought about by the upper flange.
- Due to the bending the flange will be loaded unevenly resulting into bending stresses in the weld between the flange and the steel ring. The weld may fail due to the stresses, which leads to the failure of the structure.

4.4.3 Consequences of cracking

Water leaks into the foundation and around the reinforcement through the cracks at the top. Furthermore, the water may find its way to the lower parts of the foundation through the gaps and cracks between the insert ring and the concrete. The consequences are leaching of concrete, frost damage and reinforcement corrosion.

Only a very careful covering of the cracks and formation of a resilient groove in the transition between steel and concrete may be a suitable remedial action. However, the fundamental problem still remains.

The cracks at lower part of the foundation are not accessible. There may be risk for corrosion. The conditions for the corrosion to take place are that there must be electrolyte, oxygen and corrosion potential. All conditions are fulfilled. However, the reinforcement is protected by the alkalinity of the concrete. The protection of the reinforcement will be ceased either by the carbonation of the concrete or by ingress of chlorides. The carbonation of concrete is not possible due to high humidity and lack of CO_2. If the soil contains chlorides the reinforcement will corrode if the chloride concentration around the reinforcement reaches the threshold value. Furthermore, if cracks are wide the alkalinity in the cracks may be too low for the protection of the reinforcement against the corrosion.

5 Conclusions

As it is described in section 1 there are many perfectly designed wind power turbine foundations, which are not cracked and fulfil all functional requirements. However, there are foundations that have been cracked in such a way that they do not fulfil the requirements with regard to service life and structural stability. It was the aim of this investigation to determine whether the cracking of the foundations is a rare phenomenon or not. If not what are the causes, in which way they may influence the structural behaviour of the wind power turbines and how the cracks can be prevented. This report, by support of the referred sources, shows that cracking of the wind power turbine foundations is an issue which must be taken seriously. The report shows that the connection of the tower to the foundation by means of insert ring may lead to cracking of the concrete and formation of gaps between the concrete and the insert ring. Although the damages have no immediate consequences on the bearing capacity of the structure, they compromise the serviceability and long-term behaviour of the structure.

It has been pointed out that the main reason for the observed damages has been poor structural design, i.e. the solutions which were developed for smaller facilities, sometimes without thinking about possible consequences, have been applied to the larger wind turbines. Furthermore, the site investigations were not conducted properly and the findings were not brought back to the design praxis.

Besides the above-mentioned damage type two other types of cracking has been observed. One type is the cracking of the mortar grout between the connection flange and the concrete foundation. This type of cracking is caused by poor workmanship performance and inappropriate material selection. The other type observed is the cracks in the high pedestals, i.e. pedestals with a height of about 4 to 8 m. The cracks are caused by poor structural design. It should be noted that foundations with high pedestals were used in order to gain higher hub heights. This type of design was, however, not commonly used in the past and it is almost abandoned at the present time. This design has not commonly been used in Sweden.

However, cracking of the pedestal type which is described in this report indicates that the design praxis and guidelines should be further developed. As it is pointed out in [15] the serviceability state has been neglected in design of the pedestal. The reason may be that the manufacturers of the turbines deliver the failure loads, which are used to design the structure at the limit state, while the serviceability state design which includes determination of displacements, vibrations, crack risks etc is neglected. Moreover, cracking of the pedestal shows that tall concrete structures subjected to dynamic loads should be prestressed. It should be noted that this type of structure is not common in Sweden.

As mentioned earlier in the report there is another method for connection of the tubular steel tower to the foundation. The connection consists of a *steel adapter* which is fixed by stud bolts cast in concrete. The author has not encountered any report or article on the cracking of foundations containing steel adapter. Neither of the persons who the author met was aware of any damage cases involving foundations containing steel adapters.

Like any other reinforced concrete structures subjected to bending moment the spread footing foundations contain cracks. Although the cracks are unavoidable in these structures their size can be limited by means of careful detailing and design. The author has not found any guidelines which deal with detailing and design of the foundations, i.e. there is no organised dissemination of the knowledge regarding design of foundations for wind power turbines. Consequently, due to the lack of knowledge the number of damage cases may increase with an increasing number of wind turbines built.

6 Proposal for future research

6.1 Background

This report deals with the results of the phase 1 of a two phase project. The original overall aims of the project (both phases included) were to:

- document the failure process of cracking wind turbine foundations from examples in Germany and Denmark,
- determine types and causes of the cracks,
- develop guidelines for inspection, assessment and analyses in order to determine the causes of the cracks and
- develop guidelines for design and detailing of the foundations for the land based wind power turbines in Sweden in order to avoid cracks.

It was decided that research contents of the phase 2 should be determined after completion of the phase 1. The aims of the phase 1 were as followed:

- State-of-the-art report which includes type of foundations used for different types of towers and ground conditions, types of connections between towers and foundations used for different types of tower solutions, loading conditions, design basis, etc.
- Site visit and documentation of the observed cracks and the structural information.
- Preliminary analysis in order to describe the cause of the cracks
- Suggestions for a detailed study.

This report presents results of the phase 1. The last item in the list presented above will be presented in the next section.

6.2 Proposal for principal research direction

The remaining aims of the project, which concerns phase 2, are to:

- develop guidelines for inspection, assessment and analyses in order to determine the causes of the cracks and
- develop guidelines for design and detailing of the foundations for the land based wind power turbines in Sweden in order to avoid cracks.

It is still important to perform both project elements that are presented above, but with some changes. Below the contents/aims of the subsequent parts of the project are described in general terms.

- Development of guidelines for planning and production of wind power turbine foundations.

- Development of guidelines for production and quality control, inspection and damage assessment.
- Development of guidelines and handbook for repair of damaged foundations.
- Development of guidelines for design and detailing of the foundations for the wind power turbines.
- Evaluate the effect of cracks and damages on structural behaviour of the integrated response of the tower, foundation and ground, by means of experiments and numerical simulation.

The first four bulletins can be written based on the current knowledge and should developed by a group of experienced personnel where knowledge from other fields will be included where applicable.

The last bulletin requires integrated and long-term resources and it is therefore recommended that a PhD-project should be started. As mentioned elsewhere in the report the turbine manufacturers have profound knowledge about the wind power turbines. Since, the knowledge is not in the public domain it is difficult to compile a state-of-the-art regarding the level of the knowledge. For an individual who is not co-operating with a manufacturer it is difficult to find thorough studies regarding structural behaviour of wind power turbines. The structural behaviour is defined here as the integrated response of the tower, foundation and ground to the loads generated by the rotor and the generator. Numerical modelling should be applied together with measurements and experimental verification in order to understand the structural behaviour of the wind power turbines, towers, foundations, connections and other details. The generated knowledge will promote developments within the field of the wind power turbines. The knowledge and results from this suggested PhD-project has the aim to improve the design and detailing and also on repair of damaged wind power foundations and it is possible that the project initiate further development of the guidelines that are suggested above.

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[10] Mobjer Entreprenad AB, Falkenberg.

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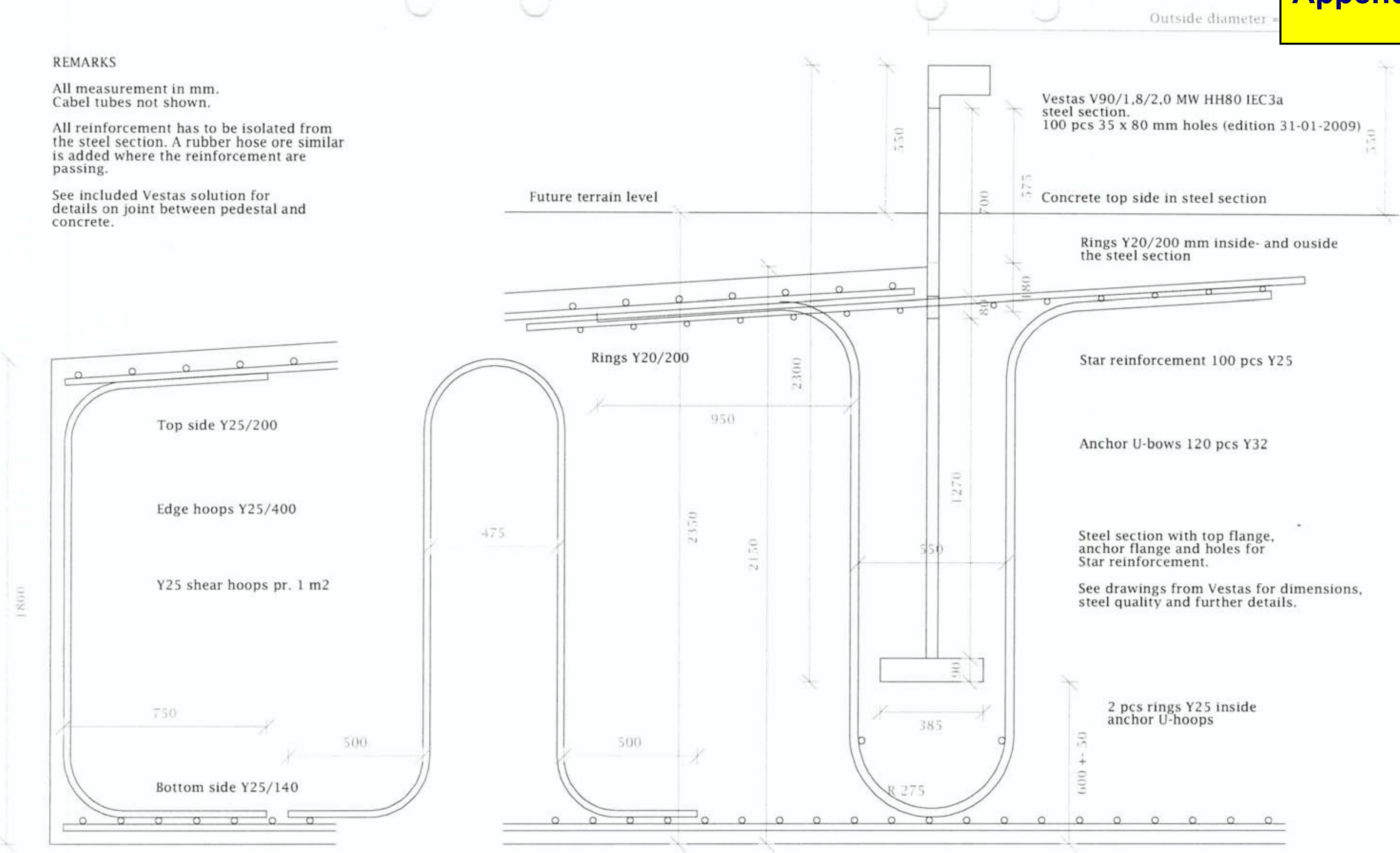
Appendix 1 – 1/1

REMARKS

All measurement in mm.
Cabel tubes not shown.

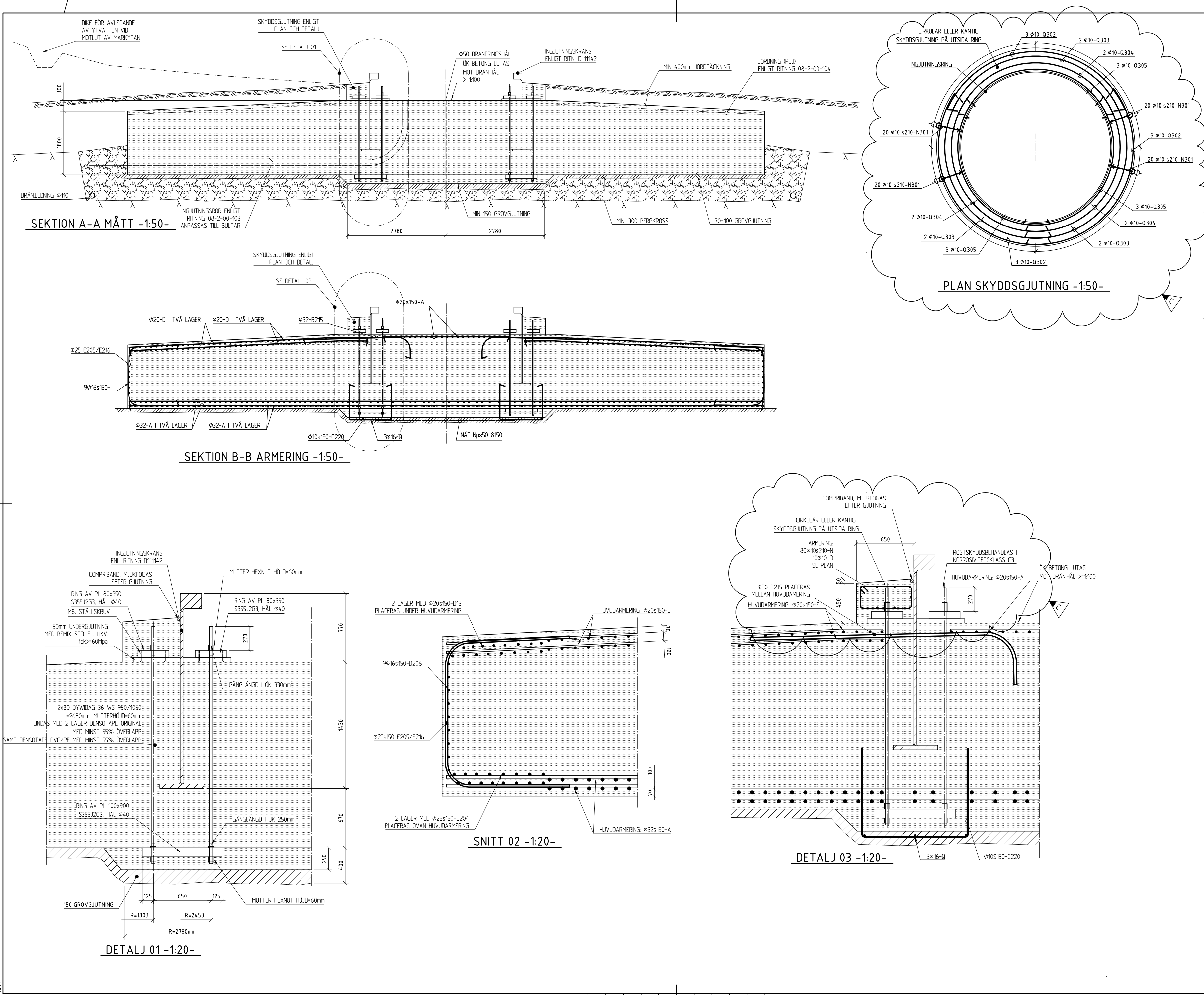
All reinforcement has to be isolated from the steel section. A rubber hose ore similar is added where the reinforcement are passing.

See included Vestas solution for details on joint between pedestal and concrete.



Case	Vattenfall Vindkraft AB Project: O stra Herrestad Vindkraftspark Sweden	Case no	034-09
Subject	Foundation for Vestas V90/1,8/2,0 MW - 80 m tower Vertical section - General	Drawing	V9580-04
		Scale	1:20
thisted aps	Strandvænget 24, DK 9340 Asaa +45 8692 6488 thistedaps@mail.dk	Date Rev	01-09-2009 17-11-2009
This drawing is our property. Copying, transfering or other forms of utilization by a third party is prohibited witout our written permission		Sign	peterthisted

Appendix 2 – 1/2



ALLMÄNNA ANVISNINGAR:

SE RITNING K32-31-02

FÖRKLARINGAR:

-
-

HÄNVISNINGAR

UNDERKANTSARMERING SE RITNING K32-31-03
ÖVERKANTSARMERING SE RITNING K32-31-04
MÅTTSÄTTNING SE RITNING K32-31-02
KABELSKYDDSRÖR SE RITNING 08-2-00-103
POTENIALUTJÄMNING SE RITNING 08-2-00-104
INGJUTNINGSKRANS SE RITN. D111142 (WINWIND)

ARMERINGSFÖRTECKNING NR. 209

BET	ANT	ÄNDRINGEN AVSER		DATUM	SIGN
C		SKYDDSGJUTNING	PL	2009-08-06	
B		UNDERGJUTN, UPPSTICK BULTAR	PL	2009-07-07	
A		FÖRTJ. UK, GROVGJ.DETALJ 02,03	PL	2009-06-22	

BYGGHANDLING

SKELLEFTEÅ KRAFT AB

ULJABUOUDA VINDKRAFTSPARK

WSP BYGGPROJEKTERING
S JÄRNVÄGSGATAN 47
931 32 SKELLEFTEÅ
Tel: 0910-58 64 00
Fax: 0910-58 64 20

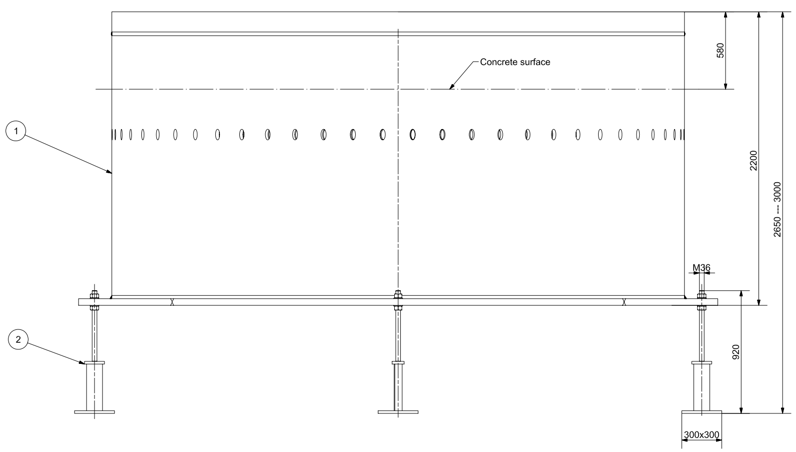
UPPDRAG NR	RITAD/KONSTRUERAD AV	HANDLÄGGARE
1011 5392	PL	Anders Enquist
DATUM 2009-06-15	ANSVARIG	

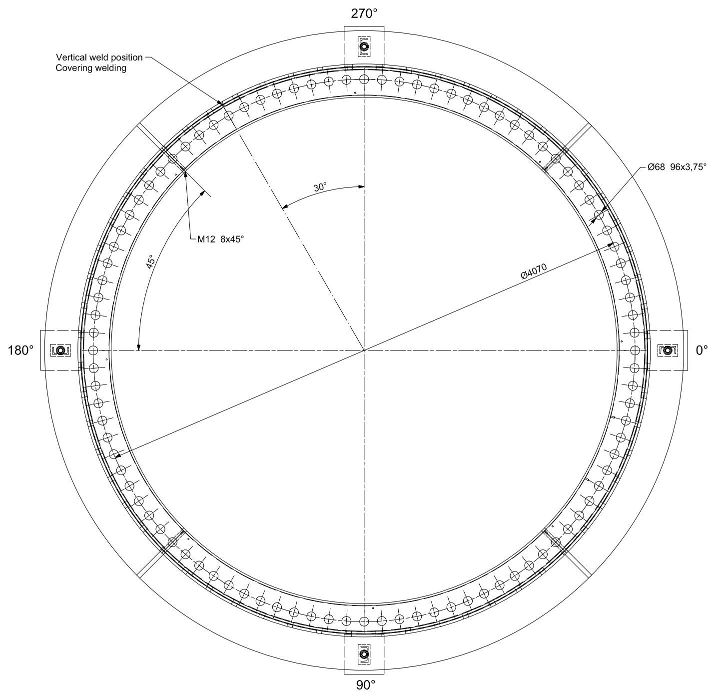
FUNDAMENT VERK 4-7
MÅTT
SEKTION, DETALJ

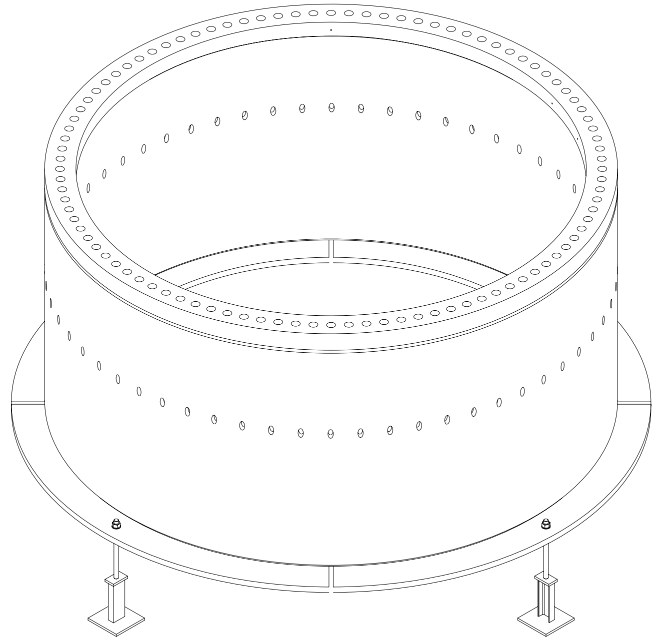
SKALA	NUMMER	BET
1:50, 1:20	K32-31-05	C

REF: LAGER:

Appendix 2 – 2/2







Item	Qty	Stock	Title/Description	Part Number/Dimensions	Material	Mass
2	4	D1123	Support	D1123		22.5
1	1	D111138	Cover	D111138		14491.7

Parts List

Following general tolerances shall apply if not otherwise stated:
Machining: ISO 2768-mK
Weldings: ISO 13920-BF
Castings: ISO 8062 CT-12
Flame cutting: ISO 10029, ISO 9013 Class II B, Field 2

Project: WWD-3
Paper size: A1
Overall scale: 1:20
Calculated mass: N/A
Title: Foundation part Tower S80

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Finland
www.winwind.fi

Designed by: ANy — Date: 13.10.2008
Checked by: MAu — Check date: 26.11.2008
Approved by: MIK — Approval date: 26.11.2008
Inc.
Stock: D111142
D111142
Revision: 0

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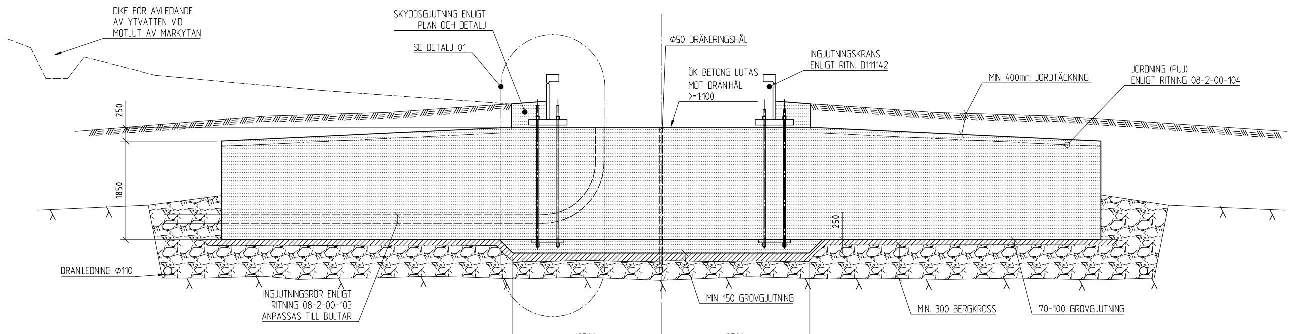
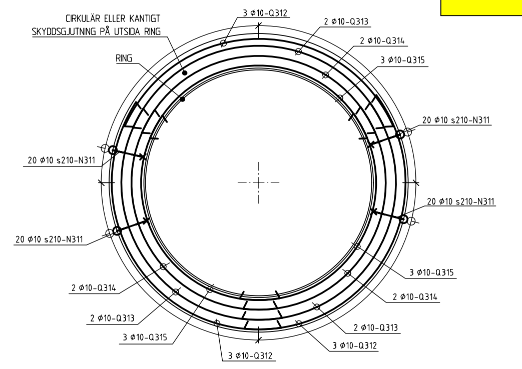
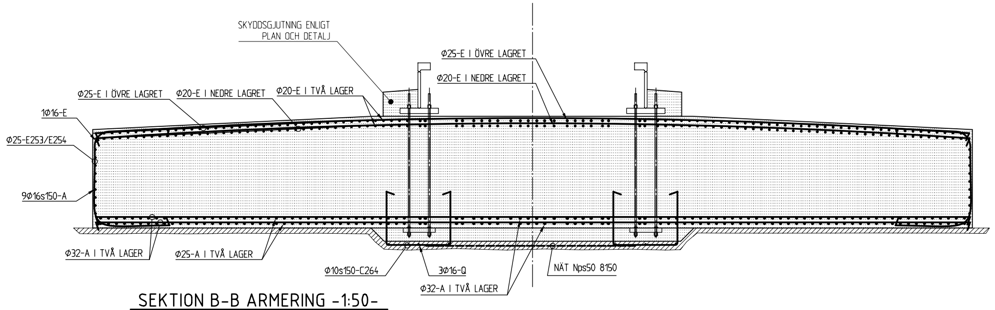
Appendix 3 – 1/4



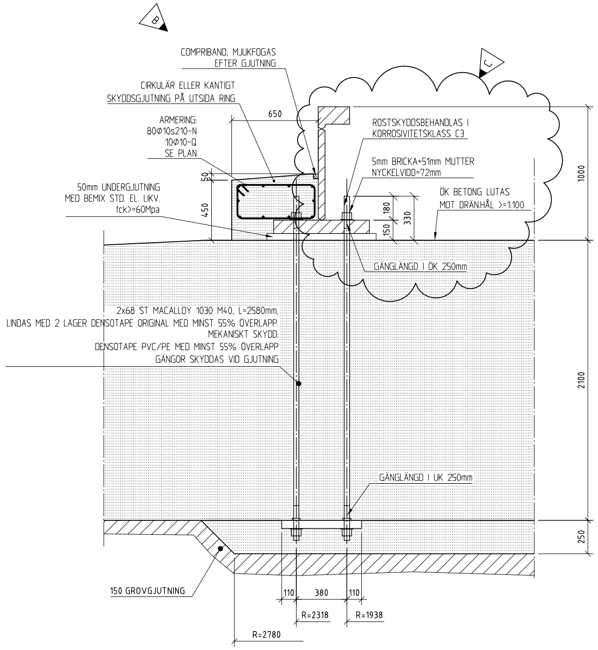
SEKTION A-A MÅTT -1:50-



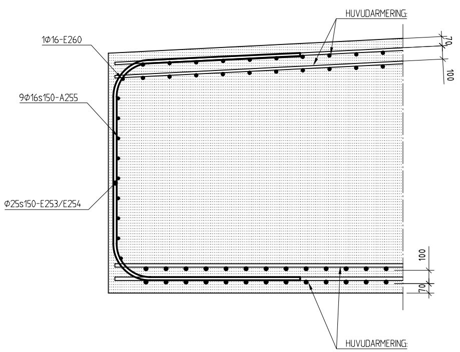
PLAN SKYDDSGJUTNING -1:50-



SEKTION B-B ARMERING -1:50-



DETALJ 01 -1:20-



SNITT 02 -1:20-

ALLMÄNNA ANVISNINGAR:

SE RITNING K32-31-02

FÖRKLARINGAR:

-
-

HÄNVISNINGAR

MÅTTSÄTTNING SE RITNING K32-31-08
UNDERKANTSARMERING SE RITNING K32-31-09
ÖVERKANTSARMERING SE RITNING K32-31-10
BYGELARMERING SE RITNING K32-31-12
KABELSKYDDSRÖR SE RITNING 08-2-00-103
POTENIALUTJÄMNING SE RITNING 08-2-00-104

ARMERINGSFÖRTECKNING NR. 210

BET	ANT	ÄNDRINGEN AVSER		DATUM	SIGN
C		MÅTT TILL BULTAR	PL	2009-08-20	
B		SKYDDSGJUTNING	PL	2009-08-06	
A		UNDERGJUTN. MACALLOY BULTAR	PL	2009-07-07	

BYGGHANDLING

SKELLEFTEÅ KRAFT AB

ULJABUOUDA VINDKRAFTSPARK

WSP BYGGPROJEKTERING
S JÄRNVÄGSGATAN 47
931 32 SKELLEFTEÅ
Tel: 0910-58 64 00
Fax: 0910-58 64 20

WSP

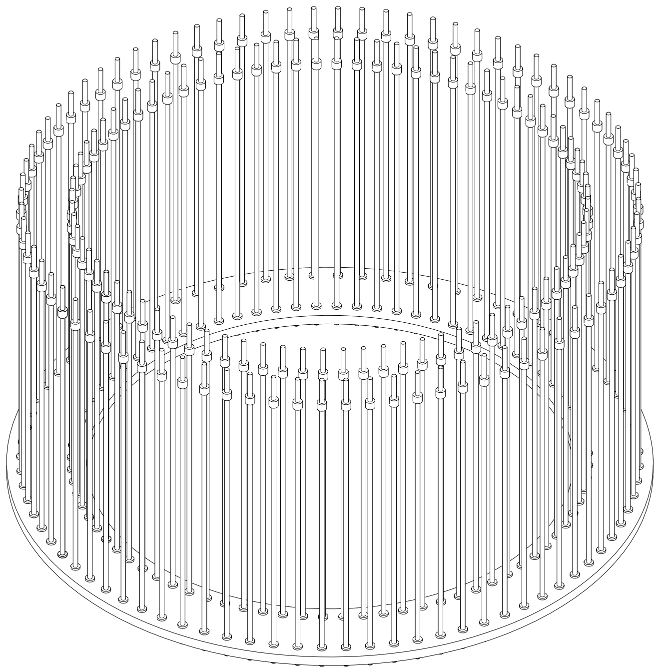
UPPDRAG NR	RITAD/KONSTRUERAD AV	HANDLÄGGARE
1011 5392	PL	Anders Enquist
DATUM	ANSVARIG	
2009-06-25		

FUNDAMENT VERK 1-3, 8-10
MÅTT OCH ARMERING
SEKTION, DETALJ

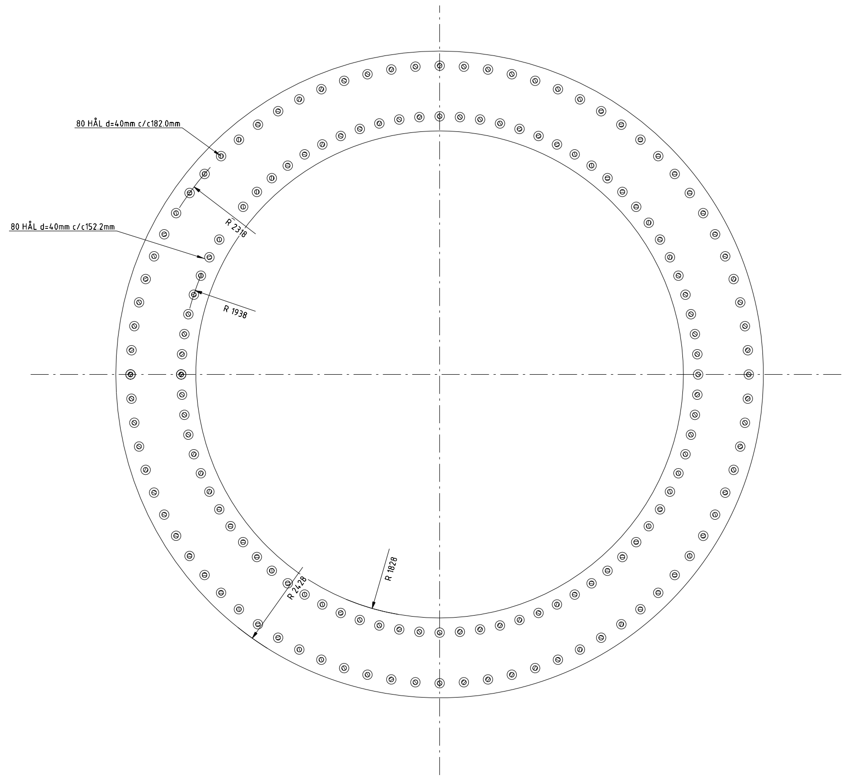
SKALA	NUMMER	BET
1:50	K32-31-11	C

REF:
LAGER:

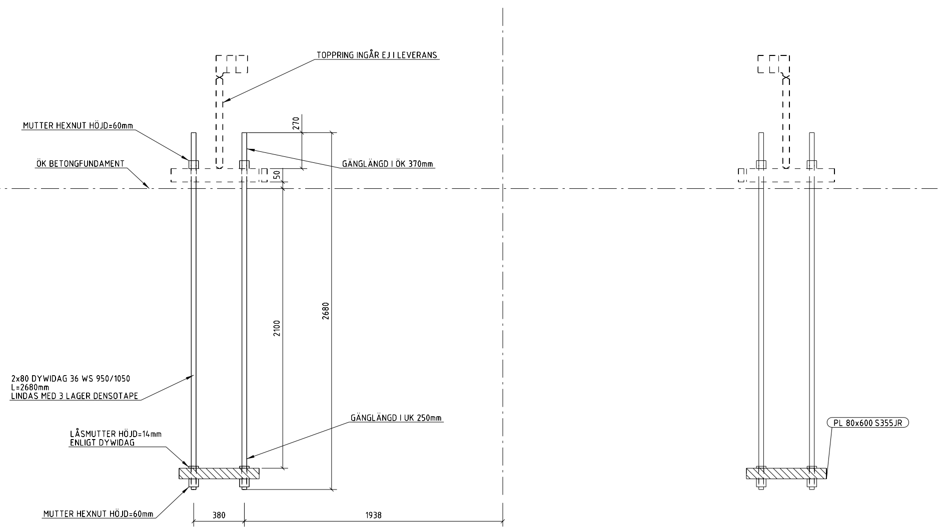
PLO: FIL: L:\2510\2008\10115392_ULJABUOUDA_CAD\K\RITDEF\K32-31-11.DWG PLOTTAD: 2009-08-28 08:08 AV ANVÄNDARE SEPL06292



ISOMETRI



Plan
1:20



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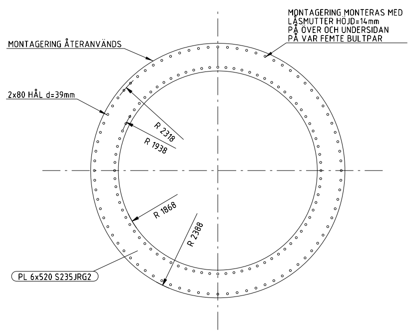


MONTAGERING
1:50

BYGGHANDLING

SKELLEFTEÅ KRAFT AB

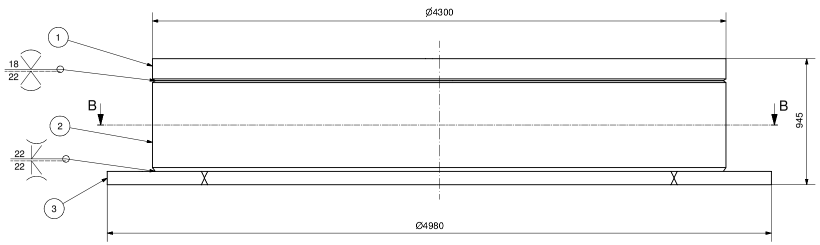
ULJABUOUDA VINDKRAFTSPARK

WSP Byggprojektering
S Järnvägsgatan 47
931 32 SKELLEFTEÅ
Tel. +46 910 58 64 00
Fax +46 910 58 64 20

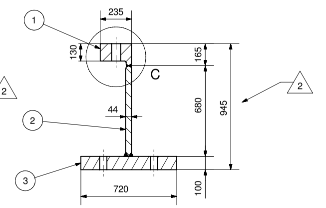
UPPDRAG NR	RITAD/KONSTRUERAD AV	HANDLÄGGARE
10115392		pl
DATUM	ANSVARIG	
2009-06-24	ae	

VINDKRAFTSVERK 1-3, 8-10
INGJUTNINGSRING
PLAN, SEKTIONER

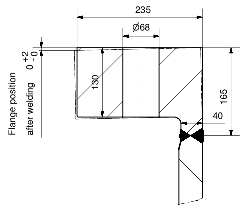
SKALA	NUMMER	BET
1:20 1:50	K32-31-13	



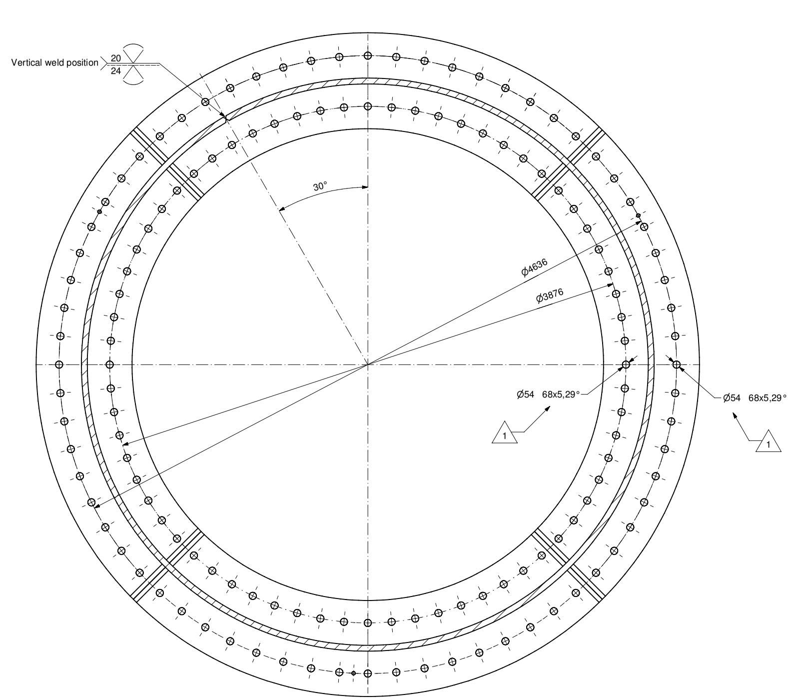
A-A (1 : 20)

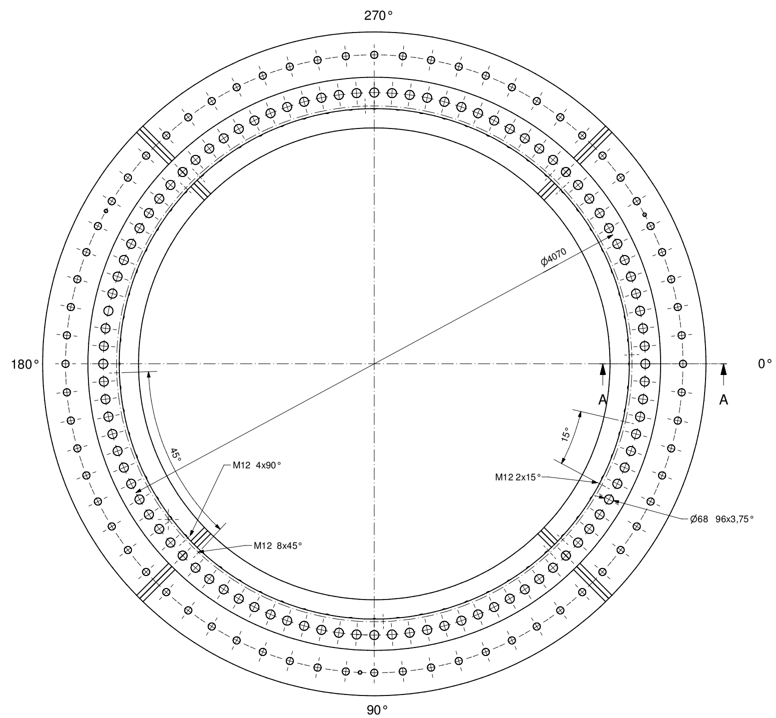


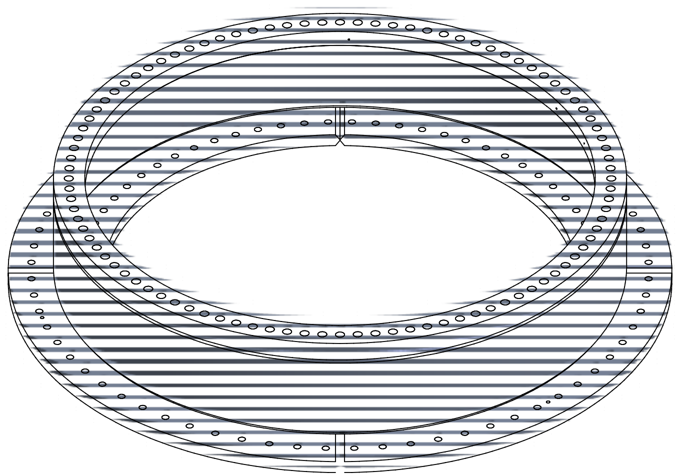
C (1 : 5)



B-B (1 : 20)







NOTES!

1. ALL WELDS FULL PENETRATION BUTT WELDS.

2. WELDS ACCORDING TO DOCUMENT S-TOW.01-A

3. FLANGES WELDING SEAM MUST BE LOCATED ON OPPOSITE SIDE FROM THE VERTICAL SEAM OF PLATES IF WELDING HAS BEEN USED IN FLANGE MANUFACTURING.

SURFACE TREATMENT ACCORDING TO DOCUMENT WWD-G-ENG-0-0346EN

Item	Qty	Stock	Title	Model code | Std, Size, Mat.	Mass
3	1	D111808	Flange		7245
2	1	D111807	Plate		3045
1	1	10023151	Flange 1		2833

Parts List

Following general tolerances shall apply if not otherwise stated:
Machining: ISO 2768-mK
Weldings: ISO 13920-BF
Castings: ISO 8062 CT-12
Flame cutting: ISO 9013 Class II B, Field 2

Project/Product	WWD-3
Paper size	A1
Overall scale	1:20
Calculated mass	13124 kg
Title	Stud bolt foundation steel adapter
Designed by	ANy
Design date	18.06.2009
Drawing made by	ANy
Drw make date	18.06.2009
Checked by	MAu
Check date	25.06.2009
Approved by	MiK
Approval date	25.06.2009
	Tower S80
Model code	20039045.1
Drawing code	20039047.3
Sheet nr.	1 / 1
Stock code	D111806
Revision	3

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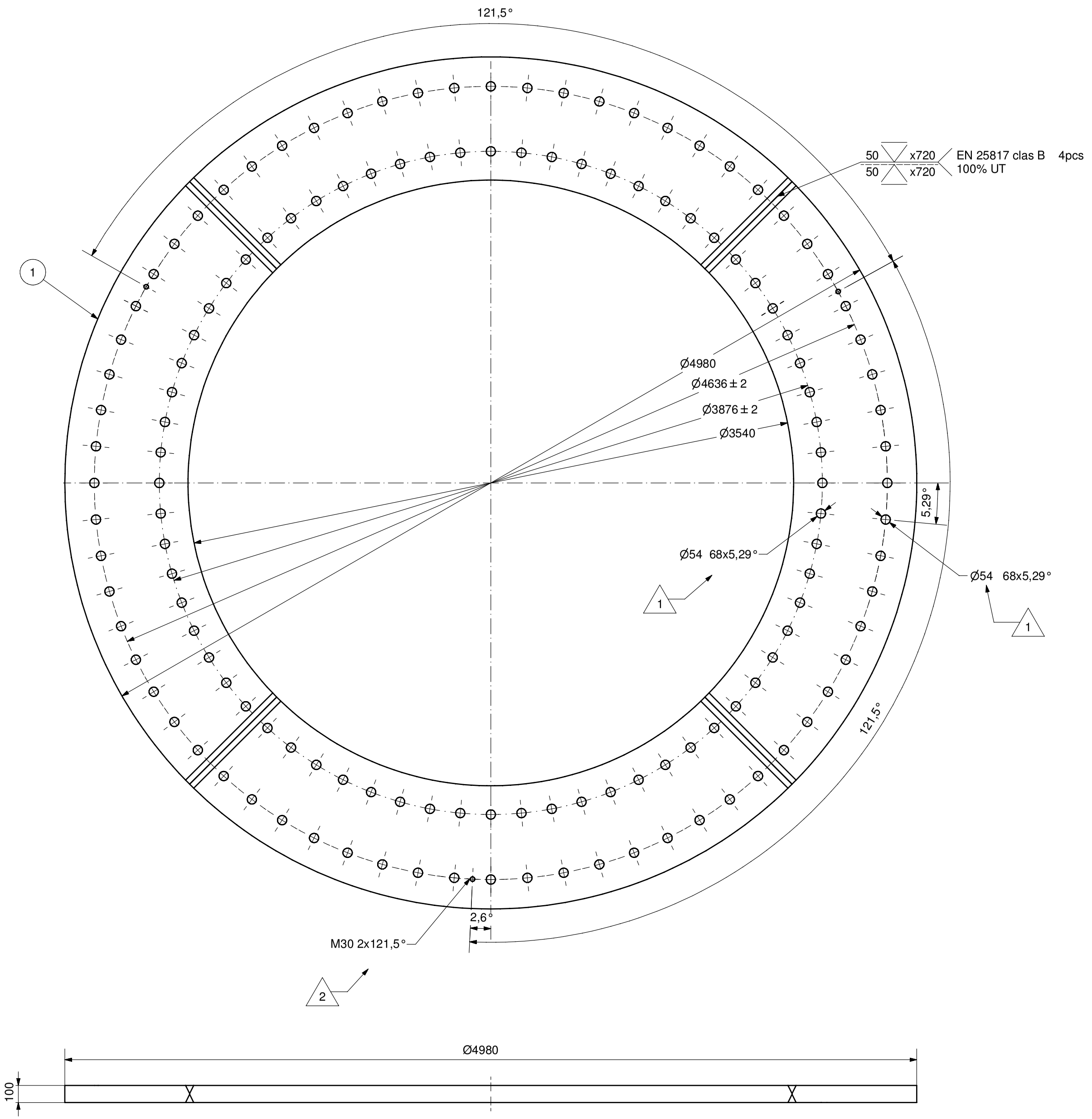
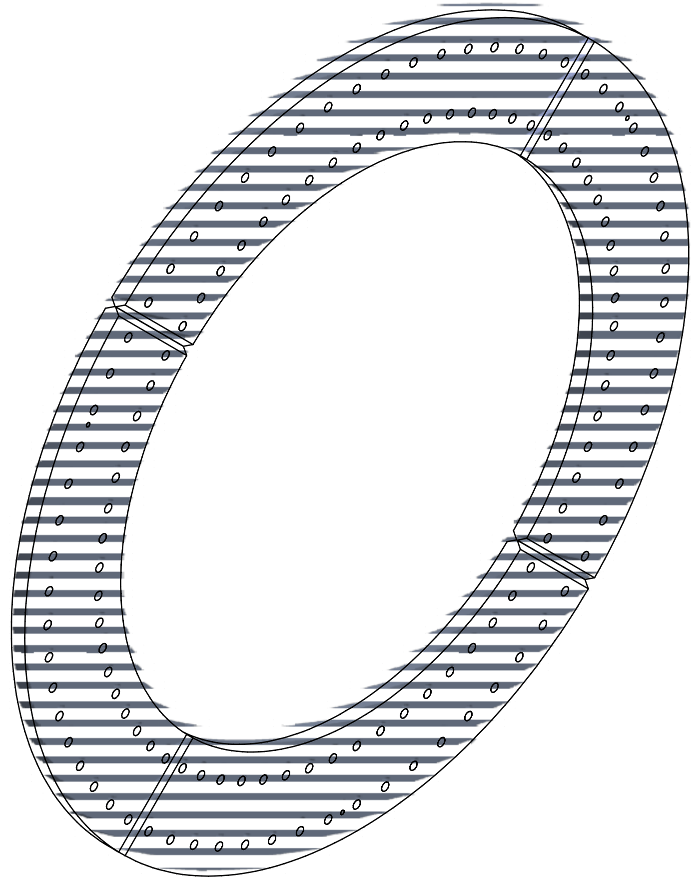
Rev	Zone	Dsgn	date	Chkd	date	Appd	date	Revision Note
3		ANy	09.02.2010	MAu	09.02.2010	MiK	09.02.2010	Parts 2 modified
2		ANy	9.10.2009	MAu	9.10.2009	MiK	9.10.2009	Parts 2 modified
1		ANy	16.09.2009	MAu	16.09.2009	MiK	16.09.2009	Dimension modified

Revision history

Appendix 3 – 4/4





Rev	Zone	Dsgn	date	Chkd	date	Appd	date	Revision Note
2		ANy	09.02.2010	MAu	09.02.2010	MiK	09.02.2010	Added M30 thread hole
1		ANy	16.09.2009	MAu	16.09.2009	MiK	16.09.2009	Dimension modified

Revision history

Item	Qty	Stock	Title	Model code | Std, Size, Mat.	Mass
1	4	D111809	Plate		1873

Parts List

Following general tolerances shall apply if not otherwise stated:
Machining: ISO 2768-mK
Weldings: ISO 13920-BF
Castings: ISO 8062 CT-12
Flame cutting: ISO 9013 Class II B, Field 2

Calculated mass: 7245 kg

Project/Product: WWD-3 | Paper size: A2 | Overall scale: 1:20

Title: Flange

Tower S80

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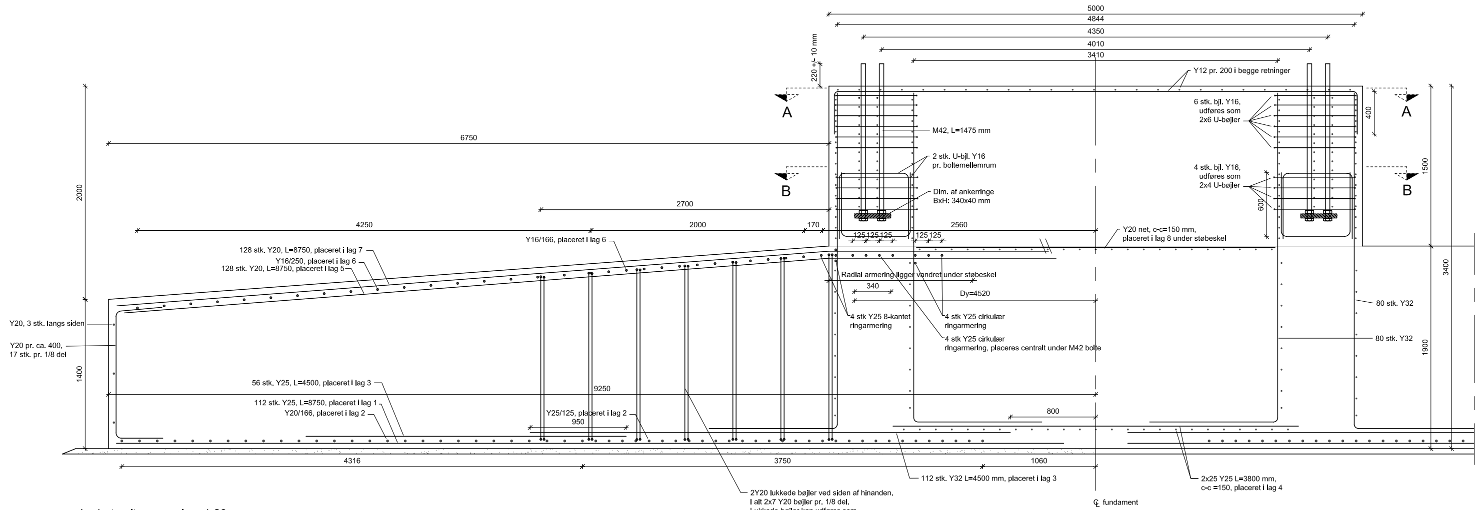
Designed by	Design date	Drawing made by	Drw make date	Checked by	Check date	Approved by	Approval date
ANy	18.06.2009	ANy	18.06.2009	MAu	25.06.2009	MiK	25.06.2009

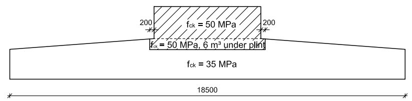
Model code: 20038902.1 | Drawing code: 20038944.2 | Sheet nr.: 1 / 1

Stock code: D111808 | Revision: 2

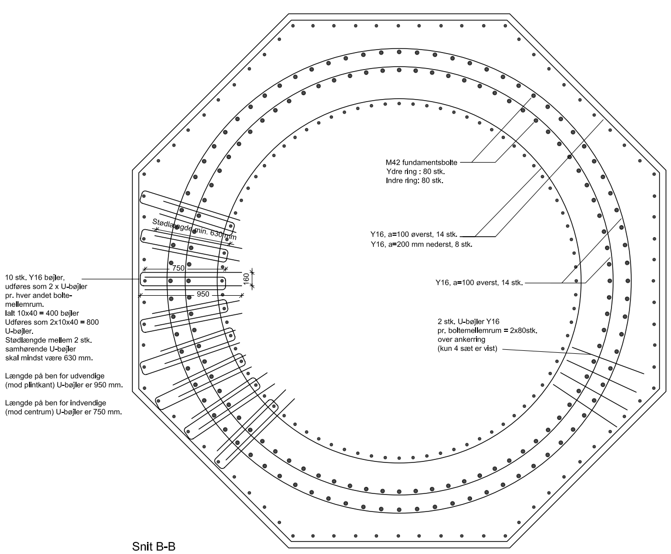
Appendix 4 – 1/1



Lodret snit - armering, 1:20

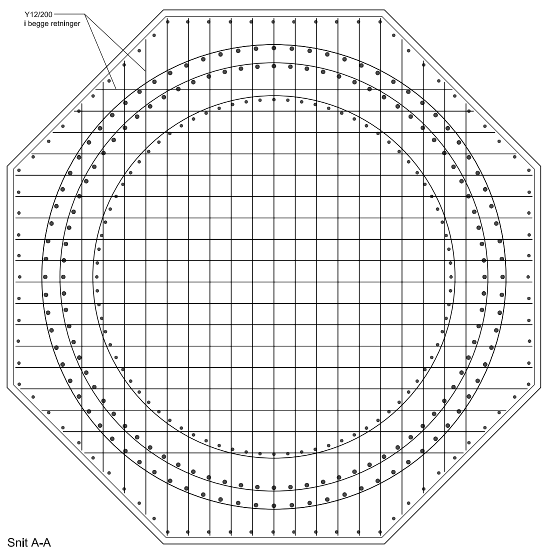


Angivelse af betonstyrke



Snit B-B

Armering i bund af plint, 1:20



Snit A-A

Armering i top af plint, 1:20

NOTE:

Ubenævnte mål er i mm.
Koter er i m, DVR 90.

Montagearmering er ikke vist.
Armering må ikke svejses.

HENVISNING:

Vedr. jording og placering af føringsrør henvises til tegn. nr. BN121 og BN122.

Vedr. materialer og udførelse henvises til tegn. nr. BN100 samt SB.

Vedr. radialarmering i bundplade, se tegn. nr. BN112 og BN113.

Index	Dato	Tegn.	Kontr.	Godk.	Beskrivelse

Filnavn:	Målestok:	Format:			
UMBN111	1:20	60x84	Tegnet	25.02.2010	BeK
			Kontrolleret	25.02.2010	DVG
			Godkendt	25.02.2010	CAN

Grontmij | Carl Bro VATTENFALL

2331.08

Vattenfall Vindkraft A/S
Hagesholm
Fundament - Armering
Lodret snit

UM/

BN111 a

Vattenfall Vindkraft A/S, c/o Oldenborggade 25 - 31, 7000 Fredericia, Danmark - Tlf. 88 27 50 00

Figure 1 – Footing founded on the bedrock. The footing is anchored to the bedrock. Please notice the load spreading plate which is anchored by the stud bolts. The tower flange will be mounted on the plate. Please also notice that the photos do not refer to the same object. However, the objects are in the same site and principally identical.

Figure 2 – Insert rings are also used for foundations on the bedrock.

ELFORSK

SVENSKA ELFÖRETAGENS FORSKNINGS- OCH UTVECKLINGS - ELFORSK - AB

Elforsk AB, 101 53 Stockholm. Besöksadress: Olof Palmes Gata 31
Telefon: 08-677 25 30, Telefax: 08-677 25 35
www.elforsk.se