Kari Avellan

LIMIT STATE DESIGN FOR STRENGTHENING FOUNDATIONS OF HISTORIC BUILDINGS USING PRETESTED DRILLED SPIRAL PILES WITH SPECIAL REFERENCE TO ST. JOHN’S CHURCH IN TARTU
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Abstract

This thesis discusses strengthening foundations of historic buildings by means of pretested, end-jacked, steel piles; pretested, end-jacked drilled spiral steel piles, as well as the geotechnical and structural design of strip foundations using ultimate limit state method. Strengthening the foundations of historic buildings is a field of engineering where every site is more or less different from another. The variation of the substructure, foundation, and superstructure make every case unique. Preserving heritage buildings by preventing structural decay requires know-how and suitable strengthening methods.

Drilled spiral piles and jacked piles were employed as strengthening methods for St. John’s Church of Tartu, because of the sinking and uneven settlement of the building. The strengthening work at St. John’s Church was challenging due to the risk of collapse of this historic church. Underpinning with jacked piles is a preferred method for strengthening historic foundations and where drilled spiral piles were employed, the author of this thesis developed special equipment.

The strengthening method employed for the tower complies with the anastylosis principle by preserving the authenticity of the structures. Old block stones are visible upon the floating piled rafts. The empty space was left to give archaeologists and engineers a chance to study the realized work “in situ” in the future.

The foundation of the tower was underpinned step by step with pretested jacked piles and a concrete raft poured step by step. The weight of the tower is 5 500 tons and it rests on four pillars. Every pillar rests on its own floating piled raft.

This thesis proves that by means of lower and upper bound theorems a floating, piled strip-foundation can be designed geotechnically and structurally using one method based on ultimate limit state (ULS) and serviceability limit state (SLS). The method takes the following into account as geotechnical requirements: contact pressure, total settlement and angular distortions; and as structural requirements; admissible plastic rotations, end moments due to displacement angle, as well as control of cracking. The chosen piling methods were suitable for St. John’s Church, and the installed piles work well in addition to the old foundations. The functionality of the strengthening techniques has been verified by test piling. Furthermore, every drilled spiral pile was preloaded twice and end-jacked for soil hardening.

Keywords: anastylosis principle, combined structural and geotechnical design in ULS, pretested drilled spiral piles, soil hardening by preloading piles and by end-jacking
Avellan, Kari, Historiallisten rakennusten perustusten vahvistaminen spiraaliporapaaluilla ja mitoittaminen rajatilamenetelmillä, esimerkkinä Johan-neksen kirkko Tartossa.

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Oulu

Tiivistelmä


Spiraaliporapaaluja ja puristuspaaluja käytettiin Johanneksen kirkon (Jaani Kirik) perustusten vahvistamisessa rakennuksen epätasaisen painumisen ja sortumisvaaran vuoksi. Perustusten vahvistamista puristuspaaluilla pidetään yleisesti parhaana menetelmana historiallisille rakennuksille. Tämän tutkimustyön tekijä suunnitteli erikoiskaluston työtä varten kehitetyn kierrepaalun.


Tornin perustukset vahvistettiin vaiheettaan esikuormitteilla ja esikuormitetuilla spiraaliporapaaluilla. Torni painaa 550 tonnia ja se on neljän pilarin varassa. Jokaisella pilarilla on oma, erillinen kelluva laattaperustuksensa.

Tässä työssä on osoitettu, että plastisuusteorian ala- ja ylärajalauseiden avulla myötävää-kitkamaan ja myötävää paalutukseen varaa perustettu pitkänomainen anturaperustus voidaan mitoittaa geotekninen ja rakenteen murtorajatilassa samalla menetelmällä. Menetelmä sisältää geoteknisinä vaatimuksin anturan pohjapaineen, kokonaispainumman ja kulmakiertymän (= epätasaisen painumisen) määrittämisen, rakenteellisina vaatimuksina plastisoituvien kohtien riittävän muodonmuutoskyvyn, kenttä- ja tukimomenttien, epätasaisen painumisen aiheuttamien pakkomomenttien sekä halkeamatakastelin määrittämisen.

Spiraalipoorapaaluja ja esikuormitettuja koepaaluilla. Maanaljittumista varten jokainen spiraaliporapaalu koestettiin kahdella esikuormituskella ja esikuormiteuvalla.

Asiasonat: anastylosis periaate, maanaljittuminen esikuormituksilla ja loppupuristuksilla, rakenne- ja geotekninen suunnittelu murtorajatilassa, spiraalipoorapaalu
Acknowledgements

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The subject of this study is partly based on the issues already drafted in my licentiate thesis, “Geotechnical and Structural Ultimate Limit State Design of Foundations Resting on Soil (in Finnish)”, written in 1992. I am also grateful to the supervisor of my licentiate thesis, Prof. Emer. Kalle-Heikki Korhonen, and Prof. Emer. Aarne Jutila, who was instructor of my licentiate thesis. In addition, special thanks to Konsultointi KAREG Oy for essential help in realisation of this thesis. In addition, special thanks to M. Sc. Nanna Ronkainen for essential help to realize this thesis. I thank Mrs. R. Vílmi for finalizing the English edition of the manuscript.

Helsinki, September 2011

Kari Avellan
Abbreviations and symbols

A  Area
A_F  Sub-area of foundation
A_s  Area of tensile reinforcement
A_p  Area of prestressing tendons
B  Width of the strip foundation, diameter of frame
B_{eff}  Effective breadth
B_0  Valid effective breadth
CT  Construction time of floating piled raft foundation
D  Depth of foundation
E_c  Long-term value for Young’s modulus of concrete
E_{cm}  Value of E_c at 28 days
E_d  Modulus of soil deformation in drained state
F_{adm}  Admissible force
F  Force
F_d  Design value of external vertical action defined from superstructure in ultimate limit state
F_{kp}  Value of external vertical pile load in serviceability state
F_{test}  Test load to prove resistance of pile
F_{yk}  Lateral loads
G_F  Dead weight of foundation
G_k  Characteristic value of a permanent action
H  Height of foundation, plastic hinge, structural depth
I  Influence factor depending on stress distribution
I_c  Effective second moment of area
I_p  Influence factor depending on rigidity, foundation size, depth of hard strata
K_{FA}  Subgrade reaction
L  Length
L_0  Length of strip foundation
M  Bending moment
M_p  Plastic bending moment
M_{gy}  Fixed end moment resulting from angular distortion
N_B  Bearing capacity factor
N_D  Bearing capacity factor
P_p  Passive pressure at failure
P_{p(S)} \quad \text{Passive pressure resulting from settlement, passive pressure resulting from movement}

PT \quad \text{Pretesting and end-jacking date}

Q_c \quad \text{Creep load}

Q_d \quad \text{Variable action in ULS, design value}

Q_k \quad \text{Variable action in SLS, characteristic value}

Q_s \quad \text{Soil reaction force}

R_u \quad \text{Ultimate compressive resistance of the ground}

R_d \quad \text{Design value of the resistance to an action}

RP \quad \text{Rotation point}

S \quad \text{Settlement}

S_B \quad \text{Shape factor}

S_D \quad \text{Shape factor}

S_F \quad \text{Settlement of foundation}

S_f \quad \text{Settlement at failure}

S_{(L0)} \quad \text{Settlement of total strip foundation}

S_p \quad \text{Settlement of pile}

S_{sq} \quad \text{External design load resulting from superstructure divided by design value of ultimate resistance of strip foundation}

V_d \quad \text{Vertical load in ULS, design value of a vertical load in ULS}

V_{dp} \quad \text{Vertical design load of piles}

V_k \quad \text{Vertical load in SLS, characteristic value of a vertical load}

V_{kp} \quad \text{Characteristic load of superstructure to piles}

W_i \quad \text{Virtual internal work}

W_e \quad \text{Virtual external work}

Z_P \quad \text{Depth from foundation level to non-compressible strata}

\text{subscript F} \quad \text{Symbol for individual foundation or part of strip foundation}

\text{subscript S} \quad \text{Symbol for slab part of strip foundation}

\text{subscript d} \quad \text{Symbol for ULS; design value of load, design value of material strength etc}

\text{subscript k} \quad \text{Subscript for characteristic value of action effects and material properties etc. SLS}
d effective depth of tension reinforcement

e_{yk} Eccentricity on y-axis

f_{0.1k} Characteristic (0.1 % proof) strength of (prestressing) steel

f_{cd} Design value of concrete cylinder compressive strength

f_{ck} Characteristic compressive cylinder strength of concrete at 28 days

f_{yd} Design yield strength of reinforcement

f_{yk} Characteristic strength of reinforcement

i Numerical symbol

k_F Spring coefficient of foundation

k_p Spring coefficient of pile

k_{PT} Total pile spring coefficients

l Effective span

p_d Design value of contact pressure from the superstructure

p_{0d} Mean design contact pressure calculated from superstructure beneath the whole strip foundation

q Contact pressure, foundation base resistance pressure

q_d Design value of contact pressure from the substructure

q_u Soil pressure at failure, ultimate value of q

q_0 Average contact pressure

q_{0d} Mean design contact pressure of substructure beneath the whole strip foundation

z_p Depth, downwards from the foundation level

\alpha Angle, form factor, aspect ratio

\beta Angle, coefficient for calculating A_s

\gamma Safety factor, combination of safety factor

\gamma_s effective unit weight of soil

\nu Poisson’s ratio

\rho_{ed} Equivalent reinforcement ratio

\rho_p Reinforcement ratio for the prestressing tendons

\rho_s Reinforcement ratio for tensile reinforcement

\sigma_s The stress in the reinforcement

\sigma_{sk} Steel stress in SLS resulting from bending moment

\psi Angular distortion
<table>
<thead>
<tr>
<th>Abbr.</th>
<th>Full Name</th>
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<tbody>
<tr>
<td>CEB</td>
<td>Comité Euro-International du Béton</td>
</tr>
<tr>
<td>CPT</td>
<td>Russian cone penetration test</td>
</tr>
<tr>
<td>DIN</td>
<td>Deutsches Institut für Normung (The German Institute for Standardisation)</td>
</tr>
<tr>
<td>FIP</td>
<td>International Federation for Prestressing (Fédération Internationale de la Précontrainte)</td>
</tr>
<tr>
<td>HGT</td>
<td>Hugo Treffner Gymnasium</td>
</tr>
<tr>
<td>ICCROM</td>
<td>International Centre for the Study of the Preservation and Restoration of Cultural Property</td>
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<tr>
<td>ICOMOS</td>
<td>International Council on Monuments and Sites</td>
</tr>
<tr>
<td>ISCARSAH</td>
<td>International Scientific Committee on the Analysis and Restoration of Structures of Architectural Heritage</td>
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<tr>
<td>ISO</td>
<td>International Organisation for Standardisation</td>
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<tr>
<td>RakMK</td>
<td>Former National Building Code of Finland</td>
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<td>RakMK B3</td>
<td>Foundations. Regulations and Guidelines 2004</td>
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<td>RakMK B4</td>
<td>Concrete Structures. Guidelines 2005</td>
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<tr>
<td>SLS</td>
<td>Serviceability limit state</td>
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<td>ULS</td>
<td>Ultimate limit state</td>
</tr>
<tr>
<td>UNESCO</td>
<td>United Nations Educational Scientific and Cultural Organization</td>
</tr>
<tr>
<td>WST</td>
<td>Swedish weight sounding test</td>
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1 Introduction

1.1 Background to the study

A structure, its foundations, and the surrounding ground always interact with each other. The foundation system is an essential part of the building, technically as well as culturally. However, the foundation systems of historic buildings were long considered only as support structures of the built heritage. Foundation engineering is an unusual topic in Renaissance and Baroque literature about architecture. It was the less investigated topics from the historiographical point of view. It is only during the last century that foundation structures have began to be acknowledged not only as utilitarian, but also as historical heritage, with a cultural value of their own. Therefore, nowadays heritage structures should be considered and preserved for their own sake, not merely as supports for the rest of the historic material (ISO 13822:2010 Annex I). (Jappelli & Marconi 1997, Iwasaki & Tsatsanifos 2006).

Problematic foundations of historic buildings are most often made of wood. Wooden rafts or wood pile systems are typical of the foundation building tradition in the swampy regions of northern Europe. Such foundation systems work well when lying below the ground water level, but above it the wood begins to rot. Therefore, problems with wooden foundations are often caused by the lowering in time of the water table which surrounds them. The changing groundwater level, causing settlements on soft ground can lead to a situation where strengthening of the foundations becomes necessary in order to ensure the safety and preservation of an historic building. Rotten wood foundations can be replaced entirely or partly by new timber, by jointing the new timber members to the original parts which are still in good service condition. In the case of historic buildings, the strengthening method should affect the authenticity of the foundations as little as possible. (Iwasaki 2005, Iwasaki & Tsatsanifos 2006).

The establishment of international organisations and conservation principles concerning historic foundations was a result of increasing interest in the protection and preservation of cultural heritage which arose soon after the Second World War. The decision to establish an intergovernmental organisation for the conservation of cultural heritage was made at the General Conference of UNESCO in New Delhi in 1956. Consequently, the International Centre for the Study of the Preservation and Restoration of Cultural Property (ICCROM) was
founded in Rome in 1959. ICCROM is an intergovernmental organisation dedicated to the conservation of all types of cultural heritage, both movable and immovable.

The principles of the conservation and restoration of historic monuments were laid down in “The Athens Charter” (1931), as well as in “The Venice Charter” (1964). These were respectively agreed at the International Congresses of Architects and Technicians of Historic Monuments in Athens in 1931 and in Venice in 1964. The Congress in Venice was also the beginning of the International Council on Monuments and Sites (ICOMOS), which is a non-governmental organisation working for the conservation and protection of cultural heritage. The objective of the organisation is to co-operate globally and to bring together conservation specialists. ICOMOS also has several international scientific committees. The one acting as a forum and network for engineers involved in the restoration of building heritage is the International Scientific Committee on the Analysis and Restoration of Structures of Architectural Heritage (ISCARSAH) which was founded in 1996.

Within the main principles of “The Athens Charter” (1931) it was agreed that techniques of restoration should be based upon anastylosis (The Athens Charter 1931). This is a method of conservation that aims to keep the authenticity of the monuments. The word “anastylosis” is Greek and combines the words ana (re-establish or going back in time), stylos (column), and is (a suffix to make the noun). Anastylosis has been internationally recognised as a standard method to be applied for the preservation of heritage works. According to the principle of anastylosis, the original material of the foundation and construction methods proper to it should be used for restoration. The purpose is to preserve the authenticity of the monuments as long as the original materials are available the methods are possible to implement. In some cases degraded structures cannot be repaired solely with original materials and methods. This is why it may be necessary to add some internal reinforcement or protective measures to counteract the mechanical degradation of the structures. (Iwasaki 2005).

In “The Venice Charter” (1964) it was stated that the aim of the restoration of foundations is to preserve and reveal the historic and aesthetic value of the monuments (The Venice Charter 1964). The process should be based on respect for original materials and authentic documents. Only if traditional techniques prove inadequate, consolidation of a monument may be achieved using modern techniques. (Iwasaki 2005).
In addition to these two charters, the authenticity principle was concluded in “The Nara Document on Authenticity” (1994), drafted by the participants in the Nara Conference on Authenticity in Japan in 1994 (The Nara Document on Authenticity 1994). Authenticity has often been discussed for structures, but until recent times, soils and foundation structures have seldom been mentioned in terms of authenticity. In many cases the foundations of historic structures have been strengthened using the most economical means available, without taking the authenticity of the foundations into consideration. However, in historic buildings the foundation itself can be considered of historical significance or at least restored by using some methods and materials that are in harmony with regional building characteristics. According to the Nara Document, the authenticity should be determined in a manner respectful of cultures and heritage diversity, i.e. to include some variation in the conservation of heritage. (Tsatsanifos 2005, Iwasaki & Tsatsanifos 2006).

Recently, an international standard (ISO 13822:2010, Annex I), “Bases for design of structures – Assessment of existing structures”, was drafted with regard to the preservation of heritage structures. According to Annex 1 of the document, the structural performance and its value as a cultural resource are both to be taken into account in any decision pertaining to restoration work. To retain the integrity and authenticity of the structure, original materials and structural concepts should be preserved as far as possible.

Every historic structure has its own particular history and, in addition to physical conservation benefits, a restoration project can also have a high symbolic value. According to Avellan and Lange (2008), the symbolic value was significant in the case of strengthening the foundations of St. John’s Church in Tartu. The restoration work not only prevented the church from sinking but also contributed to reawakening faith and the spirit of place for the city of Tartu. The church was restored completely and became of symbolic value to the whole nation of Estonia. It was an affirmation of the national pride of its past, and an understanding of the necessity to conserve its built heritage for future generations, that gave rise to this work.

1.2 Objectives of the thesis

Underpinning is one of the few fields of engineering which offers the possibility of developing new working methods and where every location presents a particular challenge. The substructure, foundations, or soil circumstances vary at
each site. Underpinning by piles is a method, which is usually preferred in order to strengthen historic foundations. The piling methods used for the strengthening of St. John’s Church in Tartu include end-jacked close-ended steel tubes and pretested “end-jacked” drilled spiral piles.

One purpose of the present study is to develop a technical method with equipment for drilled spiral piles, which are drilled, pretested and then end-jacked to a dense layer. Functionality of the technique is verified by test piling. The other purpose is to clarify soil hardening by rejacking.

Nowadays there are several software programs to evaluate the interaction of soil, foundations, and structures. There is, however, a lack of examples of manual calculation methods by which it is possible to verify the other calculations, or which can be used directly for estimating soil-foundation interaction.

The main contribution of this thesis is to develop a simple design method based on the lower and upper bound theorem of the theory of plasticity, were a piled, floating strip foundation lying on frictional soil can be designed geotechnically and structurally using a single method based on the ultimate limit state (ULS) and serviceability limit state (SLS). The geotechnical part of the procedure takes into account the following requirements: bearing capacity, mean settlement, pile settlements and angular distortions. The structural part of the procedure takes into account the following requirements: admissible plastic rotations, end and field moments to respective rotations, end moments resulting from angular distortions, and control of cracking.

The thesis intends to prove that piled, floating strip foundations can be designed geotechnically and structurally using one method based on the ultimate limit state (ULS) and serviceability limit state (SLS).

1.3 Outlines of the thesis

The first part of the thesis contains a study on related literature. Chapter 2 presents historical foundation systems, reasons for their decay, and methods to strengthen various old foundations. The working methods which were used in St. John’s Church in Tartu in Estonia are described in detail in Chapter 3. As well as the normal use of jacked piles, the author of the thesis also devised a special screw pile and piling system which was implemented using hydraulic machines. The special pile was developed especially for the demands of a new situation, in this case for the wall line A which was without a roof as horizontal support and nearby the rebuilt columns. (Fig. 5) The author also developed a practical
pretesting and end-jacking procedure for drilled spiral piles. Chapter 4 presents the theoretical background of the author’s method for strip foundation lying on frictional soil.

Finally, Chapter 5 presents a new design method concerning the strengthening of line B in St. John’s Church (Figs. 3 and 5). In line B the interaction of the pile foundation with the foundation slab on piles is examined in ULS and SLS. The structural design of the slab is based on the kinematic method of plasticity.

1.4 Research assumptions

The strata under the foundations of the church consist of silt and silty sand. The following assumptions and limitations are valid in this thesis:

– The subsoil under the foundation is assumed to be cohesionless soil and to behave like ideal elastic plastic material. The resistance of the piles that were used consists of tip resistance and the stratum of the pile tip is dense or very dense silty sand.
– When settlements for verification of ductility are determined, soil material is assumed to be homogeneous, isotropic, and to follow the linear stress deformation graph up to the yield limit. Deformations (settlements) are assumed to take place in drained deformation state.
– When the ultimate limit load is being determined, soil material is assumed to behave as rigid-plastic material. The failure condition of soil material is the linear Mohr-Coulomb failure criterion.
– Ultimate contact pressure is assumed to be uniformly distributed to the effective width B of the strip foundation.
– The external loading on the strip foundations consists of point loads situated apart from each other.
– The soil pressure loading under the foundations consists of a uniformly distributed or triangular load.
– The shear resistance and punching resistance of reinforced concrete foundations are assumed not to be critical.
– The foundation is a strip foundation if the ratio of its length L to its width B is more than 3.
– The distance between separate parts of the strip foundation is more than B.
- The solution according to the upper bound theorem is considered correct when determining the mean ultimate contact pressure of the strip foundation, and the plastic moments of the strip foundations.
- The well-known settlement formula of Schleicher (1926) and Bordunov-Godunov (Tsytovich 1981) is valid.
- The settlement criterion can be checked by Schultze-Sherif and Burland – Burbidge methods.
- With regard to concrete structures, the so-called FIP recommendations (1984), and RakMK B4 are assumed to be known and accepted basic knowledge; the flexural rigidity of the strip foundation is assumed constant for the whole length of the strip foundation.
- The influence of the work joints is not taken into account separately.
- The Bernoulli-Navier hypothesis on planes, which remain planar is valid.
- Cracking is controlled by using high-bond bars spaced at maximum 200 mm and limiting the stress in the bars to 200 MPa.
2 Foundation systems of historic buildings

In this chapter historic foundation systems and methods for preserving them are presented. The process of renovating foundations is presented step by step, first by describing the reasons for strengthening them, then by reviewing typical methods used to strengthen them and concluding with a review of strengthening cases around the world.

2.1 Historic foundation techniques

Foundation systems have not always been appreciated as works of art, but simply as structures bearing buildings. According to Jappelli and Marconi (1997), it should be emphasised that ancient architects and builders had to cope with a number of difficulties concerning the relationship of buildings and the subsoil. These authors state, however, that Renaissance and Baroque monuments were not considered in their time as going below ground level, as if there were shame in mentioning the perceived “ugliness” of parts of the structure whose sole function was to preserve a building’s integrity. The most renowned treatise-writers, however, regarded foundation engineering as actual art, whose works had to be conceived exclusively by the most brilliant architects. They recognised the fact that without proper foundations structures would, and at many times did indeed collapse.

As many Renaissance and Baroque buildings are still in use today, and foundation investigations are seldom performed, there is only scarce data about the foundation systems that support them. However, many ancient foundation systems are well-known because of an exhaustive bibliography existing about them. Studies from archeological excavations revealing their forms and techniques have provided the main contributions to this bibliography. In Italy, especially in Rome, it was a widespread custom to reuse pre-existing foundations during the Renaissance age. Many old foundations were well preserved and available for reuse. It would have been considerably cheaper to use pre-existing foundations than to build new ones. Some of the old foundations were perfectly fit for reuse, but in several cases the attempt to save costs lead to the collapse of buildings. (Jappelli & Marconi 1997).

The science of foundation technology was initiated during the 17th century. Before this, knowledge of soil behaviour depended on experience within the particular localities. Builders relied essentially on know-how based on previous
experience of the behaviour of foundations, while not understanding the reasons behind it. The underpinning of structures depended on practical skills which were usually passed down from master to apprentice. In the 17th and 18th centuries the application of scientific thinking to foundation engineering began. The development of the theories of plasticity and elasticity provided mathematical tools to analyse various foundation problems. (Thorburn 1985).

In wooden pile foundations the length of the piles is critical, as it is the key parameter affecting the settlement and estimation of their bearing capacity. Until the 18th century most pile foundations were built using sharpened short piles with lengths ranging from 1.5 to 3.0 m. The piles were driven close to each other into the subsoil, without any consideration of its characteristics. In the Middle Ages some new pile construction types were developed, for example, horizontal timber, as well as a combination of piles and sleepers. (Ladjarevic & Goldschreider 1997).

In Venice, where most of the buildings and monuments were built before the 17th century, the two mostly commonly used foundation techniques were larch wooden slab foundations and pile foundations (Jappelli & Marconi 1997). Foundation systems were planned carefully in order to limit cracking and damage caused by water and the highly compressible superficial soil layers (Colombo and Colleselli 1997). Many Venetian buildings have foundations over a system of closely spaced timber piles with their top at the same level as the bottom of the canals. The piles usually rest against a raft of horizontally placed timber and are only 1–3 m long, but still managed over the centuries to spread the structural loads in a manner appropriate to the soft soils below. On top of these piles there is usually a stone or brick foundation. (Birck & Jerbo 1997).

Until the end of the 19th century foundation system materials were usually stone, brick, or wood. During the 19th century the Italians considered alder, olive, or English oak to be suitable pile material. All the major handbooks up until the first half of the 19th century proposed the use of the same kinds of woods, also adding chestnut, oak, and other long-lasting species to the list. In Italy, for instance, metal piles are only mentioned for the first time in treatises on architecture and engineering at the end of the 19th century. Soon afterwards reinforced concrete also emerged as a suitable material for fabricating piles. (Jappelli & Marconi 1997).
2.2 Reasons for strengthening foundations

Underpinning works are divided into three principal categories; conversion work, protection work, and remedial work. The majority of underpinning works of heritage structures are for protection, remedial in nature, or concern problems caused by settlement and soft ground (Iwasaki 2005). Different reasons for settlement are, for example, variation in the ground water level, natural soil consolidation, seismic events, or some human activities, such as traffic vibration (Lizzi 1985). Small piles are especially vulnerable to vibration caused by traffic and other human activities.

Burland & Standing (1997) stated that ground movements may result from load-induced settlements, slope instability, or subsidence resulting from various causes, such as a change in ground water level, adjacent or nearby underground works, or seasonal effects, such as swelling. Thus, as Gajo et al. (1997) stated, the present situation of historical structures is generally greatly affected by subsoil characteristics, foundation types, and soil-structure interaction. That is why geotechnical problems have plagued structures, in many instances over long periods of time, if not the entire existence of historic building, and should be well considered in the design of preservation projects.

A change in the ground water level is one typical reason for the decay of historic foundations. According to Izhar-ul-Haq (1997), in Pakistan the salinity carried by the rising tide is the most disastrous factor undermining the stability of the historic heritage. The salt causes the structural material to disintegrate and the high water table reduces the bearing capacity of the soil. In risk areas crustal movements and seismic activities may also cause the decay of buildings. For example, the historic centre of Tricarico in Southern Italy (Crespellani & Garzonio 1997) and Gubbio in north-central Italy (Bromhead et al. 1997) have both been damaged by several moderate earthquakes.

The historic city of Venice is a well-known example of the need to strengthen foundations. Originally, the buildings in Venice settled only minimally as a consequence of normal consolidation and creep from the structural load itself. However, during the last century the settlement process was accelerated by the collection of fresh water from aquifers below the city. Today, most of the city of Venice is threatened by rapid deterioration and large-scale restoration works are required to preserve the historic city. Mainly because of differences in settlement between the foundations of the inner and outer walls, considerable overall settlement has taken place. Venetian buildings are also exposed to several other
factors contributing to the deterioration of their foundations. Colombo and Colleselli (1997) listed several of these factors, namely the deterioration of building materials, insufficient maintenance work, mechanical scouring and erosion by water movement, high water and tides, and the frequently deepened canal bottoms, all of which have an adverse effect on the foundations. (Birck & Jerbo 1997, Colombo & Colleselli 1997).

It is only seldom that damage occurring in historic buildings can be attributed to one cause only. Heise et al. (1997) remind us that damage is usually of a complex nature. Harm to the integrity of buildings can be the result where soil mechanics play a role, as well as when the hydrology or change in soil consistency affects the foundations. Damage can also occur simply because of inadequate foundations or poor construction or simply because the constructions have aged. Sometimes long-term geological processes such as weathering, slope mass movements, and sub-surface erosion can cause damage to structures. According to Heise et al. (1997), e.g. the ageing process of the foundations of Heidecksburg Castle in Germany occurred because of differential movements, slides, blockages, and deformations, as well as a combination of a host of other factors. In addition these were leaching off, weathering, loosening, softening, and swelling, as well as underground erosion and suffusion resulting from water penetrating into fractured rock masses.

The wooden foundations of historic buildings are especially vulnerable to damage. They do not have the same longevity in changing environmental conditions as that of stone or brick foundations. As a result of natural ageing, external influences, and chemical and biological processes, wood can lose its strength and load-bearing capacity. Kept below the groundwater table in an anaerobic and cool environment, wooden foundations may remain stable for centuries. However, even temporary changes in the environment near the wooden foundations may cause rapid and severe decay and impact negatively on the whole foundation system. (Heise et al. 1997, Birck & Jerbo 1997).

The biological attackers of wooden foundation systems are fungi and bacteria. The most common types of damage found in the foundations of historic structures are loss of material and strength as a result of fungi. Under certain circumstances, fungal attack may lead to a complete loss of load-bearing capacity within, only a few years. When the top of the pile becomes exposed in the unsaturated soil above the groundwater table, the environment around it, as well as inside it, becomes aerobic. In aerobic conditions the wood will be drained and the voids become filled with air. In some cases, as a result of a fluctuating groundwater
level, the top of the pile can alternatively be in an aerobic or anaerobic state. (Heise et al. 1997, Birck & Jerbo 1997).

### 2.3 Investigations prior to strengthening foundations

The safe execution and economical design of underpinning works require sound theoretical knowledge of foundation engineering, as well as wide experience of the actual behaviour of constructions and soil. Before the foundations are strengthened, careful investigative work should be carried out to verify the actual conditions of load-bearing components. In historic buildings the materials of load-bearing walls, piers, pillars, and buttresses are often of composite construction. According to historical records, in medieval times too great reliance was placed on the supportive ability of rubble masonry contained by relatively thin ashlar facing stones. (Thorburn 1985).

Long-term monitoring of an historic structure can be of great benefit in assessing the need for stabilisation of a building. Many buildings have been monitored for years. According to Burland and Standing (1997), the famous leaning Tower of Pisa in Italy has been monitored since 1911, by measuring the inclination of the tower by means of a theodolite. To understand the behaviour of the tower, the study of its movements has been important. By the 1990s the foundations of the tower had inclined over 5 degrees due south, as its inclination was increasing during most of the 20th century. (Burland & Standing 1997).

Unlike the Tower of Pisa, in many cases strengthening work must proceed as quickly as possible to ensure the stability of the structure. The reinforcement that is installed must often be effective immediately in order to counter any further movement of the building. When time is limited and investigations of the foundations of the damaged building are restricted, the judgements that are required may be based on minimal evidence. Should the strengthening work applied directly on the structures be potentially damaging, an intermediary work may prove necessary. For example, when piles are being installed, a needle beam can be inserted to support the load temporarily while the existing footing is undermined. A new beam can then be inserted to distribute the load to the new pile foundation. In addition, pockets can be made to allow the insertion of piling jacks. (Cole 1985).
2.4 Discussion of various methods

Several techniques are available nowadays for strengthening the foundations in historic buildings. The development of new strengthening techniques has allowed the adoption of a wide range of solutions. The techniques used most frequently are micropiles, jet grouting, cement and chemical grouting and strengthening with cemented bars (Rodríguez Ortiz & Monteverde 1997). The anastylosis principle should be observed in choosing the most appropriate method for preserving heritage structures.

The best method to strengthen the foundations of a heritage structure is underpinning. The root pile method is considered the second best option to use. Piling from the side of the foundations is not recommended. Iwasaki (2005) has compared several methods for underpinning historic buildings. The methods evaluated were new wooden rafts, grouting, minipiles, and underpinning with jacked piles. According to Iwasaki, underpinning with jacked piles is a preferred method for several reasons, including the use of man-power, simplicity, and cost. The method requires a minimum working space and is reliable. However, as far as the preservation of the authenticity of the foundations is concerned, Iwasaki recommended that whenever feasible, reconstruction of the original wooden raft pile foundation should be the preferred solution.

The analysis of underpinning piles is a complex problem because it requires the behaviour of the original and new foundation systems to be combined. Georgiadis and Anagnostopoulos (1997) state that engineers tend not to take account of the contribution of the old foundations, and the new underpinning piles carry the entire load of the structure, instead of the old and new foundations working in tandem. This usually results in the installation of unnecessary piles, which may cause excessive disturbance to an old structure.

The methods used for computing structures and soil circumstances may have advantages but in some instances ancient foundation solutions based on practical skills work better than newer solutions. In many practical cases it is important to understand how old piled foundation structures function. A fictitious example of an insufficient strengthening technique is shown in Fig. 1. Inclined piles, such as the old piles of the building in Fig. 1, were well known in previous centuries (Jappelli & Marconi 1997). Actions such as wind load, earth pressure, and nowadays traffic load causing vibrations and break loads may require the transfer of horizontal forces to inclined piles. The need for inclined piles also depends on
the number of floors the building has and the undrained shear strength of the soil. (Avellan & Etelälahti 1983).

![Diagram of a fictitious example of an unsuccessful case of strengthening.]

**2.4.1 Jacked piles**

Underpinning with jacked piles is a widely used method suitable for a variety of foundation cases (White 1975). Installing the piles in short stubs, e.g. by using a hydraulic jack, is a suitable method if the headroom is less than that required for conventional equipment. The cost of using short segments is usually greater than conventional piling because of the additional cost of the joints themselves and the time required to make them. However, in many cases this is the best underpinning option. It can be used in challenging circumstances, but it is not applicable when there are boulders, old foundations, or other obstructions that make penetration of the soil with segmented piles difficult. (Lizzi 1985) One alternative would be to construct bored piles using very low-headroom tripod rigs and percussion boring.
equipment. This approach may also be replaced by minipiling methods. (Cole 1985).

2.4.2 Small piles

Piles with a diameter of less than 300 mm are usually considered small piles or micropiles (Cole 1985). Micropiling is an old technique which was originally adopted for wood-driven piles. During recent decades micropiling has been developed with improvements in the use of hollow high-resistance steel tubes and high-pressure grout injection techniques. Various root piles and micropiles are especially suited for underpinning in circumstances where the construction space for piles is restricted. (Pinto et al. 2001).

Root piles, which are small diameter cast-in-place piles, were invented in the 1950s to solve the problems for which conventional piling systems proved to be inadequate (Lizzi 1985). According to Lizzi (1985), root piles can be used at any site and in any subsoil conditions. They can be drilled into any soil, no matter how old the foundation or boulders it may contain. The most significant feature of root piles is their capability to respond quickly to any movement, however slight, of the building. The method is not suitable for Scandinavian soft clays, however. In clayey soils small piles can easily buckle, a factor which should be considered when small pile foundations are being designed.

About 20 years after the introduction of root piles, micropiles, consisting essentially of heavy metal pipes, were proposed (Lizzi 1997).

New techniques make it possible for steel micropiles of a diameter less than 130 mm to carry axial service loads greater than 700 kN. Micropiles are made of steel and are cemented into the subsoil. They can bear considerably higher loads than root piles. Nevertheless, the adhesion between the pipe and the soil and the risk of corrosion must be considered when micropiles are being used. (Pinto et al. 2001, Lizzi 1985).

2.4.3 Jet grouting

In addition to the strengthening of the foundations, the stability of historic structures can be improved by strengthening the soil. The improvement of soils can be achieved with different injection techniques, such as permeation grouting, displacement grouting, encapsulation grouting, or jet grouting. These techniques are based on principles of the reduction of permeability and an increase in the
strength of the soil. Jet grouting, meaning breaking up the soil by means of a jet of water or grout, has been applied worldwide since the 1980s. In jet grouting the soil is mixed in place with a stabilising mixture under very high nozzle pressures, generally 20 to 50 MPa. In some cases the nozzle pressure may go as high as 70 MPa. The soil can be mixed in place with a suitable grout. In an alternative procedure soft fine-grained soils are partly removed by air-water jetting and then replaced by the grout. Jet grouting techniques make it possible to treat any type of soil even by means of simple cement grouts, but special attention should be paid to the concrete, because of sulphate, which can lead to problems (since the presence of sulphate makes the volume of the concrete increase). (Pinto et al. 2001, Perelli Cippo & Tornaghi 1985).

2.5 Case studies of strengthening the foundations of historic buildings

2.5.1 Underpinning with large piles

Every case of strengthening the foundations of an historic building is a unique one. Chartes (1997) reported on underpinning works for the Round Tower at Windsor Castle in Berkshire, United Kingdom. The tower, built in the 12th century, is located on top of a mound of chalk fill. In 1988, foundation movements occurred under the tower, causing serious damage to the building, such as cracks in the walls and broken windows. The likely reason for the movement was a circular slip of material within the mound, probably as a result of very heavy rainfall before the event occurred. The damage to the tower caused by the sudden displacement of its foundation required the installation of ring beams adjacent to the inside and outside of the existing medieval masonry walls. Needles were installed under the walls. Building loads were transferred from the needles to the ring beams through jacking points which allowed the controlled transfer of the building loads onto the piles. 600-mm-diameter bored cast-in-place concrete piles were chosen because of space and access limitations. They were formed with tripod rigs and reached a depth of about 25 m. As for the main tower wall, the Round Tower was underpinned on a foundation beam supported on piles.

Fadeev et al. (1997) reported on the strengthening of the castle tower in Vyborg in Russia. The tower of the Vyborg castle, founded in the 13th century, had been heavily damaged by subvertical fissures and was in danger of full failure.
Numerous, mainly subvertical cracks had developed in the tower walls. To avoid further damage the tower was strengthened by a series of measures. At soil surface level, a reinforced concrete bandage was constructed around the tower to prevent lateral creeping. The bandage itself was fastened to the rock by 12 cast piles. Additionally, two prestressed anchor belts were installed to compress the tower wall. Cracks at the outer surface were tightened by mortar and walls injected with lime-cement compatible with the ancient mortar mixture.

Segovia et al. (1997) investigated the geotechnical conditions, settlement, and structural safety of the temple of Saint Augustine in Mexico. The temple, built in the 16th century, is one of the most important religious monuments of Mexico City, and is situated over highly deformable subsoil. The safety of the structure had become quite uncertain as a consequence of damage brought about by differential settlements. The structure is a composed of a grid of stone masonry arches resting on wooden stakes. The longer arches join the columns in the transverse direction, and the smaller arches run along the longitudinal axis of the building. The structure was measured to have settled approximately 4.0% towards the west, which caused the columns to deviate from their vertical axis and also cracked the walls and vaults. To prevent further harm, the structure was underpinned with precast reinforced concrete piles supported on deep deposits. The piles were jacked with a metal casing to prevent negative friction from the further loading of the piles. The piles support the entire load of the building and were equipped with reaction frames to adjust to them. The piles on the western edge of the structure were also fixed and together function as a hinge. The temple should re-level itself in 30 years or more because the regional settlements will slowly bring down the eastern side, while the western edge remains fixed.

The present residence of the German Bundestag, the “Reichstag”-building in Berlin, was renovated after the reunification of Germany. According to Quick et al. (1997), during the original construction of the old foundations in 1884, the northern parts of the building required soil improvement and extensive supporting measures, including 3,000 timber piles. The groundwater level is between 1 and 2 metres below the bottom of the basement, but in the 1930s it was lowered to ten metres over a period of six years because of the construction of a suburban railway line. Examinations of soil stratification, macro- and microscopic laboratory tests, and load tests of the timber piles were carried out to estimate the long-term serviceability of the foundations. During the renovations in the 1990s the cellar of the building was lowered and this reconstruction influenced the loading conditions considerably. A total of 500 MN of structural load was
removed and 590 MN of new structural load was added. The plenary hall, with its glass-dome in the central area, was founded on 90 new shafts and foot-grouted bored piles which are less affected by settlement.

Gendel' (1984) reported on the underpinnings of the foundations of the Amusement Palace in Moscow. The palace was built in the 17th century and over the years had settled more than one metre. As a result of this settlement the building had experienced significant deformations, such as cracks, up to 40 mm wide. Below, the foundations, recesses had formed, because of the rotting pile heads. The underpinning work began in 1978 after several methods that had been proposed were rejected and finally an experiment with two tubular piles proved successful. The building was underpinned with steel tubular piles, 351 mm in diameter and with a surface thickness of 12 mm. The purpose was to conduct the work without interrupting the normal functions of the palace. Each pile was installed in a row, following the previous pile, and throughout the work no more than two piles were sunk simultaneously into their positions in one area of the building. After the piles had been jacked into their places, their casings were filled with grade 30 MPa concrete. Additional designs, for example as transverse beams or struts, serving to take the load of the building temporarily, were not used, and for this reason the building settled by up to 20 mm during the underpinning work.

In Finland Avellan (2009) dealt with the strengthening and underpinning of the foundations for the buildings of the Ministry of Employment and the Economy in Helsinki. The buildings, originally constructed at the beginning of the 19th century, had foundations partly lying on layered silty sand and silty clay, and partly on moraine or rock. Because of the high vehicular traffic vibration velocity (10 mm/s) near the buildings, the foundations had to be strengthened. On the sides bounded by Alexander Street and Maria Street, the walls were underpinned by jetted Mega-piles with a section of 300 x 300 mm². On the side of the common courtyard and adjacent wall, in some cases underpinnings were done with Mega-piles. The additional bearing capacity needed for these special piles was obtained by injecting the pile tips with stiffening gel created by mixing sodium silicate with calcium chloride. For the part of the building resting on a smooth area, the walls were underpinned with jacked steel piles and concrete Mega-piles with a section of 300 x 300 mm². As for the wall parts of the building which were resting on a hard area, they were panelled (concrete cast in place). Every pile was subsequently tested for its characteristic load and the results proved more than satisfactory.
2.5.2 Underpinning with small piles

Avellan et al. (2005) reported on strengthening the foundations of the main building of Tartu University, built in 1805–1807. The underpinning work was made in 1995 and was the second stage of the entire strengthening project and covered the central portion of the building. The northern part had been strengthened earlier, by the end of 1977, with bored piles, but the middle part of the building was still slowly sinking since the ground water level had dropped below the level of the timber raft structures. The strengthening work was carried out by means of the jacked pile method. Steel tube piles were jacked down under the structure, using the structure itself as a counterweight. Reinforced concrete beams were constructed on both sides of the thick external wall of the main entrance. The concrete beams were pressed with a post-tensioned anchorage against the structure before piling. Inside the building massive pillars were supported by steel beams and jacked piles. A field load test was performed on each pile and the rate of settlement was measured. Archaeological diggings were carried out at the same time as the strengthening work.

In the 1980s Avellan and Nissinen (2007) dealt with the strengthening and underpinning of the foundations of the main building of the Ministry of Foreign Affairs of Finland, located in Helsinki. The building, built in the 1820’s, was built on massive stone foundations lying on moraine or rock. The purpose of the underpinning was to realise a new basement in the eastern part of the building and the old basement floor level had to be lowered in other parts. The walls of the old naval barracks were shored up during the reconstruction, underpinning operations, rock blasting, and excavation of the basement. The bearing structures in the underpinning consisted of drilled steel piles, prestressed rock- and soil anchors, bolting stone walls, and concrete arches. During the construction stages the weight of the substructure decreased temporarily because of the demolition of old ceilings. From the structural point of view, the building had three load-bearing walls and two sectors, the northern wall on the sea side, the middle “kernwall”, and the southern wall on the earth side. Throughout the work the horizontal displacements of the middle wall remained small because loads were taken temporarily by steel piles which were grouted in rock and the loads on the beams were jacked against deflection. Possible dangerous situations were avoided by monitoring displacements at regular intervals during the work.

Goldschreider et al. (1997) reported on the stabilisation of the foundations of Schwerin Castle in Germany using soft micropiles. Most of the castle buildings,
located on an island, were built during the 19th century. They rest partly on flat foundations from earlier buildings, standing on the remains of the old ramparts or fill, and partly on pile foundations from the 19th century. The buildings had exhibited different settlements and corresponding settlement cracks as a result of the creep of the soft layer and rotting of the wood in the ground. This was especially noticeable in the old ramparts, which were above water level. Using old plans, test pits, time-lapse levelling, and comparisons with calculated creep settlements, Goldschreider et al. (1997) were able to detect the type of construction and the condition of each of the foundations. A stabilisation of the foundations was only recommended where the settlement was caused by rotting wood above the ground water level. For all other constructions, it was recommended that the foundations remain untreated, even if future settlements would occur. For stabilisation two options prevailed: micro-piles with an enlarged foot to soften the bearing behaviour or an adjusting device at the head to make future settlements uniform. The general outer bearing behaviour of the micropiles with an enlarged foot was first examined in a pile load test.

According to Źmudziński and Karczmarczyk (1997), the Juliusz Słowacki theatre in Cracow in Poland was underpinned because of the modernisation of the stage and the addition of technical facilities. The theatre was originally built in 1891–1893, and to improve its technical facilities it was decided to build two-storey rooms under the roadway surrounding the theatre. For this reason the foundations of the outer walls of the main building and the contour of the lift designed for container transport of stage props were underpinned. Because of the difficulty of access and high unitary pressure values, the foundation of the eastern wall was reinforced by means of micropiles. The method used also made it possible for the theatre to perform its functions during the underpinning works. The eastern part of the theatre also had cracks which appeared shortly after the building was put into use. To ensure the functionality of the micropiles two test loadings were performed. In load tests each pile was loaded in increments from 58 kN up to 400 kN, with the load being kept constant for 10 to 30 minutes. Subsequently, the pile was unloaded and after this the load was gradually increased to about 750 kN.

Bustamante et al. (1997) reported on the underpinning of the Pont de Pierre Bridge, which is the oldest bridge in Bordeaux. The Pont de Pierre was built by Napoleon and its structure of brick and masonry is supported by piers resting on lime concrete footings. These are built on a timber platform supported by 250 driven timber piles 300 mm in diameter. The bridge had started to settle down
soon after its completion in the 1820s. According to investigations, a number of piers had been settling significantly faster than most others. The underpinning work was performed in the 1990s. An investigation of the bridge structure and subsoil was carried out and after different strengthening methods had been considered, underpinning by micropiling was adopted. After a test micropile had demonstrated that the pile type selected performed very well, a total of 16 micropiles were installed with an allowable bearing capacity of 150 tons each. The piles were driven through masonry with a bond length of 8 m into the calcareous marl.

### 2.5.3 Strengthening with jet grouting

Bustamante *et al.* (1997) studied the case of strengthening a viaduct by jet grouting. The Levallois viaduct, located west of Paris in France, had suffered from differential settlements since its completion during the 1920s. The structure was originally built on top of driven piles 400–800 mm in diameter. The arches of the viaduct had already been strengthened by a consolidation technique which consisted of equipping the viaduct with a prefabricated and prestressed counter-vault, as well as pressure-grouting the cracks using grouts with expanding additives. Despite this, the differential settlements of the viaduct did not stop, but additional damage affected the structure. It was found that certain piles of the viaduct were simply too short, ending right above a layer of hard marls with insufficient support capacity. To increase the bearing capacity of the existing piles the 120-m-long viaduct was jet-grouted. Following the pile and jet-grouting and, column load tests, the strengthening was performed without traffic on the viaduct. A total of 156 jet columns 800 mm in diameter were installed. This appeared to be a successful operation since no significant settlements were observed during the following two years after its implementation.
3 Strengthening the foundations of St. John’s Church in Tartu

3.1 History of St. John’s Church

Among the many examples of medieval architecture in Estonia, St. John’s Church in Tartu remains an outstanding piece of art within the European context. It is the most prominent building in the Gothic style in the country, and is unique for its terracotta sculptures and decorative details. More than 1000 sculptures have been made for this church and about half of them are preserved.

It is understood through written records that in 1323 the Church, or, to be more precise, a congregation existed. The fragments of wooden buildings found on the site have been considered the earliest evidence of an existing church. From the evidence of archaeological findings, the building of the stone church can firmly be established as dating from the end of the 13th century or the 14th at the latest. The church was not built according to a uniform plan and acquired its final medieval shape after several changes of plan, reconstructions, and also catastrophes. The church was originally Catholic; however, from the time of the Reformation, it became a Protestant church for German-speakers.

The story of the church in the post-medieval period is a history of multiple destructions and rebuilding, with many of the original features being lost during the process. The church suffered seriously in the Northern War in 1708. Probably the upper part of the tower fell on the nave and the vaults of the nave and as a result the choir caved in. In the course of urgent repairs the nave was built lower than previously. In 1899–1904 the church fronts were rebuilt under the direction of the Latvian architect W. Bockslaff. During World War II, Estonian became the language used in services and in 1944 the church burned down and remained in ruins for the whole period of the Soviet occupation. During the following decades more than one plan for the restoration of the church was drawn up. Some practical measures connected to the restoration of the church were already taken during the late Soviet era. Indeed, some surveying of the building was done in the 1970s. In 1989 a team of Polish restorers embarked on the task, but owing to the changing economic situation the work stopped in its initial stage. (Alttoa 1994, Avellan & Lange 2008). Fig. 2 shows the state of St. John’s Church before the strengthening work in the 1990s.
After the re-establishment of the Republic of Estonia, the restoration continued under the command and financial support of the Ministry of Culture. Other important supporters were the Church of the North-Elbe and the City of Lüneburg. To improve and quicken the restoration process the Tartu St. John’s Church Foundation was established in 1992. In 1995 the Lutheran congregation re-emerged. (Avellan & Lange 2008).

Although the church has undergone destruction and reconstruction more than once, its original shape from the medieval period can easily be perceived even now. The three-aisled body of the basilica, with its powerful west tower, is linked to an elongated choir with a polygonal apse, the vestry being on the north side of the choir. On the south side of the nave stood the so-called Lübeck Chapel – a reminder of the times when Tartu (Dorpat), as a Hanseatic city, was an intermediary in the trade between Lübeck and Russia. The wooden rafts below the tower supporting the foundations of the church probably date from the end of the 13th century. Other parts of the church also rest on wooden rafts, except the choir, which is on a mound and does not have supporting rafts. A plan and section of St. John’s Church are shown in Fig. 3.
In the first stage of building, the choir of the church was established on a foundation of sand and granite stones. Initially the choir possessed a straight apse, which, after subsequent re-buildings, has become an octagonal one. During the second stage the central nave and tower were added. Since Tartu is located on relative loose ground, the stability of the building was achieved by having wooden rafts underneath the stone foundations.

The dominant feature of the building is its massive western tower. According to its plan, St. John’s Church resembles St. Jacob’s Church in Torun (Thorn), Poland. Both buildings possess a central nave which is not rectangular but trapezoid. In connection with this fact, it is important to stress that both churches belonged to the Diocese of Riga, and both were built with the involvement of the Teutonic Order.

Today the church works as a multifunctional building: its primary function is a working church. The other functions linked to its main use are as a cultural monument open to the public with exhibited terracotta originals, as well as a concert hall. The association of St. John’s Church and the University
congregation combined their ecclesiastical spirit with the academic spirit of the University of Tartu (Tartu Ülikool, -originally Academia Dorpatensis), founded in 1632.

### 3.2 Existing foundations of the church

The existing foundations of the church are of massive stones, which themselves sit on wooden rafts. The height of the stone setting is about 3 m. On the lowest parts of the building, situated in the south-western and south-eastern corners, the stone setting is 1.5–2.0 m high. On the top layer these stones are joined together with mortar, but at the lower level the joints are filled with sand. The double wooden raft under the stone setting was built from Ø 30–40 cm wood trunks.

The church tower is made of bricks; its outside dimensions are 12.5 m by 14 m, with a height of 38 m from ground level, and it has an estimated weight of approximately 5500 tons. The base of the church tower is divided into four pillars (P1, P2, P3, and P4 in Fig. 5), two of them measuring approximately 7.5 m x 3 m and the two others 4 m x 3.5 m. Calculated for each column, the stress applied to the stone layers is 685 kN/m². (Avellan & Lange 1997).

The ground under the church consists of a variety of different soil layers. The thickness of the many strata in different sectors is also quite variable. The classification of coarse grained soils is based on samples and on penetration tests (Bergdahl & Eriksson 1983). The approximate thicknesses of the soil layers under the wooden rafts are as given in Fig. 4 and Appendix 2:

- 4.0...5.0 m layer of loose / sandy silt
- 3.5...5.8 m layer of dense to very dense silty sand
- 0.4...0.8 m layer of middle-gauge dense sand
- 1.8...5.5 m layer of dense clayey silt
- layer of very dense gravel.

Fig. 4 shows the results of the Russian cone penetration test (CPT) and Swedish weight sounding test (WST) at the sample point near the church wall. The location of the sample point close to one corner of the church is shown in Fig. 5 of Section 3.3.
3.3 Reasons for strengthening the foundations

The consolidation process of the loose lime-silt layer has surely ended because of its thickness (5 m) and time-lapse (more than five hundred years). The outside earth surface of the Church has risen in the past because of the addition of “cultural layers”. It was during recent decades that the building began to sink
because of the lowering of the ground water table. In the last few years before the strengthening work was started, the water level had dropped below the level of the wooden rafts. As a result, the wooden rafts had begun to rot, thus accelerating the sinking process. The settlement map in the church area for the period from 1963 to 1987 is shown in Fig. 5 as contour lines.

If the strengthening of these foundations had not been implemented, the overall structure would have been threatened with serious damage over the coming years. The restoration work of the church building began with underpinning the foundations in order to stop the ongoing sinking process as a result of the lowering of the ground water level. The soil under the foundations has consolidated, as have the foundations lying on wooden rafts, over a period of hundreds of years. The thickness of the loose silt layer is a maximum of 5 m. (Avellan & Maanas 2001).
To stop the sinking of the church, the foundations were strengthened during the years 1993–96. The work began with strengthening the foundations of the tower and was mainly executed in 1993–94. The last supporting structures and concreting work for the tower were executed at the beginning of 1995. The other parts of the church foundations were strengthened in 1995–96. The foundations of the choir were not underpinned since they lie on hard soil and were not considered to be in immediate need of strengthening.

3.4 Underpinning methods implemented at St. John’s Church

The foundations of the church were underpinned by piling, using jacked piles and spiral drilled piles. In total 287 pile segments were installed. The location of each pile is shown in the layout drawings of Appendix 1 in Figs. 1 and 2. The location of the lines and sections is also presented in Fig. 5. These pile types were employed because in its present state the building would not tolerate the vibrations of driven piles. Under the tower it would have been impossible to use hammered piles or root piles because of the large size and weight of the tower. The layer of dense to very dense silty sand starting at approximately 6 m below the old foundation level was chosen as the bearing stratum. The reason for this choice was that the first 4.0–5.0 m thick layer of loose sandy silt did not have enough bearing capacity, Fig. 4. Additionally, the old block foundation could not have withstood the greater forces which would have been needed to jack piles through the compact silty sand layer.

3.4.1 Jacked piles

The type of piles chosen was steel pipe piles Ø 218 x 10 and Ø 140 x 8 with closed toes. The piles were jacked down under the structure in segments of lengths between 0.8 m and 1.0 m, using the weight of the structure as a counterforce in the jacking process. The pile segments were joined by welding. Piles were filled with concrete and wedged against the old structure. Before wedging, every jacked pile was end-jacked and load-tested to at least 1.5 times the characteristic load. The allowable maximum loads were 400 kN for the Ø 218 x 10 piles and 150 kN for the Ø 140 x 8 ones.
3.4.2 Drilled spiral piles Ø 218×10

It is known that earlier piles torqued with separate screw tips using manpower (Lille 1901).

Drilled spiral piles were used in those parts of the structure where jacked piles could not be used because of the absence of a counter-weight as in line B and the general lack of good condition and form of the old structure as in line A. The drilled spiral piles developed for the purpose were of steel and had closed toes. The length of the first segment was 1160 mm and dimensions of the pile was $210 \times 10$ mm$^2$ with spiral ridge (thread) around it, as shown in Fig. 8. The drawing of the first segment of the pile is in Appendix 3. The piles were embedded in the soil by twirling. The joints of the spiral drilled piles were made by welding the parts together and the completed piles were filled with concrete. Load testing with a minimum of 1.5 times the characteristic load was performed for every pile. The maximum allowable load on the spiral drilled piles that were installed was 375 kN. The total length of the piles on axis A varied from 5.5 m to 6.5 m.

A practical and simple technique and equipment was devised to enable the assembly of the drilled spiral piles by screwing in very limited workspaces. All the parts of the necessary equipment could be carried by workmen. The screw equipment is shown in Fig. 6. Section drawings of the equipment are presented in Appendix 1 in Figs. 3 and 4.
3.4.3 Pile test in the Hugo Treffner Gymnasium

To ensure the functionality of the drilled spiral piles to be installed, a pile test was carried out near the church in August 1994. The pile specimen was situated about 5 m from line E indicated in Fig. 1 of Appendix 1. The location is shown in Fig. 7 which presents the test pile in the Hugo Treffner Gymnasium between jacked piles with numbers 13 and 15. A cross-section of the test pile and weight sounding test results are shown in Fig. 7.

The test piling lasted about 20 hours, which almost fulfils the recommendation of 24 hours. The test method used in creep load tests is described in ISSMFE Axial Pile Loading Test-Part 1: Static Loading, 1985. Before the pile was installed, its entry place was water-jetted. The diameter of the jetted hole was about 2 ½ inches. The test pile was loaded gradually so that fixed loads were kept for 15 to 20 minutes before gradually adding more load to the pile. After approximately an hour from the beginning of the test piling, an hour’s break took place. During the break the pressure of the hydraulic pump decreased from 350 bar to 320 bar without any significant settlement difference. The testing was also terminated for the night and resumed the next morning. During the night break of 18 hours about 70 tons of the load was lost due to lack of the pressure...
control and internal leaks in the hydraulic system. The settlement results of the pile test are shown in Table 1 and in Fig. 9.

The test with a drilled spiral pile in the Hugo Treffner Gymnasium was performed to verify the creep load of the pile. The creep results of the test are pretested in Fig. 10.

Fig. 7. Location of the drilled spiral test pile (TP) situated in the School of H. Treffner.

The scope of the work of the test pile included (Fig. 8).

- jetting a hole in the earth \( \phi \approx 2 \frac{1}{2}'' \),
- water pressure \( p = 10 \ldots 12 \) bar,
- drilling the pile as deep as possible,
- jacking and testing the pile.
Fig. 8. Section of drilled spiral test pile (TP No. 19) and Swedish weight sounding test results.
Table 1. Settlement results of the tested drilled spiral pile No. 19 in the HGT.

<table>
<thead>
<tr>
<th>Pressure (bar)</th>
<th>Load (kN)</th>
<th>Holding time (min)</th>
<th>Total settlement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>85</td>
<td>121</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>100</td>
<td>143</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>140</td>
<td>200</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>170</td>
<td>243</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>210</td>
<td>300</td>
<td>0</td>
<td>9</td>
</tr>
<tr>
<td>240</td>
<td>343</td>
<td>0</td>
<td>19</td>
</tr>
<tr>
<td>280</td>
<td>400</td>
<td>0</td>
<td>34</td>
</tr>
<tr>
<td>300</td>
<td>429</td>
<td>15</td>
<td>51</td>
</tr>
<tr>
<td>315</td>
<td>450</td>
<td>15</td>
<td>63</td>
</tr>
<tr>
<td>330</td>
<td>472</td>
<td>20</td>
<td>74</td>
</tr>
<tr>
<td>350</td>
<td>500</td>
<td>5</td>
<td>83</td>
</tr>
<tr>
<td>A break of one hour</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>320</td>
<td>457</td>
<td>-</td>
<td>80</td>
</tr>
<tr>
<td>350</td>
<td>500</td>
<td>5</td>
<td>91</td>
</tr>
<tr>
<td>370</td>
<td>529</td>
<td>15</td>
<td>113</td>
</tr>
<tr>
<td>385</td>
<td>550</td>
<td>20</td>
<td>135</td>
</tr>
<tr>
<td>400</td>
<td>572</td>
<td>20</td>
<td>149</td>
</tr>
<tr>
<td>420</td>
<td>600</td>
<td>60</td>
<td>171</td>
</tr>
<tr>
<td>A break of 18 hours</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>370</td>
<td>529</td>
<td>-</td>
<td>170</td>
</tr>
<tr>
<td>375</td>
<td>536</td>
<td>0</td>
<td>170,5</td>
</tr>
<tr>
<td>420</td>
<td>600</td>
<td>10</td>
<td>171</td>
</tr>
<tr>
<td>440</td>
<td>629</td>
<td>12</td>
<td>173</td>
</tr>
<tr>
<td>455</td>
<td>650</td>
<td>17</td>
<td>183</td>
</tr>
</tbody>
</table>

After 18 hours a test load of 600 kN indicates no significant creep effect. The design creep load \( Q_c \) defined for the pile on account of the test is 525 kN as indicated in Fig. 10. The allowable load specified for the drilled spiral piles in line A of the church (the continuous wall) is 375 kN. In line B and C the maximum allowable load used was 300 kN as shown in Figs. 1 and 2 of Appendix 1.
Fig. 9. The pile test of a drilled spiral pile in Hugo Treffner Gymnasium. Total settlement, total load and effective level of water jetting in test.

The creep load is defined without pretesting and end-jacking procedures, Fig. 13, which were determined after testing the piles Nos. 220 and 222, Fig. 14.
Fig. 10. Creep velocity in the creep load test (pile No. 19) in the HGT. Qc is the load at which the curvature of the diagram corresponds to the minimum radius of the curvature.
3.5 Strengthening operations in various areas of the church

3.5.1 Work description of drilled spiral piles

The underpinnings at line A/2–7 and line C/4–7 were made with drilled spiral piles. The location of these lines is shown in Fig. 5 and in more details in the specialty report by Avellan & Nissinen (1997). The walls were pre-injected with a water-cement-peipsisand mixture. In the first stage the pile locations were jetted and the piles were screwed as deep as possible counting on the reliability of the torque moment of the equipment. Then reinforced concrete beams were cast on both sides of the wall above the piles, against an injected stone composition; see Fig. 11. The concrete beams were forced against the old structure with a prestressed anchorage. As the timber material of the plank foundations rots, all the vertical loads will gradually be taken on by the new piles.

Fig. 11. Section of line A showing the injected stone composition, lower and upper pretensioned anchors and concrete beams. Chiselling was required prior to fabricating the concrete beams that can take the load from the superstructure to the piles.
At line A, the piles also have the lateral loads \((F_{yk})\) and moments \((M_{yk})\) from the rebuilt arch. The lateral load is \(F_{yk} = 51.0\) kN and the moment \(M_{yk} = 624.5\) kNm. Associated to line A there are three piles of 340 kN allowable load each, and two piles of 360 kN outside the line as combined. The lateral load of 10.2 kN for each pile is neglected in the design considerations.

Each prestressed anchorage consists of five plastic-sheathed greased strands (monostrand) SUP1630/1680 \(A_s = 140\) mm\(^2\). The tendons were fitted in a hole drilled with a diamond drill through the old foundation. The stressing of the tendons was performed by jacking up and wedging with VSL-anchorage clamps. The prestressing force on the anchorage was \(F = 750\) kN.

The drilled spiral piles under the concrete beams were test-loaded and the piles are against the beam and wedged against them, as explained in Section 3.4.2. Finally the steel wedges become embedded inside the concrete. The second prestressing sequence was carried out six months later to eliminate any loss of prestress force resulting from the creep effect in the previously injected foundation. After renewals of prestressing, the drilled holes were injected with cement mortar and concrete was cast on the anchor clamps (Avellan & Maanas 2001). The sequence of the work on line A is shown In Fig. 12 and may be described by the steps explained below:

(Note: some of the works can be done simultaneously)

1. Digging, first outside, then inside
2. Injection of the wall
   - day 1: erection of pipes and shotcreting by hand
   - day 2: waiting for the shotcreting to mature
   - day 3: injection of lower part
   - day 4: injection of upper part
3. Preparation of pile places and cutting timber for plank foundation if needed
4. Jetting of pile places and installation of piles
5. Torqueing the pile step by step
6. Boring of openings for anchors
7. Chiselling of the old stone foundation
8. Fabrication of concrete beams
9. Prestressing of the concrete beams together
10. Jacking the piles with special pretesting procedure in line A in incremental loading steps and at 15-minute intervals (Table 2)
11. End-jacking procedure (Table 3)
Fig. 12. Working phases of drilled and jacked spiral piles.
<table>
<thead>
<tr>
<th>Step</th>
<th>Jacking time (min)</th>
<th>Pressure (bar)</th>
<th>Jacking force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15</td>
<td>30</td>
<td>43</td>
</tr>
<tr>
<td>2</td>
<td>15</td>
<td>60</td>
<td>86</td>
</tr>
<tr>
<td>3</td>
<td>15</td>
<td>90</td>
<td>129</td>
</tr>
<tr>
<td>4</td>
<td>15</td>
<td>120</td>
<td>172</td>
</tr>
<tr>
<td>5</td>
<td>15</td>
<td>150</td>
<td>215</td>
</tr>
<tr>
<td>6</td>
<td>15</td>
<td>180</td>
<td>257</td>
</tr>
<tr>
<td>7</td>
<td>15</td>
<td>210</td>
<td>300</td>
</tr>
<tr>
<td>8</td>
<td>5</td>
<td>250</td>
<td>357</td>
</tr>
<tr>
<td>9</td>
<td>5</td>
<td>300</td>
<td>420</td>
</tr>
<tr>
<td>10</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>11</td>
<td>5</td>
<td>350</td>
<td>500</td>
</tr>
<tr>
<td>12</td>
<td>5</td>
<td>390</td>
<td>557</td>
</tr>
<tr>
<td>13</td>
<td>1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>14</td>
<td>5</td>
<td>400</td>
<td>572</td>
</tr>
<tr>
<td>15</td>
<td>5</td>
<td>420</td>
<td>601</td>
</tr>
<tr>
<td>16</td>
<td>5</td>
<td>450</td>
<td>644</td>
</tr>
</tbody>
</table>

Table 3. End-jacking procedure of drilled spiral piles in line A.

<table>
<thead>
<tr>
<th>Pressure (bar)</th>
<th>Time (min)</th>
<th>Distance $h$ of the top of the pile and structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>450</td>
<td>5 min</td>
<td>$h_1 = h_1 - h_2$</td>
</tr>
<tr>
<td>420</td>
<td></td>
<td></td>
</tr>
<tr>
<td>450</td>
<td>5 min</td>
<td>$h_2 = h_2 - h_3$</td>
</tr>
<tr>
<td>420</td>
<td></td>
<td></td>
</tr>
<tr>
<td>450</td>
<td>5 min</td>
<td>$h_3 = h_3 - h_4$</td>
</tr>
<tr>
<td>420</td>
<td></td>
<td></td>
</tr>
<tr>
<td>450</td>
<td>5 min</td>
<td>$h_4 = h_4 - h_5$</td>
</tr>
<tr>
<td>420</td>
<td></td>
<td></td>
</tr>
<tr>
<td>450</td>
<td>5 min</td>
<td>$h_5 = h_5 - h_6$</td>
</tr>
</tbody>
</table>

If the sum of the five steps $\Delta = \Delta_1 + \Delta_2 + \Delta_3 + \Delta_4 + \Delta_5$ was $\leq 10$ mm, then the pile was wedged against the concrete structure using the end load of 601 kN corresponding to hydraulic pressure of 420 bars.
The pretesting procedure is explained in Table 2 and illustrated as diagram in Fig. 13, which also includes the end-jacking procedure. The pressure was held constant for 15 minutes at each step and the settlement was measured at 3 intervals (5, 10, and 15 min). When the pressure of 450 bar (644 kN) decreased in 5 minutes by less than 50 bar (72 kN) to 400 bar (572 kN), the end-jacking could start.

![Fig. 13. Time used for test loading of each of the drilled spiral piles in line A.](image)

### 3.5.2 Test loading and spring coefficient

All of the drilled spiral piles that were installed went through the pretesting procedure with end-jacking to ensure their functionality. An example of the pretesting and end-jacking procedure of piles Nos. 220 and 222 is presented in Fig. 14. The layer with soils from dense to very dense silty sand has about the same thickness (approx. 4 m) but the upper level of the layer is slightly inclined. Line A is situated roughly on line III in Appendix 2 (page 4). Line B is situated roughly on line VI in Appendix 2 (page 3). The area surrounding the church has the same geological strata.

The outcomes of the procedures with piles Nos. 220 and 222 indicate that using the same pretesting and end-jacking procedure of the piles in line A, the allowable maximum load can be 400 kN. The safety factor $\gamma$ defined in DIN 1054 is 1.56:
\[
\gamma = \frac{602 \text{ kN} + 645 \text{ kN}}{2} \cdot \frac{1}{400 \text{ kN}} = 1.56. 
\]

The allowable load above is less than the creep load \( Q_c \) in Fig. 10.

Fig. 14. Load-settlement diagrams of drilled spiral piles Nos. 220 and 222 showing the repeated load cycling between 645 and 602 kN at the end of the pretesting procedure in line A.

The spring coefficient \( K_p \) for the piles may be evaluated from the settlement diagrams of Fig. 14 and as presented in Fig. 15 it is:

\[
k_p = \frac{F}{S} = \frac{375 \text{ kN}}{0.025 \text{ m}} = 15000 \text{ kN/m}. 
\]
Fig. 15. A parallel line in the pretesting diagrams of piles 220 and 222 justifies the evaluation of the spring coefficient for drilled spiral piles.

### 3.5.3 Settlement of drilled spiral piles

As based on the practical experience of the author, it may be estimated that without the pretesting and end-jacking procedure carried out for piles in line A, the settlement there could have been approximately 25 mm. This is obvious from the settlement diagrams in Figs. 14 and 15.

The surveying results from 21st October 1995 up to 9th June 2011 verify that the pretesting and end-jacking procedure developed worked well, as there is no noticeable settlement in line A. It is thus evident that the procedure is suitable for the dense and very dense silty sand layer of Tartu. The strata nearby lines A and B are shown in Appendix 2.

One of the levelling points (No. 3) has risen, probably due the settlement in the strata of the tower. The maximum settlement during the period mentioned above is 5 mm ± 2 mm. (Figs. 16 and 17). The rate of the settlement has been approximately 0.006 mm/week.
Fig. 16. Data from line A: Differences in level on 5.8.1993, 22.10.1995, and 9.5.2011 and date of pretesting and end-jacking procedures.

<table>
<thead>
<tr>
<th>Level point</th>
<th>Settlement before underpinning</th>
<th>Settlement after underpinning</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>P.H. 5.8.1993—P.H. 22.10.95 (mm)</td>
<td>P.H. 22.10.95—V.H. 9.5.11 (mm)</td>
</tr>
<tr>
<td></td>
<td>mm/week</td>
<td>Pretesting day</td>
</tr>
<tr>
<td>1.</td>
<td>227</td>
<td>14</td>
</tr>
<tr>
<td>3.</td>
<td>224</td>
<td>3</td>
</tr>
<tr>
<td>25.</td>
<td>212</td>
<td>11</td>
</tr>
<tr>
<td>26.</td>
<td>206</td>
<td>3</td>
</tr>
</tbody>
</table>

Survey technicians Pančić Hoopašari (P.H.) and Vešiška Veškera (V.H.)
### 3.6 Small jacked piles

In the area denoted by axes D/1–8 – E/4–8 (Fig. 5 and Appendix 1, Figs. 1 and 2) the foundations were mainly strengthened using jacked piles Ø 218 × 10. Smaller jacked piles Ø 140 x 8 were used in areas with lesser weighing separating walls, since the counter-weight by the structure did not allow the employment of larger jacking piles. In these areas the demolished working spaces, which had originally been in a stone setting, were fully cast after finishing the piling work with non-structural concrete.

---

**Fig. 17.** Data from line A: Differences in level on 5.8.1993, 22.10.1995, and 9.5.2011 and date of pretesting and end-jacking procedures.
3.7 Rebuilt columns

At line B/4–7 (Figs. 3 and 5) the old column structures destroyed during the Second World War, had to be rebuilt. The new structures were founded on drilled spiral piles in the same way as in line A, discussed in Section 3.4.2. The reinforced concrete foundation was made continuous to improve rigidity. During the jacking some screw piles were also used as tension piles. Test loading, erecting angular steels and wedging them against the superstructure and piles were carried out as explained in Section 3.5.1. The Pretesting procedure was made using max. pressure of 315 bars (450 kN) and the end-jacking procedure was made using pressures of max. 315 bars (450 kN) and min. 290 bars (415 kN). Reinforced concrete slabs with a depth $h = 300$ mm and reinforced concrete walls with thickness $h = 200$ mm were built between the new concrete foundations to balance the loads in the test piling. In addition to that, steel beams were installed between lines A and B to distribute the loads.

3.8 Tower foundations

Jacked steel piles $\varnothing 218 \times 10$ were used to strengthen the structure of the tower. Piling proceeded gradually by partly excavating stone masonry while simultaneously piling in the freed spaces. To ensure the stability of the structure, the piling work was scheduled in such a way that only one section of each pillar was set at one time. None of the piles were installed completely at once, but all of them were under construction at the same time. The stability of this work schedule was verified using a logarithmic spiral. The piling order was planned beforehand according to the settling and inclination measurements. The whole work was carried out with constant verification of measurements. In total 136 piles were installed below the underground extension of the tower, at approximately 0.8 m spacings. End-jacking procedure was performed with a load of 600...650 kN, which is 1.5 times the allowable maximum load of 400 kN.

Because of the test pile forces near the corners of the piers (Pi 3 and Pi 4) the columns had to be strengthened against splitting. This was carried out by using light pretensioning “stirrups” of steel and timber. It was estimated that for each of the four pillars of the foundation, the stress applied to the stone layers was 685 kN/m$^2$. The underpinning of the tower proceeded step by step while the concreting was going on.
A reinforced concrete raft foundation with thickness of $d = 600$ mm was cast part by part at the same time as the piling work went on. A stabilisation effect was achieved by this reinforced concrete raft, which was wholly cast as a floating piled raft foundation. The settlements during the work were monitored (Fig. 17). A concrete wall with thickness of 300 mm and height of 1200 mm shown in Fig. 18 was fabricated at the outer perimeter of the concrete slab to transfer the load from the stone and brickwork lying above. An empty space (Fig. 18) was left so as to give archaeologists and engineers a chance to study the accomplished work “in situ” in the future. All visible steel parts located in this space were painted against corrosion. The strengthening work was executed by following the anastylosis principle. One principal objective of the work was to conserve the authenticity of the old structures as much as possible.

![Fig. 18. Empty cavity space for future observation (Avellan & Maanas 2001).](image)

### 3.9 Burial chamber

The area which was excavated between lines 4–7 in lines A and B was not refilled (Appendix 1, Figs. 1 and 2). Instead, the ground was resurfaced with a concrete floor and a reinforced concrete slab was laid between the rafts in line B and the steel concrete beam in line A. In this way the open area formed a cellar space which was later used as a burial chamber for bones found in previous
archaeological excavations. The vault was equipped with drainpipes under the floor of the chamber. Because of differences in level, incoming water from the drainpipes had to be pumped to a sewer on the north side of the church.

3.10 Displacements monitored during the work period

The rate of settlement and horizontal displacements, particularly those of the tower, were measured throughout the strengthening process. As a result of the construction technique, the overall settlement of the tower was only 18 to 20 mm (Figs. 19, 20 and 21). At level 38 from the ground level the horizontal displacements were 30…50 mm during the underpinning work (Fig. 22). After the completion of the work the top part of the tower had a horizontal displacement of 20 mm, corresponding to 1/1900 (Fig. 21).

The settlement in other parts of the building during the strengthening work was monitored by levelling. According to the measurements, the inclination in other parts of the building was considerably less in comparison to that of the tower, as a result of continuous walls. In special places such as line A without roof, rebuilt columns in line B and columns in line C were underpinned using drilled spiral piles.
to 4 levelling in 22.10.1995 is done prior to pretesting.

Fig. 19. Settlement as a function of time in levelling points. Note: in line C in points 7

Note: Point 14. Local erosion failure due to pumping water before underpinning.
Fig. 20. Level differences in levelling points of the tower of St. John’s Church.

<table>
<thead>
<tr>
<th>Level point</th>
<th>Level difference P.H. 5.5.1993–P.H. 22.10.95 (mm)</th>
<th>mm/week</th>
<th>Level difference P.H. 22.10.95–V.H. 9.5.11 (mm)</th>
<th>mm/week</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>14</td>
<td>0.12</td>
<td>5</td>
<td>0.006</td>
</tr>
<tr>
<td>3.</td>
<td>3</td>
<td>0.02</td>
<td>-3</td>
<td>-0.003</td>
</tr>
<tr>
<td>18.</td>
<td>1</td>
<td>0.008</td>
<td>-5</td>
<td>-0.006</td>
</tr>
<tr>
<td>20.</td>
<td>7</td>
<td>0.05</td>
<td>2</td>
<td>0.002</td>
</tr>
<tr>
<td>23.</td>
<td>29</td>
<td>0.25</td>
<td>-1</td>
<td>-0.001</td>
</tr>
<tr>
<td>63.</td>
<td>18</td>
<td>0.16</td>
<td>4</td>
<td>0.004</td>
</tr>
<tr>
<td>81.</td>
<td>18</td>
<td>0.16</td>
<td>5</td>
<td>0.008</td>
</tr>
<tr>
<td>82.</td>
<td>-</td>
<td>-</td>
<td>3</td>
<td>0.003</td>
</tr>
</tbody>
</table>

Nr. 90002 Survey technician Pentti Haapasaari (P.H.)
h +42.668 and Veikko Heisko (V.H.)

Fig. 20. Level differences in levelling points of the tower of St. John’s Church.
Fig. 21. Monitoring data for the underpinning of pillars of the tower of St. John's Church in 1993–1995. Observed settlements and respective temperatures during the monitoring time. Results of the monitored points (Avellan & Lange 1997). Data added from the monitoring in 2011.
Fig. 22. Horizontal displacements of upper corners of the tower of St. John's Church at monitoring intervals (Avellan & Lange 1997).
4 Limit state design of a strip foundation

4.1 Plasticity theory of structures

When the limit state design is used for designing load-bearing structures, bending moments and shear forces within the structure can be determined by means of theory of elasticity and theory of plasticity. The verification of the structure is carried out both in ultimate limit state (ULS) and serviceability limit state (SLS).

When employing the theory of plastic hinges and the kinematic method is applied for considering the redundancy of the structure to a mechanism, pre-evaluation of the plastic bending moments associated with various hinge locations is required. The limit load associated with the mechanism may be determined by means of virtual work or alternatively the analysis may be based on the static method and redistribution of bending moments, if any.

The development of plasticity onsets from the section stressed highest prior to yielding and the lowest estimate for the limit load may be determined accordingly. However, the redistribution of bending moments allows that the load can increase and gradually more plastic hinges will develop until a number of hinges required for a mechanism have formed. The highest load that a structure can bear is associated with the plastic mechanism. The draw-back in using the theory of plastic hinges is that the designs are always associated with a fixed distribution of loads and a number of cases should be analyzed so as to find the most critical one (e.g. Neal 1970).

The limit theories applied to foundations imply that these structures perform acceptably in the ULS, provided that the structural deformations remain within the boundaries acceptable in the state of serviceability. The design procedure may be started by assuming some possible mechanisms for the structure and the foundation, and further assuming that the soil is locally failing (Fig. 23). For simplicity it is presumed that only the structural foundation member and the respective soil are locally in the ULS (Fig. 24) (Avellan 1992, Avellan 1994). For piled floating foundations it is possible to employ the same simplified design principle, as illustrated in Fig. 25.
Fig. 23. Two-storey structural frame founded on reinforced concrete strip foundation lying on the ground. The mechanism shown is one of the possible mechanisms of failure for the building frame and its foundation. (Avellan 1992).

Fig. 24. One distribution of contact pressure assumed for the strip foundation. a) In serviceability limit state. b) In ultimate limit state. (Avellan 1992).
With reference to the theories of plasticity and elasticity the design may further be simplified by applying the lower and upper bound theorems for the limit load. In doing so, however, the deformations of the substructure have to be estimated by means of elastic theory and based either on a loading specified especially for the purpose or on a load derived from the ultimate one including the partial load factors. A natural consequence in the application of boundary theorems is that the principle of superposition is not valid due to the non-linearity associated and the deformations should be estimated by theory of elasticity or they may be based on practical experience.

Fig. 25. One assumed distribution of contact pressure applied to the piled floating strip foundation. a) In serviceability limit state. b) In ultimate limit state.

4.2 Application of upper and lower bound theorems

In principle, conditions of equilibrium are sufficient to determine the displacement field, stress distribution, and ultimate load. In addition, the way of treating the boundary conditions influences the accuracy of the solution. It may be impossible to find a closed form solution for the action effects of the foundation when the weight of the soil and inclined forces are considered in detail.
4.2.1 Application of the lower bound theorem

For the purpose of employing the theorem it is written as follows:

If the stress state assumed for an object is statically admissible and fulfils the condition of equilibrium with the load and the stresses do not violate the failure condition, then the load referred to is at maximum equal to or less than the ultimate one that can develop (Neal 1970).

A balanced stress state can be determined based on external load and the associated boundary conditions; for instance one can start with the unloaded soil pressure. In principle, several stress states fulfilling the conditions mentioned above can be determined at the same time. The stress state physically closest to the correct one gives the best approximation for the ultimate (yield) load. This means that the solution can be kinematically impossible.

4.2.2 Application of the upper bound theorem

The upper bound sentence means that the actual ultimate load is equal to or higher than the boundary load or failure load based on the theory of plasticity (Neal 1970). The estimate for the failure load is evaluated using the following conditions:

− The displacement state of the system and any changes in it accord with the continuity conditions in the object (mass) by developing a kinematic mechanism.
− When the mechanism experiences any displacements, the associated work done by the external loads is equal to the internal plastic work of the action effects done by them in developing strain increments.

The upper bound theorem does not include a requirement for the equilibrium conditions, as it is associated only with the kinematic compatibility of the internal work consumed in creating strains in the member considered and external work done by the loads. The validity of the upper bound sentence is justified by the fact that only the plastic strains are always allowed for, omitting the elastic strains and deformations while they are considered small compared to the plastic ones.
4.3 Limit state design of a strip foundation

The ultimate load of a strip foundation loaded by concentrated loads in Fig. 24 is based on occurrence of one of the seven options listed below (Avellan 1992):

1. Failure of soil beneath the loading support ①
2. Failure of soil beneath the loading support ②
3. Failure of soil beneath the loading support ③
4. Columns in supports ⑦,⑧, and ⑩ punch the strip foundation.
5. Supports ⑦,⑧, and ⑩ cause the development of shear failure of the strip foundation.
6. Supports ⑦,⑧, and ⑩ cause the development of bending failure of the strip foundation.
7. In fields ⑦,⑧ or ⑩, ⑩ cause the development of bending failure of the strip foundation.

The bearing resistance of a strip foundation resting on frictional soil is a special case of soil plasticity and a case of 3-dimensional passive pressure. The ultimate contact pressure $q_u$, the soil pressure with a safety factor of 1, is a special ULS-case. Thus the bearing capacity and ultimate contact pressure are functions of passive pressure.

Horn (1970) found the relationship shown in Fig. 26 between the ratio $P_{p(s)}/P_p$ and $S/S_f$ to be non-linear in Fig. 26. He investigated the relationship between passive pressure and horizontal wall movements by test-loading rigid wall parts ($h/b < 3.33$) against frictional earth. The elastic-plastic model according to the another (Fig. 26) is based on that the elastic-plastic pressure-settlement relationship includes an elastic behaviour up to the settlement ratio $S_k/S_f = 0.2$. For this settlement ratio the passive pressure $P_{p(s)}$ is 0.7 times the passive pressure at failure $P_p$. 
The limit state design of a strip foundation can be performed using axioms 1 to 4 which accord to the lower bound theorem.

Fig. 27. a) Strip foundation, design loads, and design contact pressure of superstructure b) Separate parts, resistant contact pressures, and resistances of substructure (Avellan 1992).
Axiom 1: All individual parts in the loaded strip foundation can be designed independently of other parts, representing thus a minimum foundation to meet the requirements for the design in ULS (Figs. 27 and 28)

\[ F_{d1} \leq R_{d1} \]
\[ F_{d2} \leq R_{d2} \]
\[ F_{d3} \leq R_{d3} \].

Axiom 2: The resistance of the whole strip foundation is not less than the sum of the resistances in individual parts.

\[ R_{dT} \geq R_{d1} + R_{d2} + R_{d3} \].

Thus the failure capacity of the soil for the whole strip foundation is verified by the individual parts of the strip foundation considered separately.

![Diagram of strip foundation with parts I, II, and III](image)

Fig. 28. Total strip foundation and separated minimum parts I, II, and III. Forces acting in ULS (Fd) from the superstructure and opposing forces Rd (Avellan 1992).

Axiom 3: Distribution of contact pressure shall satisfy the requirement of equilibrium in all individual parts and the mean contact pressure due to loads shall not exceed the mean design contact pressure of substructure.

The distribution of contact pressure for the ULS analysis can be taken into account of by choosing contact pressures according to the following conditions which means that the lower bound theorem becomes fulfilled in the ULS in the...
most simple way for the minimum effective foundation, i.e. \( p_d \leq q_d \) (Figs. 28 and 29).

\[
\begin{align*}
& p_d^{(1)} \leq q_d^{(1)} \\
& p_d^{(2)} \leq q_d^{(2)} \\
& p_d^{(3)} \leq q_d^{(3)}.
\end{align*}
\]

In addition, the distribution of the contact pressure \( p_d(x) \) must be applied in such a way that the mean contact pressure is \( p_{od} \) in Fig. 30.

Fig. 29. Contact pressure \( q_d \) of parts I, II, and III and contact pressure \( p_{od} \) of part TF (Avellan 1992).

Fig. 30. Balanced contact pressure \( p_d(x) \) (Avellan 1992).
Axiom 4: The settlement of the total strip foundation is less than the sum of the settlements of the parts in the foundation.

The relative values of the settlements in relation to the settlement of the total strip foundation are shown in Fig. 31 and the settlement of the total strip foundation, denoted as TF, is less than the sum of the settlements of foundations I, II and, III.

Fig. 31. Relative settlements of parts I, II, III, and TF when the settlement of foundation TF (S(L0)) is given a unity value (Avellan 1992).
Fig. 32. Geotechnical And structural design procedure in limit states for strip foundation. Double lines around boxes represent serviceability state (Avellan 1992).

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A strip foundation can be designed in geotechnical and structural ultimate limit states in accordance with the chart presented in Fig. 32. According to the design procedure, the first values to be determined are the forces acting on the foundation resulting from the superstructure. Subsequently the dimensions of the footing and foundation depth D are defined. Then the simple transformation conditions below are taken into account and treated:

- The mean contact design pressure $q_{0d}$ for the strip foundation length $L_0$ is determined for the whole strip foundation using the formula of Brinch Hansen (Brinch Hansen 1961);
- The mean design contact pressure $p_{0d}$ resulting from the external load is determined using the Equation (3):

$$p_{0d} = \frac{\Sigma F_d}{L_0 \cdot B}, \quad (3)$$

where

- $F_d$ is the design value of external vertical action to be transferred from the superstructure to the soil in the ultimate limit state;
- $L_0$ is the length of the strip foundation, and
- $B$ is the width of the strip foundation.

The limit state design of a strip foundation can be performed using the following procedure.

The elastis-plastic pressure-settlement relationship in Fig. 26 includes an elastic behaviour up to the settlement ratio $S_k/S_f = 0.2$. For this settlement ratio the passive pressure is 0.7 times the pressure in failure.

Because the ratio $p_{0d}/q_{0d}$ due to load from superstructure, $p_{0d}$, and design contact pressure of substructure, $q_{0d}$, correspond to the ratio of $P_p(s)/P_p$, the ratio of settlement $S_k$ and settlement at failure, $S_f$, can be defined by the ratio $p_{0d}/q_{0d}$, which is marked as $S_{sq}$ yielding thus:

$$S_{sq} = \frac{p_{0d}}{q_{0d}} = \frac{\Sigma F_d}{L_0 \cdot B \cdot \Sigma R_d/L_0 \cdot B}, \quad (4)$$

where

- $S_{sq}$ is the external design load resulting from the superstructure divided by the design value of the resistance of the strip foundation;
\[ F_d \] is the design value of the external vertical action defined from the superstructure in its ultimate limit state;
\[ R_d \] is the design value of the resistance of substructure in ultimate limit state;
\[ p_{0d} \] the mean design contact pressure beneath the whole strip foundation calculated for the load transferred from the superstructure, and
\[ q_{0d} \] is the mean design contact pressure of the substructure beneath the whole strip foundation.

When the conditions mentioned in box 4 in Fig. 32 are fulfilled, then a settlement examination required for the feasibility of the strip foundation is carried out. As the contact pressure, \( q \), is used, the design value

\[ q_d = \frac{q_u}{\gamma}, \]  \hspace{1cm} (5)

where
\[ q_u \] is the soil pressure at failure and
\[ \gamma \] is the overall safety factor representing a combination of safety factors.

Depending on the soil characteristics, the value of mean settlement can be estimated by settlement calculation or by penetration testing using the methods of Schultz & Sherif (1973) and Burland & Burbidge (1985). Both of these methods are based on in situ soil explorations. From the settlement evaluation the ULS design procedure continues to boxes 6 and 7 in Fig. 32. In homogenous soil conditions the contact pressure can be expected to be in the plastic state (in ULS) and to be uniformly distributed in accordance with Brinch Hansen (1961).

**Conditions of equilibrium**

**Part I (Fig. 28)**

\[ (L_u + x_1) \leq 2L_u. \]  \hspace{1cm} (6)

Equation \( R_{d1} = F_{d1} \) has to be resolved for \( x_1 \).
\( R_{d1} \) is design value of the resistance of the foundation part I according to Brinch Hansen (1961).
Part II (Fig. 28)

$R_{d_{III}} = F_{d_{III}}$; $x_2$ has to be resolved.

$R_{d_{III}}$ is design value of the resistance of the foundation part III according to Brinch Hansen (1961).

Since $\frac{p_{0d}}{q_{0d}}$ is less than 0.70, it can be assumed that a correlation, in accordance with the equation of Schleicher (1926), exists between the settlement, load, and geometry of the strip foundation. On account of this correlation, the settlements of parts I, II, and III, as well as part TF (= the total strip foundation), can be determined using the following equation (7) based on Schleicher (1926) and Gordunov-Posanov (Tsytovich 1981):

$$S = q \cdot \frac{B \left(1 - \nu^2\right)}{E_d} \cdot I_p,$$

where

- $S$ is the settlement;
- $q$ is the contact pressure;
- $B$ is the width of the strip foundation or the diameter of the circular foundation;
- $\nu$ is Poisson’s ratio;
- $E_d$ is the modulus of deformation in drained state, and
- $I_p$ is an influence factor depending on rigidity, aspect ratio of the foundation area, and the depth between the foundation and hard strata (Tsytovich 1981).

The influence factor $I_p$ has been determined for the ratio $Z_p / B = 3$, where $Z_p$ is the depth from the foundation to the non-compressible strata (Tsytovich 1981). Denoting the settlement of Part I as $S_I$, the settlement of Part II as $S_{II}$ and the settlement of Part III as $S_{III}$, the settlements of these parts are by virtue of Eq. (7):

$$S_I = q_{d_{I}} \cdot \frac{B \left(1 - \nu^2\right)}{E_d} \cdot I_p,$$

$$S_{II} = q_{d_{II}} \cdot \frac{B \left(1 - \nu^2\right)}{E_d} \cdot I_p,$$  

and

$$S_{III} = S_I.$$
Plastic moments are calculated by the kinematic method; plastic hinges and rotation points are illustrated in Fig. 33.

![Fig. 33. Plastic mechanism of strip foundation, plastic hinges (H), and rotation points (RP) (Avellan 1992).](image)

In Fig. 33 the numbering of the parts differs from the previous figures. Parts I, II, III, and IV constitute the mechanism, supports 1 and 9 are rotation points, and the H-points are plastic hinges. As one possible distribution of contact pressures has been determined, the plastic bending moments in the strip foundation can be defined on account of the upper bound theorem. Next, the differences in the settlement of separate parts of the strip foundation Fig. 28 are determined in the serviceability limit state, assuming the contact pressures equal to the design values $q_d$, shown in a formula (8)...(10).

The strip foundation and superstructure are checked in SLS by the mean total settlement (Equation 11) and angular distortions both divided by safety factors. Simultaneously, the fixed end moment $M_o$ with a safety factor of 1 is determined by means of the settlement differences of the separated parts. The design moments of the foundation are determined by combining the fixed end moment $M_o$ and plastic moment $M_p$. The ductility of the foundation is ensured by limiting the value of the equivalent reinforcement ratio as given in the FIP.
recommendations (1984). The condition given in Eq. (13) will ensure the rotation capacity for plastic hinges in normal grades of the concrete and is conservative when compared to rules given in EN 1992-1-1 and CEB-FIP Model Code 1990:

\[ \rho_{ed} = \rho_s + \rho_p \frac{f_{0.1k}}{f_{yk}} \leq 0.02. \]  

(13)

Here the reinforcement ratios of the concrete section are

\[ \begin{align*}
\rho_s &= \frac{A_s}{B \cdot d} \\
\rho_p &= \frac{A_p}{B \cdot d} 
\end{align*} \]  

(14)

where

- \( \rho_{ed} \) is the equivalent reinforcement ratio;
- \( \rho_p \) is the tension reinforcement ratio for the prestressing tendons;
- \( \rho_s \) is the tension reinforcement ratio;
- \( f_{0.1k} \) is the characteristic (0.1 % proof) strength of the (prestressing) steel;
- \( f_{yk} \) is the characteristic strength of the reinforcement;
- \( A_s \) is the area of tension reinforcement;
- \( A_p \) is the area of the prestressing tendons, and
- \( B \) is the width of the section.

If the deformation capacity is not sufficient, it may be increased by specifying a greater structural depth or higher concrete strength, provided that the strength grade of C50/60 is not exceeded (Fig. 32, box 17). When the conditions in boxes 15 and 16 in Fig. 32 are fulfilled, reinforcement drawings can be produced accordingly. A numerical example of the theory can be found in Avellan (1992). The theory in this section can also be used for floating piled foundations discussed in Section 5.
5 Foundation behaviour in rebuilt area

The method developed by the author is explained in three phases
- Soil-foundation-pile interaction in SLS
- Soil-foundation-pile interaction in ULS
- Soil-foundation-pile-slab interaction in ULS

5.1 Soil-foundation-pile interaction in SLS

In this chapter the theory explained in Section 4.3 is used to estimate the settlement of the piles and foundations in line B in St. John’s Church. As described in Section 3.4.2, lines A and B were underpinned with drilled spiral piles. The location of line B in the church is shown in Fig. 5 and in Appendix 1 (Figs. 1 and 2), as well as in Fig. 34.A. The coordinate system is explained in Fig. 34.B. The work joints and piles in line B are shown from the top in overhead view and from the side in Figs. 35 and 36.

Fig. 34. A) Line B in St. John’s Church. Piled foundations of reconstructed columns and foundation of wall for the burial chamber.
Fig. 34. B) Line B. Coordinate system, displacements and direction of positive moments.

Fig. 35. Line B in St. John's Church (overhead view) work joints and piles.

Fig. 36. A) Section in direction x of line B in St. John's Church and piles.
The columns and the roof of the church were totally destroyed during World War II and had to be rebuilt. Because of the lack of a sufficient counterweight of the spiral piles in line B, the work was different from that of line A. The pretesting and end-jacking process differs little compared to the process used in axis A. In axis B the pretesting process consisted of forces, load steps and jacking times as in axis A up to 415 kN then to zero and from 450 kN to zero and the end-jacking procedure consisted of forces of 450 kN and 415 kN, and the sum of settlement was the same, ≤ 10 mm.

The counterweight consisted only of the dead load of the foundations, estimated at 180 kN. There are six piles in foundations I and I’. Two of the piles are used as tension piles during the construction stage, with a force of 200 kN per pile. The section of foundation in line B is presented in Appendix 1 (Fig. 5).

The determination of the admissible force $F_{adm}$ was to be $F_{test}$ divided by a safety factor of 1.5. Concerning the work programme of the drilling of the spiral piles, initially some of them were used as tension piles for column foundations I and I’ and then as compression piles. For the purpose of minimising the number of piles used in tension, the foundation slabs II, IV, and II’ were backfilled and the walls for the burial chamber were erected.

Two tension piles are

$$2 \cdot 200 \text{ kN} = 400 \text{ kN}. \quad (15)$$

The load resulting from the foundation is

$$\frac{F_{g}}{2} = 90 \text{ kN}. \quad (16)$$

Together, this is 490 kN and $F_{adm}$ is
According to the pretesting and end-jacking procedures and the fact that the foundation system is a floating strip foundation, we first take as the admissible pile load the end-jacking force divided by the safety factor 1.3. Then

\[
\frac{415 \text{ kN}}{1.3} = 319 \text{ kN}. \tag{18}
\]

The admissible load was chosen to be \( F_{\text{adm}} = 300 \text{ kN} \).

A force of 450 kN was used in the test loadings, as presented in Appendix 1. Theoretically, the settlement of the foundation should be calculated using a reconstruction path. A simplified practical calculation is made at the end of the reconstruction stage by using pile springs and foundation springs. Because of the reconstructed brick arch between axes A and B the top of the column at axis B is loaded with a horizontal load. This is caused by the fact that the arch between lines B and C could not be constructed because of the lack of old drawings. The loads in SLS at the level +37.4 provided by the engineer of the superstructures were

\[
G_k + Q_k = 1404 \text{ kN} \quad \text{and} \quad M_{y_k} = 624.5 \text{ kN}\cdot\text{m}. \tag{19}
\]

The subscript \( k \) is used as a symbol for SLS, which means that characteristic values are used.

The vertical load \( V_k \), including the dead load of the foundation, is

\[
V_k = G_k + Q_k + G_f = 1584 \text{ kN}. \tag{21}
\]

The load eccentricity \( e_{y_k} \) is

\[
e_{y_k} = \frac{M_{y_k}}{V_k} = \frac{624.5}{1584} = 0.394 \tag{22}
\]

Thus the resultant is in the kern.

The effective breadth \( B_{\text{eff}} \) is then

\[
B_{\text{eff}} = 2 \left( \frac{B}{2} - e_{y_k} \right) = 2 \left( \frac{2.5}{2} - 0.394 \right) = 1.71 \text{ m}. \tag{23}
\]
Then the characteristic pile loads of the three piles in Fig. 35. are

\[(21B\ldots19B) = 479.8 \text{kN} \quad (24)\]

and for the three piles

\[(22B\ldots24B) = 1,104.3 \text{kN}. \quad (25)\]

Before the brickwork scaffolds of the arch are taken away, the eccentricity is zero and the pile loads are about

\[
\frac{1584 \text{kN}}{6} = 264 \text{kN} \quad \text{(neglecting the positive influence of the soil)}. \quad (26)
\]

The maximum load of one pile is 368.1 kN and this is more than the admissible pile load (300 kN). To get approximately the same level of safety as the other piles that were tested, we use soil structure interaction as follows: \(V_{kF}\) is the characteristic load of the superstructure on three piles without soil action and it is

\[3 \cdot 368.1 \text{kN} = 1,104.3 \text{kN}. \quad (27)\]

Using the empirical correlation between the Swedish weight sounding test and the frictional angle of the soil, we use \(\varphi\) as \(\varphi = 29^\circ\) (Bergdahl & Eriksson 1983). With the bearing capacity formula of Brinch Hansen we obtain

\[q_s = q_b + q_d \quad (28)\]

\[q_s = 0.5 \cdot \gamma_s' \cdot B \cdot N_b' \cdot S_b + \gamma_s' \cdot D \cdot N_d' \cdot S_d, \quad (29)\]

Note: \(q_d\) is associated with \(\gamma_s'\) instead of \(\gamma_s\) (transient situation).
where

- \( q_u \) is the ultimate base resistance pressure;
- \( \gamma' \) is the effective unit weight of the soil;
- \( B \) is the breadth of the foundation;
- \( N_B, N_D \) is the bearing capacity factor;
- \( S_B, S_D \) is the shape factor, and
- \( D \) is the depth of the foundation.

\[
q_u = 89.9 \text{ kN/m}^2 + 83.5 \text{ kN/m}^2 = \frac{q_u}{1} = 173.4 \text{ kN/m}^2. \tag{28} \text{bis}
\]

According to the Riiklik Ehitusuuringute Instituut, \( \phi \) is 28° and \( c \) 10 kN/m² but for practical simplicity, the design is made as if the soil were pure frictional earth.

Using \( q_d \) as the soil and foundation pressure divided by 1.6, we obtain

\[
q_d = \frac{q_u}{1.6} = \frac{173.4}{1.6} = 108.4 \text{ kN/m}^2, \tag{30}
\]

where

- \( q_d \) is design value of the contact pressure from the substructure.

This means that we are in the linear part of the curve in Fig. 26, because of \( 1/1.6 = 0.625 \), which is less than 0.7.

Using \( B_{eff} \) as the breadth of the foundation the settlement is

\[
S_p = \frac{q_d \cdot B_{eff} \cdot (1 - v^2)}{E_d} I_p, \tag{7} \text{bis}
\]

Using the empirical correlation with the Swedish weight sounding results (Bergdahl & Eriksson 1983), we get

\[
E_d = 15 \, 000 \text{ kN/m}^2 \tag{32}
\]

\[
S_p = \frac{108.4 \cdot 1.71 \cdot 0.91 \cdot 0.865 \cdot 10^{-3}}{15 \, 000} \text{ mm} = 9.7 \text{ mm} \tag{33}
\]

\[
K_{FA} = \frac{q_d}{S_p} \tag{34}
\]

where

- \( K_{FA} \) is the subgrade reaction.
\[ K_{FA} = \frac{108.4 \text{ kN/m}^2}{0.0097 \text{ m}} = 11175 \text{ kN/m}^3 \]  

(35)

\[ k_F = K_{FA} \times A_F, \]  

(36)

where 

- \( k_F \) is the spring coefficient of the foundation, and 
- \( A_F \) is the effective sub-area of the foundation.

\[ k_F = 11175 \text{ kN/m}^3 \cdot (1.71 \cdot 2.6) \text{ m}^2 = 49685 \text{ kN/m}. \]  

(37)

The total pile spring \( k_{PT} \) is for the three piles carrying the maximum load.

\[ k_{PT} = \Sigma k_p = 3 \cdot k_F \quad \text{and} \quad \Sigma k_p = 3 \cdot 15000 \text{ kN/m} = 45000 \text{ kN/m}. \]  

(38, 39)

Compatibility

\[
\begin{align*}
S_F &= S_p \\
Q_F + \Sigma F_{ip} &= V_{4p}
\end{align*}
\]  

(40)

\[
\begin{align*}
S_F &= \frac{Q_p}{k_F} \\
S_{ip} &= \frac{\Sigma F_{ip}}{\Sigma k_p}
\end{align*}
\]  

(41)

\[ Q_F = V_{4p} - \Sigma F_{ip} \]  

(42)

\[ S_F = \frac{V_{4p} - \Sigma F_{ip}}{k_F} = \frac{\Sigma F_{ip}}{\Sigma k_p} \]  

(43)

\[ \frac{\Sigma k_p}{k_F} \cdot V_{4p} = \Sigma F_{ip} \left( 1 + \frac{\Sigma k_p}{k_F} \right) \]  

(44)

\[ \Sigma F_{ip} = \frac{k_F}{1 + \frac{\Sigma k_p}{k_F}} \cdot V_{4p} \]  

(45)
Thus, $F_{kP} < \text{resistance of the piles, while every pile is test loaded.}$

The settlement of the piles is $S_p = 11.7 \text{ mm}$ and the settlement of the foundation is

$$S_f = \frac{V_{kF} - \sum F_{kP}}{k_f} = \frac{1104.3 - 524.8}{49685} = 11.7 \text{ mm}. \quad (48)$$

Now we have to prove that the designed soil pressure is smaller than $q_u$. We have used $q_{kF}$ as $q_u/1.6 = 0.625 q_u$. We notice (Fig. 26) that the calculated settlement, 9.7mm is about 0.18 times the settlement at failure ($S_f$), which means that $S_f$ is about 54 mm. According to SLS, $S_p$ is 11.7 mm and less than $S_f = 54$ mm. The $S_p/S_f$ ratio is 0.22 and then $q_{kF}$ is 0.72 times $q_u$ (Fig. 26, non-linear line).

$$0.72 \cdot 173.4 \text{ kN/m}^2 = 124.8 \text{ kN/m}^2, \quad (49)$$

$$\gamma = \frac{173.4 \text{ kN/m}^2}{124.8 \text{ kN/m}^2} = 1.39. \quad (50)$$

The piles have enough resistance, but for limiting theoretical settlements we study the calculation in SLS to see the influence of slabs, Section 5.3.

### 5.2 Soil-foundation-pile interaction in ULS

The previous soil foundation interaction calculations are based on SLS calculations. It is easy to make these same calculations in ULS as follows:

- using, for the sake of simplicity,
  $$1.20G_{k} + 1.6Q_{k} + 1.20G_{f} = 1.4 \cdot (G_{k} + Q_{k} + G_{f}) \quad (51)$$

- the load eccentricity remains the same
- the subscript $d$ is used as a symbol for ULS; design value of load, design value of material strength etc.
– the subscript k is used as a symbol for SLS; characteristic values of loads, characteristic value of material strength etc.

\[ V_{dp} \text{ for three piles is} \]

\[ 1.4 \cdot 1104.3 = 1546.0 \text{ kN} \] (52)

and for one pile \( V_{dp} = 515.3 \text{ kN} \).

Now we use \( q_{	ext{et}} \) because of elastic linearity (\( l/1.6 = 0.625 < 0.7 \)).

As in the previous SLS calculation, we obtain

\[
\begin{align*}
S_p &= Q_p + \Sigma F_{dp} = V_{dp} \\
S_p &= \frac{Q_p}{k_F} \\
S_{dp} &= \frac{\Sigma F_{dp}}{\Sigma k_p} \\
Q_F &= V_{dp} - \Sigma F_{dp} \\
\frac{V_{dp} - \Sigma F_{dp}}{k_F} &= \frac{\Sigma F_{dp}}{\Sigma k_p} \\
\frac{\Sigma k_p}{k_F} \cdot V_{dp} &= \Sigma F_{dp} \cdot \left( 1 + \frac{\Sigma k_p}{k_F} \right) \\
\Sigma F_{dp} &= \frac{\Sigma k_p}{k_F} \cdot V_{dp}
\end{align*}
\] (53-58)

(Resistance of the piles, every pile is pretested and end-jacked.)

\[ F_{dp} = \frac{\Sigma F_{dp}}{3} = 244.9 \text{ kN} < \frac{415}{1.1} \approx 377 \text{ kN} \] (59)

The safety factor 1.1 above is a value based on author’s practice.
The settlement of the piles is \( S_p = 16.3 \) mm and the settlement of the foundation is

\[
S_f = \frac{V_{ap} - \Sigma F_{ap}}{k_f} = \frac{1546.0 - 734.8}{49685} = 16.3 \text{ mm}
\]

\[ (61) \]

\[
\frac{S_p}{S_f} = \frac{16.3 \text{ mm}}{54 \text{ mm}} = 0.30; q_{ap} \sim 0.80 \cdot 173.4 \text{ kN/m}^2 = 138.7 \text{ kN/m}^2
\]

\[ (62) \]

\[
\gamma = \frac{173.4}{138.7} = 1.25
\]

\[ (63) \]

Piles have enough resistance but for limiting theoretical settlements we study the calculation in ULS to the influence of slabs; see Chapter 5.3.

### 5.3 Soil-foundation-pile-slab interaction in ULS

The piles, foundation I, and slab II can be calculated in the same procedure using the method described in Sections 4.3 and 5.2, (Fig. 38). The bearing capacity of slab II is (Equations 28, and 29):

\[
q_u = 0.5 \cdot \gamma \cdot B_{ef} \cdot N_b \cdot S_b + \gamma \cdot D \cdot N_d \cdot S_d
\]

\[
= 0.5 \cdot 1.21 \cdot 13 \left( 1 - 0.4 \cdot \frac{1.21}{2.9} \right) + 11 \cdot 0.31 \cdot 16.55 \left( 1 + \sin(29) \cdot \frac{1.21}{2.9} \right)
\]

\[ = 139.9 \text{ kN/m}^2 \]

\[ (64) \]

The value \( q_u \) is associated with \( \gamma_s \) instead of \( \gamma_e \) (transient situation).

The settlement of slab II

\[
S_{II} = q_u \cdot B_{ef} \cdot (1 - \nu^2) \cdot \frac{1}{E_d} = 139.9 \cdot \frac{1.21 \cdot 0.91}{15000} \cdot 1.15 \cdot 10^3 = 7.4 \text{ mm}
\]

\[ (65) \]
The design process explained in Section 4.3 is presented in Table 4.

Table 4. Calculation process. Settlements and resistances of the parts of the structure.

<table>
<thead>
<tr>
<th>Parts of the structure</th>
<th>I</th>
<th>II</th>
<th>III</th>
<th>IV</th>
<th>III'</th>
<th>II'</th>
<th>I'</th>
</tr>
</thead>
<tbody>
<tr>
<td>L (m)</td>
<td>2.6</td>
<td>2.9</td>
<td>2.6</td>
<td>2.9</td>
<td>2.6</td>
<td>2.9</td>
<td>2.6</td>
</tr>
<tr>
<td>B_{eff} (m)</td>
<td>1.71</td>
<td>1.21</td>
<td>1.71</td>
<td>1.21</td>
<td>1.71</td>
<td>1.21</td>
<td>1.71</td>
</tr>
<tr>
<td>A (m²)</td>
<td>4.45</td>
<td>3.50</td>
<td>4.45</td>
<td>3.50</td>
<td>4.45</td>
<td>3.50</td>
<td>4.45</td>
</tr>
<tr>
<td>L₂ (m)</td>
<td>19.1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B₂ (m)</td>
<td></td>
<td>1.21</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>qₓ (kN/m²)</td>
<td>173.4</td>
<td>139.9</td>
<td>173.4</td>
<td>139.9</td>
<td>173.4</td>
<td>139.9</td>
<td>173.4</td>
</tr>
<tr>
<td>qₜ (kN/m²)</td>
<td>108.4</td>
<td>87.4</td>
<td>108.4</td>
<td>87.4</td>
<td>108.4</td>
<td>87.4</td>
<td>108.4</td>
</tr>
<tr>
<td>S (mm)</td>
<td>9.7</td>
<td>7.4</td>
<td>9.7</td>
<td>7.4</td>
<td>9.7</td>
<td>7.4</td>
<td>9.7</td>
</tr>
<tr>
<td>Rₓ (kN)</td>
<td>770.9</td>
<td>491.0</td>
<td>770.9</td>
<td>491.0</td>
<td>770.9</td>
<td>491.0</td>
<td>770.9</td>
</tr>
<tr>
<td>Σ Rₓ (kN)</td>
<td>4 556.6 kN</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Σ A (m²)</td>
<td>28.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Σ Vₓ (kN)</td>
<td>4 154.6 kN = 6 184 kN</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>pₓ (kN/m²)</td>
<td>6 184.0 kN</td>
<td></td>
<td>2 847.9 kN</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>qₓ (kN/m²)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>S_q (m²)</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

That means piles

\[
\sum V_{dp} - \sum R_d = 6 184.0 \text{ kN} - 2 847.9 \text{ kN} = 3 336.1 \text{ kN}.
\]  

For 12 piles F_{dp} is 278.0 kN.

Using the theory explained in Section 4.3 we get as the contact pressure the qₓ and pₜ(x) curves (Fig. 39B). The design model for the kinematic method of
plasticity theory is explained in Fig. 39C. The contact pressure of slab II is divided into a uniform part (Fig. 39D) and a triangular part (Fig. 39E).

\[ W_i = i_i \cdot M_p \cdot \varphi_i + 1.0 \cdot M_p (\varphi_I + \varphi_{II}) + i_{III} \cdot M_p \cdot \varphi_{III}. \]  

(67)

For an iteration process we assume \( \alpha = 0.45 \) and \( \beta = 0.55 \).

\[ i_I = 0.5, \varphi_I = 0.694, \]
\[ \alpha \cdot L = 1.44. \] (68)

\[ i_{III} = 1.5, \varphi_{III} = 0.568, \] (69)

\[ \beta \cdot L = 1.76. \] (69)

\[ W_i = 0.5 M_p \cdot 0.694 + 1.0 M_p \cdot 1.26 + 1.5 M_p \cdot 0.568 = 2.46, \] (70)

\[ W_i = 20 \cdot 3.2 \cdot 0.5 + 43.2 \cdot \frac{1}{3} + 52.8 \cdot \frac{1}{3} = 32 + 14.39 + 17.58 = 63.97, \] (71)

\[ W_i = W_e. \] (72)

\[ M_p = \frac{64.0}{2.46} = 26.0 \text{ kNm and} \] (73)

\[ M_{p_{III}} = 39.0 \text{ kNm}. \] (74)

This corresponds to

\[ \alpha \cdot L \geq 1.44 \text{ m.} \] (75)

The free body diagram (Fig. 40) includes only one unknown, namely \( \alpha \cdot L \), which can be solved by the principle of virtual work. \( Q_{s, \text{uniform}} \) is the resultant of uniform contact pressure and \( Q_{s, \text{triangular}} \) is the resultant of triangular contact pressure.

Fig. 40. Free body diagram of the left part of slab II.

\[ W_e = \alpha \cdot L \cdot 20 \cdot \frac{1}{2} + 30 \cdot \alpha \cdot L \cdot 0.333 = 20 \cdot \alpha \cdot L, \] (76)

\[ W_i = \frac{1}{\alpha \cdot L} \cdot 13.0 + 26.0 \cdot \frac{1}{\alpha \cdot L} = \frac{39.0}{\alpha \cdot L}, \] (77)

\[ W_i = W_e. \] (72) bis
\[ 20 \alpha \cdot L = \frac{39}{\alpha \cdot L}, \quad (87) \]
\[ (\alpha \cdot L)^2 = 1.95, \quad (88) \]
\[ \alpha \cdot L = 1.40 \sim 1.44. \quad (89) \]

The value of \( R_{\text{diff},e} \) can be solved by equilibrium (Fig. 40)

\[ (13.0 \text{kN/m} + 26.0 \text{kN/m}) - R_{\text{diff},e} \cdot 1.44 \text{ m} + Q_{\delta, \text{winge}} \cdot 0.96 + \frac{Q_{\text{uniten}} \cdot 1.44}{2} = 0, \quad (90) \]
\[ 39.0 + 43.2 \cdot 0.96 + 20.74 = R_{\text{diff},e} \cdot 1.44 \text{ m}, \quad (91) \]
\[ R_{\text{diff},e} = 70.3 \text{kN per 1 m.} \quad (92) \]

The moment \( M_x \) over foundation I is

\[ M_x = \frac{-70.3 \text{kN} \cdot 2.6 \text{ m}}{2} = -91.4 \text{kNm.} \quad (93) \]

\( \epsilon_x \) without the upper slab is

\[ \epsilon_x = \frac{-91.4 \text{kNm}}{1584 \text{kN}} = -0.06 \text{ m, can be ignored.} \quad (94) \]

Note: the moment of soil reaction \( R_{\text{diff},e} \) is divided by \( V_k \) instead of \( V_d \).

Steel area and rotation capacity

K35, \( f_{cd} = 16.3 \text{ MN/m}^2 \), A III (A400HW), \( f_{cd} = 333 \text{ MN/m}^2 \) and \( d = 259 \text{ mm} \).

\[ m = \frac{0.0390}{1.0 \cdot 0.259 \cdot 0.259 \cdot 16.3} = 0.0357, \quad (95) \]
\[ A_x = 0.036 \cdot 1000 \cdot 259 \cdot \frac{16.3}{333} = 456.4 \text{ mm}^2. \quad (96) \]

\( \phi 12 c 200 \text{ (565 mm}^2) \), high - bond bar

\[ \delta_{\phi} \approx \left( \frac{333}{1.4} \cdot \frac{456}{565} \right) = 192 \text{ MN/m}^2 < 200 \text{ MN/m}^2. \quad (97) \]

\[ \rho_x = \frac{A_x}{B \cdot d}, \quad (98) \]
\[
\frac{565}{1000 \cdot 259} = 0.002 < 0.02. \tag{89}
\]

### 5.3.1 Settlements of foundations I and III

Using the same procedure as in Section 5.2, we calculate the settlements of foundations I and III:

**Foundation I**

\[
V_{dp} = 1.4 \cdot 1104.3 \text{ kN} - 70.3 \text{ kN/m} \cdot 1.21 \text{ m} = 1461.0 \text{ kN}, \tag{90}
\]

\[
p_e(x) = 90 \text{ kN/m}^2, \text{ mean pressure (Fig. 39B).} \tag{91}
\]

\[
s_f = \frac{90 \cdot 1.71 \cdot 0.91}{15000} \cdot 0.865 \cdot 10^3 \text{ mm} = 8.1 \text{ mm}. \tag{92}
\]

\[
K_{fa} = \frac{90 \text{ kN/m}^2}{0.0081 \text{ m}} = 11111 \text{ kN/m}^3, \tag{93}
\]

\[
k_f = 11111 \text{ kN/m}^3 \cdot 1.71 \text{ m} \cdot 2.6 \text{ m} = 49400 \text{ kN/m}. \tag{94}
\]

\[
\Sigma F_{dp} = \frac{49400}{45000} \cdot 1.4610 \text{ kN} = 696.4 \text{ kN}, \tag{95}
\]

\[
F_{dp} = \frac{\Sigma F_{dp}}{3} = 232.1 \text{ kN} < \frac{415}{1.1} = 377 \text{ kN}. \tag{96}
\]

\[
S_p = 15.5 \text{ mm}, \tag{97}
\]

\[
S_t = \frac{V_{dp} - \Sigma F_{dp}}{k_f} = \frac{1461.0 \text{ kN} - 696.4 \text{ kN}}{49400 \text{ kN/m}} = 0.0156 \text{ m} = 15.5 \text{ mm}, \tag{98}
\]

\[
\frac{S_t}{S_t} = \frac{15.5 \text{ mm}}{54.0 \text{ mm}} = 0.29. \tag{99}
\]

\[
q_{dp} \sim 0.77 \cdot 173.4 \text{ kN/m}^2 = 133.5 \text{ kN/m}^2. \tag{100}
\]

\[
\gamma_d = \frac{173.4 \text{ kN/m}^2}{133.5 \text{ kN/m}^2} = 1.30, \tag{101}
\]

\[
\gamma_k = 1.82. \tag{102}
\]
Foundation III

Contact pressure reaction of slab II when $\beta L = 1.76$ m

$$R_{\text{dillight}} = \frac{2.9 m}{2} \cdot 1.21 m \cdot 20 \text{kN/m}^2 + \frac{2.9 m}{2} \cdot 1.21 m \cdot 60 \text{kN/m}^2 = 106.5 \text{kN}.$$  \hfill (103)

Contact pressure reaction of slab IV when $\beta = \alpha = 0.5$

$$R_{\text{dillight}} = \frac{2.9 m}{2} \cdot 1.21 m \cdot 20 \text{kN/m}^2 + \frac{2.9 m}{2} \cdot 1.21 m \cdot 60 \text{kN/m}^2 = 87.7 \text{kN}.$$  \hfill (104)

$$V_{\text{dr}} = 1.4 \cdot 1104.3 \text{kN} - 106.5 \text{kN} - 87.7 \text{kN} = 1351.8 \text{kN},$$  \hfill (105)

$$F_{\text{dr}} = \frac{644.4 \text{kN}}{3} = 214.8 \text{kN} < \frac{415 \text{kN}}{1.1},$$  \hfill (106)

$$S_p = 14.3 \text{mm},$$  \hfill (107)

$$S_f = \frac{V_{\text{dr}} - \Sigma F_{\text{dr}}}{k_f} = \frac{(1351.8 - 644.4) \text{kN}}{49400 \text{kN/m}} = 0.00143 \text{m} = 14.3 \text{mm},$$  \hfill (108)

$$\frac{S_f}{S_p} = \frac{14.3 \text{mm}}{54.0 \text{mm}} = 0.27.$$  \hfill (109)

$$q_{\text{dr in}} \sim 0.74 \cdot 173.4 \text{kN/m}^2 = 128.3 \text{kN/m}^2.$$  \hfill (110)

$$\gamma_d = \frac{173.4 \text{kN/m}^2}{128.3 \text{kN/m}^2} = 1.35,$$  \hfill (111)

$$\gamma_k = 1.4 \cdot 1.35 = 1.89.$$  \hfill (112)

5.3.2 Angular distortion and end moments resulting from angular distortion

The settlement difference in slab II in SLS is

$$S_{fI} - S_{fII} = \frac{15.5 \text{mm} - 14.3 \text{mm}}{1.4} = 0.86 \text{mm} = 0.86 \cdot 10^{-3} \text{m}.$$  \hfill (113)

The angular distortion $\Psi_k$ in SLS is

$$\Psi_k = \frac{S_{fI} - S_{fII}}{L_{\text{SLAB}_{11}}} = \frac{0.86 \cdot 10^{-3} \text{m}}{3.2 \text{m}} = \frac{1}{3721}.$$  \hfill (114)
The end moment $M_{e}$ reduces the plastic moment $M_{P_{III}}$ but increases $M_{P_{I}}$. $M_{\psi_{d}}$ is based on $\Psi_{k}$ (partial safety factor $\gamma_{d} = 1$).

$$M_{\psi_{Rig}} = \frac{6 \cdot E_{c} \cdot I_{c}}{L^{2}} \cdot \Delta$$ \hspace{1cm} (115)

and

$$E_{c} = \frac{E_{cm}}{1 + \varphi(\infty, t_{0})},$$ \hspace{1cm} (116)

where

- $E_{c}$ is the long-term Young’s modulus of concrete;
- $I_{c}$ is the effective second moment area;
- $E_{cm}$ is the value of $E_{c}$ at 28 days, and
- $\varphi(\infty, t_{0})$ is the creep coefficient.

Because of the testing of the drilled spiral piles, slab II was loaded by backfilling 3...4 weeks after the concreting work was finished.

$$\varphi(\infty, t_{0}) \approx 2.4$$

$$E_{c} = \frac{5000 \sqrt{K}}{1 + 2.4} = 8700 \text{ MPa},$$ \hspace{1cm} (117)

$$I_{c} = \frac{0.259}{12} \cdot 1.0 = 0.00145 \text{ m}^{3}$$ \hspace{1cm} (118)

$$M_{\psi_{Rig}} = \frac{6 \cdot 8 \cdot 700 \cdot 10^{3} \cdot 0.00145 \cdot 0.85 \cdot 10^{-3}}{3.2 \cdot 3.2} = 6.28 \text{ kNm},$$ \hspace{1cm} (119)

$$M_{d} = M_{P_{I}} + M_{\psi_{d}} = (13.0 + 6.28) \text{ kNm} = 19.28 \text{ kNm} < 39.0 \text{ kNm},$$ \hspace{1cm} (120)

$$e_{s_{1}} = \frac{(-91.4 - 19.28) \text{ kNm}}{1546 \text{ kN}} = -0.07 \text{ m}, \text{ can be ignored.}$$ \hspace{1cm} (121)

$$M_{d_{II}} = M_{P_{III}} + M_{\psi_{III}} = 39.0 \text{ kNm} - 6.28 \text{ kNm} = 32.7 \text{ kNm} < 39.0 \text{ kNm.}$$ \hspace{1cm} (122)

Thus, the structure has sufficient resistance and ductility.
6 Conclusions

6.1 General conclusions

This thesis discusses the strengthening of the foundations of historic buildings by using drilled spiral piles, jacked piles, and strip foundations. Underpinning with drilled spiral piles and jacked piles was applied to strengthen the foundations of St. John’s Church in Tartu because of the uneven settlement of the building. The strengthening work at St. John’s Church was challenging because of the risk of collapse of the historic structure. The methods chosen were suitable for these circumstances and the drilled spiral piles and jacked piles that were used perform well in addition to the old foundations. The efficiency of the strengthening techniques was verified with test piling. In addition to that, every single drilled spiral pile that was installed was test loaded using a special pretesting and end-jacking procedure. According to the test loadings, all the piles function well in their respective positions. The functional success of the new foundation structures in St. John’s Church was also proven by the fact that after the underpinning work had been performed, the settling of the church ceased.

The strengthening methods described in this thesis can successfully be used in strengthening the foundations of heritage structures. As stated by Iwasaki (2005), underpinning with jacked piles is the preferred method when underpinning historic buildings. The results of the strengthening work and underpinning in St. John’s Church proved that jacked piles and drilled spiral piles are an excellent choice to use in this kind of case. The piling methods described make it possible to execute the strengthening work according to the principle of anastylosis. The methods can be applied in challenging circumstances, even when the risk of the collapse of the building exists.

The advantages of using drilled spiral piles and the special equipment developed by the author are significant; low costs, portability of the equipment, and minimal workspace requirements. The application of these special methods does not require big investments, only the provision of a limited number of mechanical devices. In the future, this method could easily be applied in several other cases of foundations of historic buildings in need of underpinning. The low level of investment needed to implement these methods also makes them very well suited to developing countries such as those in Eastern Europe or Africa.
Another advantage is to use drilled spiral piles and concrete foundations together to when making piled floating strip foundations. It is possible by means of one simple manual calculation to verify that piled strip foundations work satisfactorily in ULS and SLS.

The study had three goals. One goal of the research was to clarify how to design a strip foundation when contact pressure is calculated as the ultimate soil bearing pressure divided by a safety factor and the forces acting from the superstructure are calculated in ULS. The study included the application of the upper bound theorem to the strip foundation. Furthermore, the bearing capacities were determined using the formula of Brinch Hansen and the contact pressure distribution in the fictitious ULS was roughly determined according to the settlement calculations of theoretically separated minimum foundations. The plastic moments of the structure were calculated by the kinematic method. The angular distortions were calculated using settlements which were estimated applying the contact pressure distribution in the ULS divided by a safety factor. Thus the settlement and the angular distortion were considered to be the criteria of the serviceability state.

Another goal was to clarify the design method of the piled floating strip foundation in SLS and ULS. Finally, the third goal was to prove the reliability of the pretesting and end-jacking procedure employed.

Summing up, the equipment, spiral pile and installation method, all developed by the author comprise a new practical piling method.

6.2 Thematic conclusions

1. Length of spiral part of the pile

The drilled and jacked spiral steel piles $\phi 218 \times 10$ referred to in this thesis, with a spiral length of 1 m, may be employed as compression piles in environments similar to the one encountered in Tartu.

2. Creep load of drilled spiral pile

The creep load of the drilled spiral pile without pretesting and end-jacking procedure

The creep load of the jacked drilled spiral piles is 525 kN, which means that the soil pressure under the tip is 15.9 MPa and under the spiral tip 9.9 MPa.
Because of jetting the skin friction has no or only a minor influence on the tip resistance.

*The creep load tested of drilled spiral pile with pretesting and end-jacking procedure*

The monitoring of levelling points indicates no significant settlement. Therefore the creep load of pretested and end-jacked drilled spiral pile is higher than 525 kN.

3. Settlement of drilled spiral pile

On account of the pretesting and end-jacking procedure in axis A by the author, the settlement of the drilled spiral pile can be estimated to be 5 mm, which means 20% of 25 mm (Fig. 15). This estimation is based on levelling (Figs. 16 and 17). The maximum settlement in the above-mentioned sixteen years has been 5 mm. The surveying accuracy between levelling points is 0.5 mm. The reason for the small settlements is probably soil hardening which occurred as a result of the use of the author’s pretesting and end-jacking procedure. The technique referred to has also been used in the 2000’s in foundations of high-rise buildings (H. Brandl 2005).

4. Drilling equipment and installation of drilled spiral pile

The developed equipment and mechanical installation method in St. John’s Church with the use of drilled spiral piles are probably unique in their kind, although there is evidence of earlier use of torqued piles with screw tips using manpower (O. W. Lille 1901). The jetting is used to investigate if there are any boulders in the soil. The jetting also makes the drilling easier and helps to have the pile in the right direction. The main machinery involved in the installation can be placed outside the workspace. The drilled spiral pile type used is especially practical for the strengthening of historical foundations. Neither injection nor vibrations are involved. The piles can be installed in places where very little space is available, unlike other methods that need more space.

5. Soil-foundation interaction

*Settlement of soil*

The Schleicher-Tsytovich formula provides high settlement values and therefore yields results which can be considered too conservative. It was not
possible in this study to clarify the stress influence of the weight and the long-term influence of the weight of the medieval church on the soil.

The author has used the same safety factor of 1.6 for estimating the settlement of the soil, in spite of the serviceability state or ULS. According to the author’s opinion, this satisfies normal cases, when the soil stratum is homogenous, as is the case around St. John’s Church in Tartu. If the soil strata is not homogenous the author’s method may be used with a greater safety factor. In principle, the author’s “separate foundations” method for soil-foundation-pile interaction and for the piled floating strip foundation automatically takes care of limit states (ULS and SLS).

A small critical point in this thesis may be the employment of the curve of Horn (Fig. 26), which is based on experimental studies. It is nevertheless only employed to check the contact pressure after the settlement of the piled foundation and thus, in the author’s opinion, has no practical influence on the author’s method.

Soil-foundation-pile interaction in SLS (Axis B), Chapter 5.

The safety factor of the piles compared to RakMk B3:

– the safety factor of a pretested and end-jacked pile is 1.5
– the calculated pile load is 174.9 kN
– the real safety factor in SLS is then

\[ \frac{319.2 \text{ kN}}{174.9 \text{ kN}} = 1.83, \]

which is greater than the required safety factor of 2.2.

The safety factor of the bearing capacity of the soil compared to RakMk B3:

– the calculated safety factor is 1.39, which is less than the required safety factor of 2.0. In spite of the theoretically low safety factor of the bearing capacity the structure is acceptable, according to the principles of ISO 13822:2010. The superstructures have loaded the subsoil for centuries.

Soil-foundation-pile interaction in ULS (Axis B), Section 5.2

The safety factor of piles in SLS:

– the calculated piled load in ULS is 244.9 kN
– the partial safety factor for the loads of the superstructure is 1.4
The real safety factor in SLS is
\[ 1.4 \times \frac{319.2 \text{kN}}{244.9 \text{kN}} = 2.37, \] (124)

The safety factor of the bearing capacity of the soil in SLS:

- the calculated safety factor is \( 1.4 \times 1.25 = 1.75 \).

**Soil-foundation-pile-slab interaction in ULS (Axis B), Section 5.3**

The safety factor of piles in SLS:

- the calculated pile load of foundation I in ULS is 232.1 kN
- the real safety factor is

\[ 1.4 \times \frac{319.2 \text{kN}}{232.1 \text{kN}} = 2.50, \] (125)

- the calculated pile load of foundation III in ULS is 214.8 kN
- the real safety factor is

\[ 1.4 \times \frac{319.2 \text{kN}}{214.8 \text{kN}} = 2.70 > 2.2. \] (126)

The safety factors for the bearing capacity of the soil are \( \gamma_{kI} = 1.82 \) and \( \gamma_{kIII} = 1.89 \).

The safety factors of piles, the safety factors of foundations and piled foundations in SLS

The safety factors are summarized in Table 5.

**Table 5. Summary of safety factors in SLS.**

<table>
<thead>
<tr>
<th>Chapter</th>
<th>( \gamma_k ) of pile</th>
<th>( \gamma_k ) of foundation</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.1. Soil-foundation SLS</td>
<td>2.37</td>
<td>1.39</td>
<td>The safety factor of piles is same because of linearity.</td>
</tr>
<tr>
<td>5.2. Soil-foundation ULS</td>
<td>2.37</td>
<td>1.75</td>
<td>The safety factors of foundation are different because of non linearity ( (q_{ur} &gt; 0.70 q_u, \text{Fig. 26}) ).</td>
</tr>
<tr>
<td>5.3. Soil-foundation-pile slab interaction ULS</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foundation I</td>
<td>2.50</td>
<td>1.82</td>
<td>Foundation I is influenced by slab II</td>
</tr>
<tr>
<td>Foundation III</td>
<td>2.70</td>
<td>1.89</td>
<td>Foundation III is influenced by slabs II and IV</td>
</tr>
</tbody>
</table>

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6. Safety factor in geotechnical and structural design

Soil-foundation-pile design calculations can be made in the same geotechnical and structural process using the method described in ULS in Section 5.2. With the use of the author’s method the settlement \((S_{Sk})\) in SLS is 11.7 mm (Section 5.1) and the settlement in ULS \((S_{Sd})\) is 16.3 mm (Section 5.2). As previously mentioned \((S_{Sk})\), \((S_{Sd})\) the partial safety factor is 1.4 for the superstructure.

The ratio of settlements in ULS \((S_{Sk})\) and in SLS \((S_{Sd})\) is

\[
\frac{16.3 \text{ mm}}{11.7 \text{ mm}} = 1.39.
\]

(127)

The non linearity of soil \((q_{df} \sim 0.8 q_u > 0.7 q_u)\) has no practical influence and thus the ratio of \(S_{Sk}\) and \(S_{Sd}\) is practically the same as the partial safety factor of 1.4 for the superstructure \(S_{Sh}\) and \(S_{Sh}\).

7. Soil-foundation-pile and soil-foundation-pile-slab design in ULS

In this study the author has developed a practical manual calculation method which can be used for estimating soil-foundation interaction and soil-foundation-pile-slab interaction, or which can be used directly for the design of piled floating strip foundations or to check computer results. This rational method proves that a piled floating strip foundation can be designed geotechnically and structurally using a single method based on ultimate limit state (ULS) and serviceability limit state (SLS). The kinematic method of plasticity method simplifies structural analysis. For SLS checks the author's method takes into account the settlements and angular distortions. It may be possible to design a piled floating raft foundation using the method described by the author.

As mentioned earlier, if the soil strata are not homogenous the author’s method may be used with a greater safety factor. In principle the author’s method of “separate foundations” for soil-foundation-pile interaction or piled floating strip foundations automatically takes care of various limit states (ULS and SLS).

As noted in Sections 6.2.5 and 6.2.6 of thematical conclusions the safety factors of piles of Soil-Foundation-Pile interaction are linear. The theoretical safety factor of the bearing capacities of foundations is slightly different because of non linearity in Fig. 26. In his Licentiate Thesis (Avellan 1992) and in an article (Avellan 1994) based on the Licentiate Thesis (Avellan
1992), the author has shown that with the use of the author’s method the bending moments in SLS are nearly the same in spite of the calculation according to the method of foundation on Winkler-springs, compressibility method or the author’s method.

8. Strengthening of foundations of historic buildings

In many cases, and also in this case, it was not possible to clarify the influence of the existence time and weight of massive old structures on the soil. There is seldom a real opportunity to make the structural and geotechnical ground design ready before beginning the strengthening of the foundations. The safety factors during the work and after it are often lower than in the construction of new buildings. Monitoring must be done during the work and the engineer must be ready to change the designs or make fully new design solutions. Monitoring after the work is sometimes necessary. In this work the levelling (10.5.2011) proved that the strengthening work fulfilled the targets.
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Appendix 1 Drawings of St. John’s Church in Tartu made by KAREG Consulting Engineers

Fig. 1. Layout of St. John’s Church in Tartu, left half.
Drawings of St. John's Church of Tartu

Fig. 2. Layout of St. John's Church in Tartu, right half.
Drawings of St. John’s Church of Tartu

Markings to layout drawing of St. John’s Church in Tartu (Fig. 1 and Fig. 2):

+ Jacked piles / spiral drilled piles \( \varphi \) 218 x 10 + filled with concrete
  - Tower \( F(\text{used}) \leq 400 \text{ kN}, \quad F(\text{test}) = 650 \text{ kN} \)
  - Line B&C / 4...7 \( F(\text{used}) \leq 300 \text{ kN} \)
  - Other parts \( F(\text{used}) \leq 400 \text{ kN} \)

JP+ Jacked piles \( \varphi \) 140 x 8 filled with concrete
  \( F(\text{used}) = 100...150 \text{ kN}, \quad F(\text{test}) = 1,5 \times F(\text{used}) \)

\( \bowtie \) Post-tensioned anchor 5 x (1630 / 1860 SUP \( A_e = 140 \text{ mm}^2 \))
  \( P_e = 750 \text{ kN} \) (Prestressing force at the stressing end after wedge draw-in)

\( \longrightarrow \) New concrete beams / -structure

\( \rightarrow \rightarrow \) Steel rails (P65) + concrete cover

h = 600 Reinforced concrete raft foundation, height 600 mm

UL = upper level
Drawings of St. John's Church of Tartu

Fig. 3. Installation of screw equipment to excavation (applicable outside line A).
Fig. 4. Screw equipment from side.
Fig. 5. Section of line B in St. John’s Church.
Appendix 2 Soil investigations (Map and three sections)
Appendix 3 Spiral pile

Drilled spiral pile of St. John's Church of Tartu
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LIMIT STATE DESIGN FOR STRENGTHENING FOUNDATIONS OF HISTORIC BUILDINGS USING PRETESTED DRILLED SPIRAL PILES WITH SPECIAL REFERENCE TO ST. JOHN’S CHURCH IN TARTU