Steel pipe piles

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FOREWORDS

Use of steel pile piles in bridge foundation has increased after year 1989, when the test application of instruction Steel pipe piles, TVH 723448, begun. Since then more experience and investigation data have been obtained about design, manufacturing and driving of steel piles. These experiences are transferred to this revised instruction.

The members of the committee treating the alternations and amendments of the instructions were following: M.Sc. Matti Kuusivaara (chairman), M.Sc. Jouko Lämsä and engineer Esko Palmu from the Brigde division of Road Administration, M.Sc. Pentti Salo from the Geoservice Centrum of the Road Administration, engineer Kari Väliaho from Turbular Products and Sections Division of Rautaruukki Ltd. and professor Jorma Hartikainen and M.Sc. Juha Heinonen from the Geotechnical Department of Tampere Univeristy of Technology.

The instruction is written as a consulting task by GT-Geotieto Ltd. The instruction is based on Aki Hyrkkönen's M.Sc. Thesis Geotechnical Bearing Capacity of Large Diameter Steel Pipe Pile and Mauri Koskinen's licensiate thesis Lateral Capacity of Steel Pipe Piles.

In revised instruction experiences obtained from performed steel pipe pilings are taken into account. Especially the experiences from Tähtiniemi Bridge in Heinola are collected systematically. Research work in mainly carried out in Geotechnical Laboratory of Tampere University of Technology. Following M.Sc. Thesis and publications are used in complementing the instruction: Juha Heinonen, Pile Driving at Tähtiniemi Bridge, 1992; Sami Punkari, Supervision of Steel Pipe Pile Driving, 1992; Minna Leppänen, Corrosion of Steel Piles, 1992; Hannu Jokiniemi, Rock Shoes for Steel Pipe Piles, 1992; Jorma Hartikainen, Bohdan Zadroga and Mauri Saari, Plugging of Open Ended Steel Pipe Piles, Own Test Results, 1992. The research has been finanzed by Road Administration, Rautaruukki Ltd. and GT-Geotieto Ltd.

In Helsinki in December 1992

Bridge center

FOREWORDS OF THIRD EDITION

The contents of this thirdh edition is the same as in the second edition in 1993. In the instructions "Sillansuunnittelun täydentävät ohjeet" (The supplement bridge design instructions) minor completions to the text has been published. Some revisions to the list of litterature and references has been made.

The English translation is made in Tampere University of Technology by M.Sc. Minna Leppänen. The grammatical control in made by James Rowland.

In Helsinki in March 2000

Road Addministration
Bridge Engineering

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DEFINITIONS

1) Steel pipe pile
The pile is composed of a steel pipe, the diameter of which is more than 300 mm. Steel pipe pile types differ from each other according to the structure of the pile point, pile shaft and the pile driving method to be used. If a steel pipe pile is filled with concrete and adhesion between steel and concrete is sufficient, the structure can be assessed as a composite structure.

2) Unplugged open-ended steel pipe pile
This pile is a steel pipe, which is open at both ends and is driven into the ground with blows to the top of the pile. After the pile driving the ground level is approximately the same both inside and outside the pile.

3) Plugged open-ended steel pipe pile
This pile is a steel pipe, which is open at both ends and is driven into the ground with blows to the top of the pile. On completion of the pile driving the ground level is distinctly lower inside than outside the pile. The state of plugging of the pile is determined on the basis of the difference between the ground levels inside and outside the pile. Normally formation of the plug requires, that the pile penetrates into the plugging soil layer not less than $10 \cdot d$ length, where d is the diameter of the pile.

4) Steel pipe pile closed with bottom plate
This pile is composed of a steel pipe, which acts as a pile shaft, and a plate welded to the lower end of the steel pipe. This plate seals the lower end of the pipe. The pile is intended to penetrate into bearing soil layer. The pile is installed using a pile hammer, which delivers blows to the top of the pile, or by using a Franki pile hammer.

5) Steel pipe pile with rock shoe
This pile is composed of a steel pipe and a rock shoe welded to the lower end of the pipe. The function of the rock shoe is to transmit the pile load to the rock and prevent sliding of the point. The rock shoe should tolerate loading during the use of the structure and pile driving. The steel pipe pile with a rock shoe is driven with a pile hammer, which delivers blows to the top of the pile, or with a Franki pile hammer, which delivers blows to the lower end of the pile.

6) Franki pipe pile
The difference between a Franki pipe pile and a Franki pile is that in the Franki pipe pile the steel tube used as a working tube remains as a permanent structure.

7) Concrete pile with steel casing
   The installation of the pile pipe requires no fulfilling of the settlement limits during the final set and the geotechnical bearing capacity of the pile is assessed according to the cast-in-place piles.

1 SCOPE OF APPLICATION

These instructions are intended as a guidance in the design and performance of pile driving of steel pipe piles whose diameter is in excess of 300 mm. Pile driving is performed using a drop-hammer equipped piling rig or other driving device. The driving energy of the piling rig has to be sufficient to enable that the bearing capacity of the piles can be verified not only on the basis of penetration depth but also on the basis of used driving force and force resistance of the pile point, when the piles can be considered as driven piles. The principles applied in this instruction are in accordance with the pile driving class I of Finnish pile driving instructions LPO-87 /16/.

In special cases different methods of pile driving can be used, especially when driving open pipes. If the driving energy transferred to the pile from the piling rig is not sufficient to verify the bearing capacity or it cannot be detected, verification of the bearing capacity of the pile is then based only on the penetration depth. In such cases the piles should normally be empty of soil and filled with concrete. This kind of pile is regarded as a friction pile or cast-in-place pile, which penetrates into the moraine layer. In such case the assessment of the geotechnical bearing capacity is carried out according to paragraphs 4.1.1 and 4.1.2 of this instruction.

With regard to pile production, foundation design and pile driving the instructions "Suomen Rakentamismääräyskokoelma" (Collection of Finnish Building Regulations) and general occupational safety standards and directions should be consulted. Notably;

- Finnish Association of Civil Engineers: "Pohjarakennusohjeet" (Foundation Instructions) 1988 RIL 121

- Finnish Geotechnical Society: "Lyöntipaalutusohjeet" LPO-87 (Pile Driving Instructions), "Suurpaaluohjeet" SPO-78 (Instructions for Large Diameter Piles) or other valid instructions for large diameter piles

- Finnish Concrete Society and Steel Structure Society: "Liittorakenteet, suunnitteluohjeet" (Composite Structures, Design Instructions) 1988

- Finnish National Road Administration: "Siltojen pohjatutkimusohje" (Instruction for Soil Survey for Bridges) (TIEL 3200537), "Pohjarakennusohjeet sillansuunnittelussa" (Foundation Instructions in Bridge Design) (TIEL 2172068-99), "Siltojen kuormat" (Loads on bridges) (TIEL 2172072-99) and "Teräsrakenneohjeet" (Instructions for Steel Structures) (TVH 723449) as well as "Sillanrakentamisen yleiset laatuvaatimukset" SYL (General Quality Requirements for Bridge Construction), "Sillanrakentamisen yleinen työselitys" SYT (General Specifications for Bridge Construction) and "Sillanrakentamisen valvontaohje" SVO (Supervising Instructions for Bridge Construction).

2 SOIL SURVEY

2.1 General requirements

When undertaking the construction project, a sufficiently thorough soil survey has to be performed in accordance with various design stages. First, for the selection of the foundation type, secondly, for planning and building of the foundation structures and other foundation engineering works.

Soil survey is performed according to reference /31/ (TIEL 3200537).

2.2 Effect of the pile performance

2.2.1 Point bearing piles

When point bearing pile foundations are used, the location of the rock surface and the structure of the upper rock layer is determined with soundings. The location and shape of the rock surface is especially investigated, when cohesive layers extend to the rock surface, when there is a loose coarse grained soil or moraine layer on an inclined rock surface and when the dense coarse grained soil or moraine layer on an inclined rock surface is thin. If the coarse grained soil or moraine layer is sufficiently dense and thick, in order that piles reach the sufficient geotechnical bearing capacity without penetration into the rock, the soundings can be finished in the hard base layer.

In general the location of the rock surface has to be determined with percussion drilling. To define the pile penetration and density of the soil layers dynamic probing has to be carried out.

When point bearing piles which extend to the rock are used, the location of the rock surface is always determined at each pile group with percussion drilling. In special cases it is necessary to clarify the upper rock structure with rock core borings.

2.2.2 Friction piles

When friction piles are used, the boundaries between the soil layers and the density of the layers, are determined, both for the penetrated layers and especially for the pile bearing layers.

Soil survey is mainly carried out using dynamic probing, standard penetration tests or cone penetration tests. During the selection of investigation methods and surveying operations it is essential to detect and localize possible cohesive soil layers between cohesionless soil layers.

Soil samples must be obtained from the soil layers that the piles are resting on to facilitate the determination of the grain size distribution. In order to determine the shaft friction it is recommended that the strength parameters of the soil are investigated utilizing, for example, triaxial testing.

A soil plug can develop at the point of the open-ended pipe pile during pile driving. If the soil plug is exploited in the design of the geotechnical bearing capacity of the pile, soil samples must be taken from the plugging soil layer to determine at the minimum the grain size distribution, but rather also the strength parameters.

2.2.3 Laterally loaded piles

When the lateral capacity is utilized or imposed loads of the structural system cause lateral loading to piles, it is especially necessary to investigate the strength and the deformation parameters of the soil layers that will support the upper part of the pile.

In fine and organic soil layers strength values are determined using vane soundings, and in coarse soil layers and moraine layers indirectly using the density definition based on the soil type determination and soundings or triaxial tests on samples.

2.2.4 Tension piles

Where piles are used, that are permanently or repeatedly subjected to larger tension loadings than the effective weight of the pile, the friction and adhesion between the pile shaft and soil layers has to be determinable from the basis of the soil survey.

Long term tension can be permitted only for piles in coarse or moraine layers. For this purpose the soil survey corresponding to the survey required for friction piles is needed. Transient tension can be permitted also in cohesive soil layers. For this purpose the vane sounding results are necessary to determine the adhesion of the pile shaft.

2.2.5 Negative shaft friction

In that part of the pile shaft, where fine grained or organic soil layers subside over 5 mm more than the pile, for example due to the filling, the adhesion force between the pile shaft and soil layers must be determined to enable the assessment of negative shaft friction.

Consolidation values determined with ödometer tests made on undisturbed samples are required to define the settlements. To define the cohesion, the strength values determined by vane soundings are needed.

2.2.6 Corrosion investigations

If steel pipe piles are used, the potential corrosion risk has to be investigated.

The degree of aggressivity of the natural and homogeneous soil is normally minimal. Notable corrosion may occur only in aerobic conditions. Aerobic conditions occur above the lowest design ground water level described in reference /20/ (RIL-121).

The aggressiveness of the soil must be investigated with corrosion measurements if there is organic soil, fillings, or sulphur clay in the area, or if water surrounding the pile is contaminated. Especially aggressive can be such soils, which have a low specific resistivity and pH-value.

Several factors affect the soil corrosion thus making the joint effect difficult to estimate. Corrosion investigations can examine the effect of one, individual factor. The most important factors are the moisture content, the amount of organic material, the acidity, the specific resistivity, the chemical composition of the pore water as well as the location and variation of the ground water level.

The specific resistivity is determined in the field using Wenner´s four electrode method or a rod electrode. In the laboratory, the specific resistivity can be determined using the soil box method or the insertion electrode method. Various corrosion probes can also be used in corrosion investigation or applicable electrochemical measurements can be performed.

During the soil survey, extra soil samples should be obtained to facilitate laboratory corrosion investigations. With regard to the handling and storing of samples, special care should be taken to prevent disturbance and oxidation. Laboratory tests should be carried out as soon as possible to avoid altering of the properties of the samples.

Electricity plants, power lines and electric rail traffic in the vicinity of the building site may cause stray current corrosion, the amplitude of which can be evaluated by measuring the leakage current and conductivity of the ground. Precautions for the changes in the corrosion environment during the planned working time of the piles must be taken, if necessary, by reserving the protected area or other measures available for the builder.

In soils containing sulphur, i.e. sulphide clays, microbiological corrosion may occur in anaerobic conditions. Microbiological corrosion can be evaluated by investigating the quantity of species and the activity of the microbes in the soil. Sulphide clays may appear in an area of Litorina clays and occasionally in Ancylus clays.

On the basis of the corrosion investigations an appropriate corrosion protection method is selected. These are discussed later in paragraph 5.3.

3 SELECTION CRITERIA FOR STEEL PIPE PILES

In selection of pile types which differ in performance, the soil conditions on the building site and the requirements set by the structures primarily affect the decision.

3.1 Pile performance

3.1.1 Point bearing pile

A point bearing pile transfers the major part of its load through its point to the rock or a dense base layer. Part of the load can be transferred to the surrounding soil layer through the shaft friction (*figure 1*).

> Generally, in the design of the pile foundation the technical and economical applicability of point bearing piles should first be determined. When large diameter steel pipe piles are used as point bearing piles, the strength of the material can be exploited effectively.

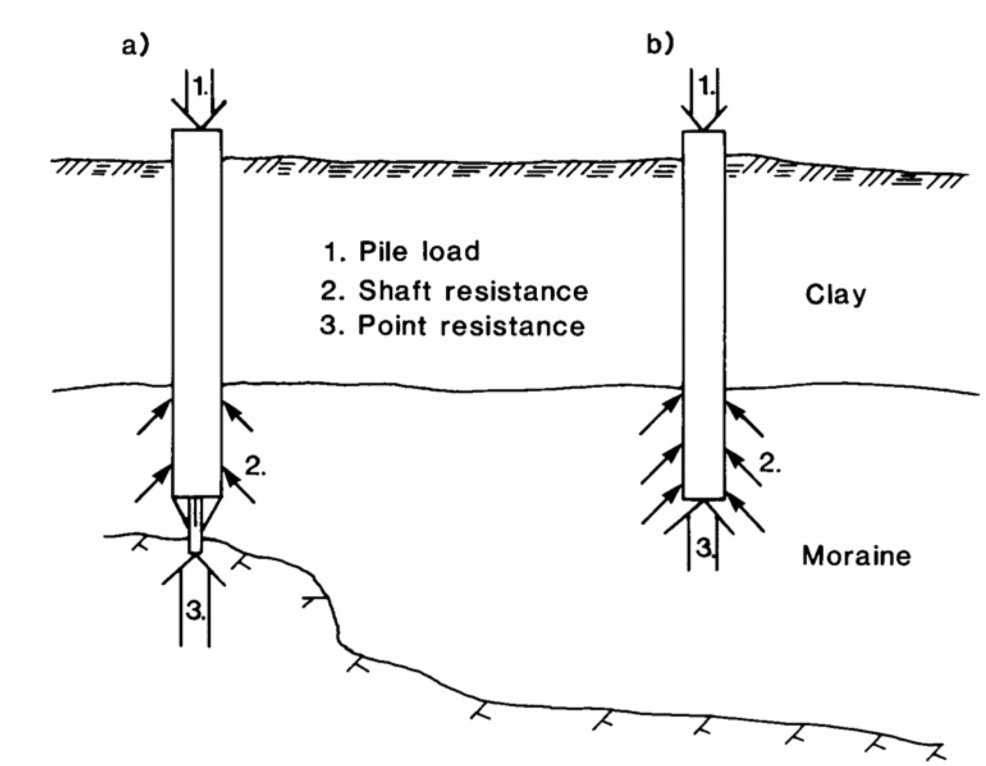

Figure 1: a) Action of the point bearing pile resting on the rock.
b) Point bearing pile supported by the ground /5/.

3.1.2 Friction pile

A friction pile transfers the major part of the load through the shaft friction to the soil layers. Line of action of an open ended friction pile can be outlined in two different cases:

1) The friction pile transfers the major part of the load to the surrounding soil layer through the external and internal shaft friction. A part of the load is transferred through the point of the pipe (*figure 2a*).

2) The friction pile transfers part of the load to the surrounding soil layer through the external shaft friction. A part of the load is transferred through the soil plug developed in the end of the pile. The soil plug is developed due to the effect of friction between the soil forced into the pile and the internal shaft surface (*figure 2b*).

   The application of friction pile is appropriate, when the coarse grained soil or moraine layer on the rock or on a dense base layer is thick.

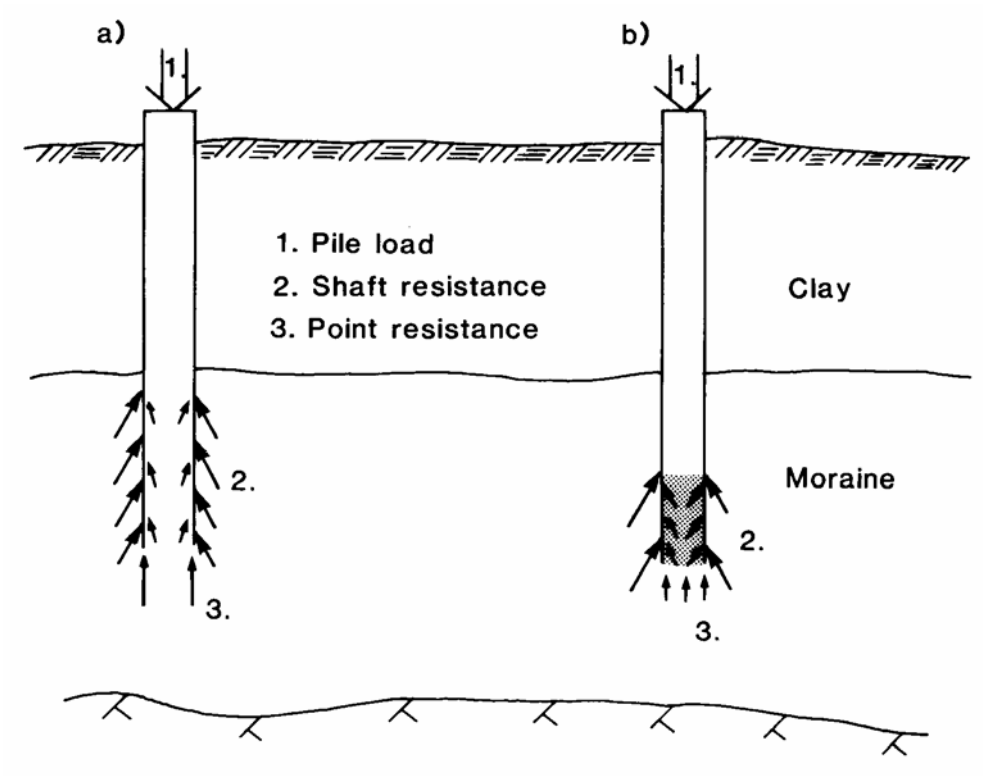

Figure 2: Action of the friction pile /5/.
a) Internal and external shaft friction.
b) External shaft friction and soil plug.

3.1.3 Cohesion pile

A cohesion pile transfers the load through the adhesion developed on the shaft surface. The point resistance is small (*figure 3*). A structure founded on a cohesion pile usually settles, because the piles are loading compressive soil layers. Permitted settlements of the structure and the evenness of actual settlements determine the applicability of the solution.

Use of cohesive piles is only possible in permanent structures in such cases as, when the cohesive layer is especially thick or hard. There are no such soil layers in Finland, in which the use of steel piles as cohesion piles would be appropriate.

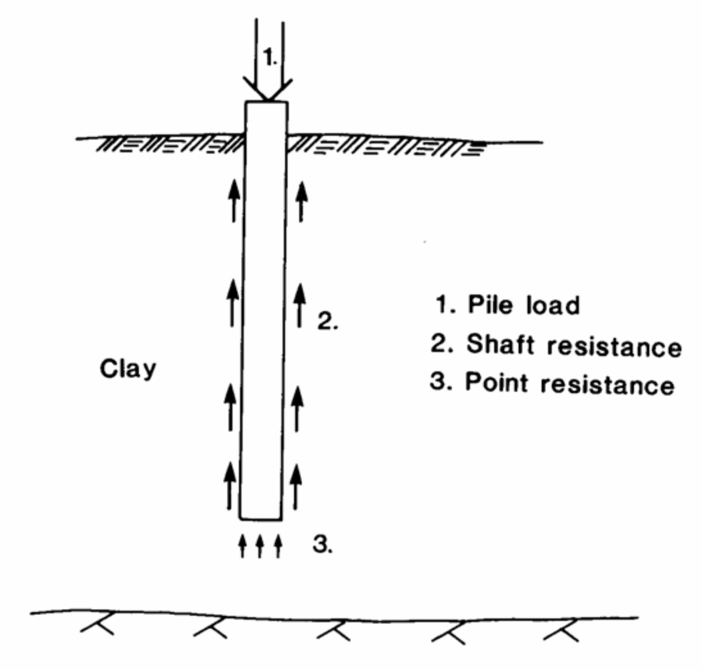

Figure 3: Action of the cohesion pile /5/.

3.2 Pile types applied in different soil and environmental conditions

Large diameter steel pipe piles are suitable for many difficult application environments and structures, such as;

- for large pile loads,
- for bending loads,
- for difficult soil conditions,
- for water structures, especially harbour structures and bridges,
- for piling in the vicinity of sensitive structures, open piles are especially suitable.

Steel pipe piles especially acting as a composite structure sustain large compression and tensile stresses.

When the soft soil layers are thick or the water depth is considerable, there is a demand for a good buckling resistance of large diameter steel pipe piles. This enables the designing of the pile cap to be close to the water level.

If a caisson is applied, the pile cap can be cast in dry conditions. When there are dense soil layers, which must be penetrated, or a large geotechnical bearing capacity has to be reached, the good penetration of steel pipe piles is a remarkable advantage. In water engineering the easy handling and especially the possibility to float the piles are important advantages. In addition, the applicability of working from the float of the steel pipe pile driving equipment, especially of the hydraulic and diesel hammers, is significantly advantageous.

If there are sensitive structures in the vicinity, use of open-ended piles can reduce vibrations and displacements developed during pile driving and diminish the decreasing of the strength of the soil due to the disturbances and the increase of the pore water pressure. Even the close-ended piles cause less soil displacements in relation to the bearing capacity than the driven concrete piles.

3.2.1 Close-ended piles

Close-ended piles include steel pipe piles equipped with a rock shoe or a bottom plate and Frank pipe piles closed with a concrete plug at the lower end.

Closed-ended piles are recommended mainly when piles are resting on a stony moraine and always when the piles are resting on rock.

3.2.1.1 Piles equipped with a rock shoe (figures 4a and 4b)

The purpose of the rock shoe is to prevent the sliding of the pile point on the sloping rock surface. In addition, the use of the rock shoe is intended to centralize the load of the pile at the pile point and reduce the development of the bending moment straining the pile.

A rock shoe is used in such cases, as when the pile point rests on the rock or the pile is driven into the bouldery moraine. A pile with a rock shoe can be driven from the top or from the lower end. The rock shoe is discussed in more detail in paragraph 6.2.

If the rock surface is steeply inclined and the supporting coarse grained soil or moraine layer is thin, the pile can be equipped with steel peg drilled through the rock shoe to ensure reliable resting on the rock.

3.2.1.2 Piles equipped with a bottom plate (figure 4c)

The bottom plate ensures the bearing of the pile point on the total cross section area of the pile.

> A bottom plate is used in such cases, as when the bearing layer above the rock is sufficiently thick as to eliminate the need to be in contact with the rock.

3.2.1.3 Franki pipe piles (figure 4d)

A Franki pipe pile is closed at the point with a moist concrete plug. Franki pipe piles enable the use of large driving force and provides simultaneously good penetration without the limiting influence of the structural capacity of the pipe pile.

> A Franki pipe pile equipped with a concrete plug is suitable for the same conditions as a pile equipped with a bottom plate.

3.2.2 Open-ended piles

Open-ended piles include open-ended plugged piles, open-ended unplugged

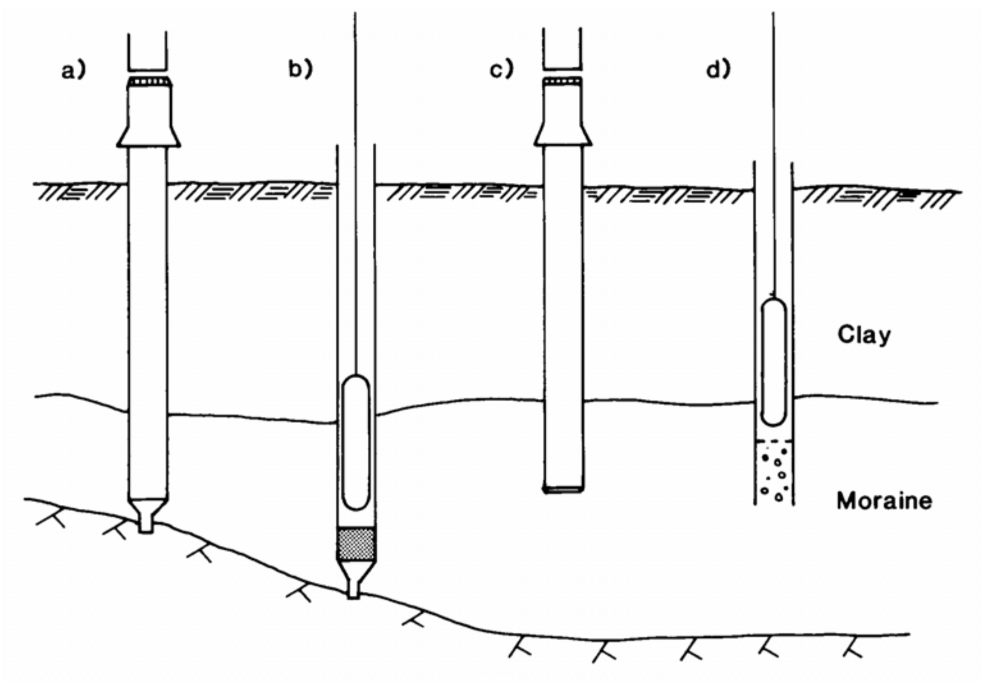

Figure 4: a) Pile equipped with a rock shoe and driven from the top of the pile.
b) Pile equipped with a rock shoe and driven from the lower end of the pile.
c) Pile equipped with a bottom plate and driven from the top of the pile.
d) Franki pipe pile.

piles and concrete piles with steel casings. An open-ended pile can not be extended to the rock.

The soil mass displaced by the open-ended unplugged piles is small and the disturbance of the soil is insignificant, thus they are suitable for use in the vicinity of sensitive structures. In addition, pile driving is less laborious and the stresses passing through the shaft are smaller than in close-ended driven piles.

Open-ended piles are used primarily as friction piles. Requirements for the use of the open-ended piles are, that the coarse grained soil or moraine layer on the rock is thick with no significant amounts of stones or boulders. Because the open-ended piles are not extended to the rock, the pile must reach the sufficient geotechnical bearing capacity in soil layers.

3.2.2.1 Plugging pile (figure 5a)

The plugging pile is suitable when a soil plug develops inside the pile due to the influence of the friction. The pile acts similar to the close-ended pile. Recommended applications of open-ended and plugging pile are represented in paragraph 3.3.

An open-ended pile is a geotechnically favorable pile, but its use requires confirmation of the plug formation. Plugging may occur, if the ratio of the thickness of the plugging soil layer to the pile diameter is sufficient and if the plugging soil layer is medium dense or dense, sufficiently well graded and of poor silt content. If the formation of the plug can not be confirmed in connection of the pile driving, it has to be confirmed with soundings. If the sufficient formation of the plug formation cannot be confirmed, piles should be designed unplugged and extended deeper.

3.2.2.2 Unplugged pile (figure 5b)

An unplugged open-ended pile can be used in such cases, in which plugging does not occur and due to the prevailing conditions, a close-ended pile is unsuitable.

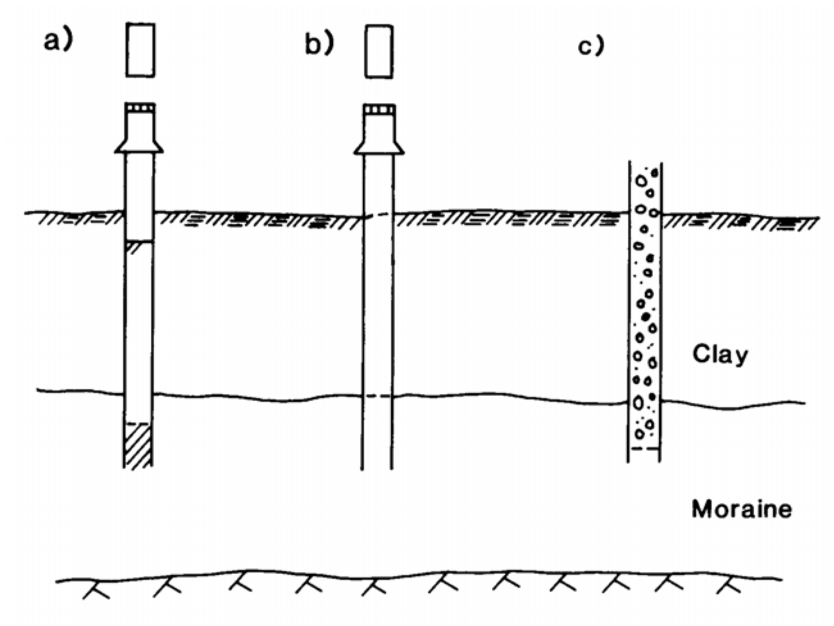

Figure 5: a) Open-ended plugged pile.
b) Open-ended unplugged pile.
c) Concrete pile with a steel casing.

3.2.3 Concrete pile with a steel casing (figure 5c)

A concrete pile with a steel casing is used in such cases, as when the steel pipe is driven with the piling rig, the driving energy of which cannot be determined. Soil must be scooped out from the steel pipe.

However, a thin dense layer of soil can be left at the lower end of the pile. The pile is concreted in accordance with the underwater work regulations. A concrete pile with a steel casing can be considered to have a bearing capacity equal to a cast-in-place pile resting on moraine or acting as a friction pile. The shaft friction is at maximum 70% of the shaft friction of the cast-in-place pile with corresponding size.

> When a concrete pile with a steel casing is applied, the pipe can be driven by hammering, vibrating, pressing or friction. The wall of the pipe in the concrete pile with a steel casing can be thinner and steel quality can be lower than in the steel pipe pile.

3.3 Environmental effects of pile driving

Environmental effects of pile driving are land subsidence or heaving and displacements, disturbance of soil layers, increase of pore water pressure, vibration and noise. Environmental effects must remain within permitted limits and must be ensured with continued control measurements, when necessary. Environmental effects can be reduced by the selection of an appropriate pile type. The use of open-ended steel pipe piles is recommended where there are structures sensitive to soil displacement, as in harbours and sites which are located near existing buildings.

3.3.1 Disadvantages caused by ground displacement

Disadvantages caused by ground displacements can be avoided using open-ended steel pipe piles, thus minimizing the displaced area the latter being equal to the steel cross section before plugging begins.

However, close-ended piles displace less soil in relation to their bearing capacity than concrete piles.

3.3.1.1 Land subsidence

When piles are driven into a loose coarse grained soil layer, the driving causes densification of the soil, which in turn results in the subsidence of the ground surface in the piling area.

Subsidence may cause settlements of adjacent structures. The volume of the subsidence depression may attain 50% of the volume of the piles driven into the ground. Subsidence appears at distances of 1/4...1 times the length of the pile in the ground. In loose cohesionless soil it is advantageous to start pile driving from the most risky place. Land subsidence and settlements of surrounding buildings are controlled using levelling techniques.

3.3.1.2 Horizontal displacement of soil layers and land heaving

When piles are driven into a fine-grained soil layer, driving causes soil displacements, because clay or silt does not compact. This results in heaving of the ground surface and horizontal displacements of soil layers.

The volume of the rise of the ground may correspond to the volume of the piles driven into the ground. Land heaving appears in area which extends to approximately the depth of the clay or silt layer outside the pile driving area. Pile driving should be started from the most risky place. Displacement may cause damage to adjacent pipes or even to buildings. Land heaving and rising of the piles and surrounding structures is controlled using levelling techniques. Displacements of the piles and surrounding structures are controlled with displacement measurements.

3.3.1.3 Increase of pore pressure

Pile driving into the soil layers causes increasing of pore water pressure, which reduces the shear strength of the ground.

In undulating terrain or in the vicinity of slopes or excavations the pore pressures generated by pile driving should be measured and taken into account in stability calculations. Development of the pore water pressure can be limited by using open-ended piles or by minimizing the cross section of close-ended piles. An increase of the pore water pressure can also be reduced by:

-removing fine grained soil layers from the location of the pile, e.g. with a suction pipe,
-by equipping the piles with vertical drainage or using the vertical drainage for the piling area, or
-by dividing the pile driving into periods or by lengthening the duration of the pile driving.

3.3.1.4 Disturbance of soil layers

Pile driving into fine-grained soil layers causes a reduction of the strength due to the disturbance. The strength returns slowly, but only partially in overconsolidated soil layers.

In undulating terrain or in the vicinity of slopes or excavations a loss of the shear strength due to the pile driving should be taken into account in stability calculations. The shear strength should be observed with vane sounding during the pile driving. Disturbance can be prevented primarily by using open-ended piles or by limiting the gross cross section of the close-ended piles. Further disadvantages can be reduced by removing the fine-grained soil layers from the pile location. More time for remediation of the strength can be given by dividing the pile driving into periods or by lengthening the duration of the pile driving.

3.3.2 Vibration damage

3.3.2.1 Vibration caused by pile driving

The risk of damaging buildings due to vibrations can normally be evaluated on the basis of the maximum vertical velocity of oscillation.

Vibration frequencies caused by pile driving vary between 2...50 Hz. The largest vibrations occur in the frequency range 5...20 Hz.

The maximum vertical velocity of the oscillation of the soil surface due to pile driving, which can be used in preliminary studies, can be derived from formula 0:

$$v_{\max} = 1,5 \frac{\sqrt{W}}{r} \qquad (1)$$

v_{max} = maximum vertical velocity of oscillation, [mm/s]
W = driving energy, [Nm/blow]
r = distance from the pile to the measuring point, [m]

The influence of soil type and the degree of saturation on vibration propagation is not taken into account in the formula. Vibration travels furthest in soft clay and silt, where the water content is high. In gravel and moraine the vibration damps out most rapidly.

The velocity of vibration oscillation transmitted to a building located in the vicinity of the pile driving site is according to some observations approximately 10...60 % from the values derived from formula (1) depending on the foundation type of the building.

The maximum permitted velocity oscillation values for buildings are presented in *table 1*.

Table 1. The maximum permitted vertical velocity oscillation values for buildings /16/.

Class of the building	Quality of the building	Maximum vertical velocity of oscillation [mm/s]
1	Old historical buildings	2
2	Cracked buildings, brick buildings	5
3	Buildings in good condition without damage	10
4	Very strong buildings	10...40

The relation between the velocity of the oscillation and the risk of damage is dependant upon, in addition to the quality of the building, local conditions, i.e. soil conditions, and properties and the duration and frequency of the vibration.

3.3.2.2 Controlling and reducing vibration level

The vibration level is controlled with vibration measurements in structures and possibly in devices.

In general it is sufficient to measure the velocity of the oscillation, but in demanding cases and when the devices are controlled the accelerations and frequency should be measured. Vibration measurement points are installed to the bearing structures of the building near to the pile to be driven. Installation of strips made from gypsum or tapes to the sensitive places is recommended.

Level of vibration can be reduced primarily by minimizing the cross section area of the piles. Other measures for preventing vibrations are:

- use of an effective piling rig, which is in good condition and suitable for the pile type in question,
- keeping the hammer blows central and parallel with the axis of the pile,
- avoiding unnecessary blows by designing the target depth of the pile and the pile driving instructions carefully,

- drilling the initial hole for the pile through the filling, frost or a dense surface layer and
- avoiding the use of a vibration hammer in cohesive soils.

3.3.3 Noise problems

3.3.3.1 Noise level caused by pile driving

Calculation methods for assessing the noise level caused by pile driving are not available. Maximum permitted noise level L_{Aeq} may be 80 dB in areas open to the general public.

Table 2 indicates some instructional values, which can be used in the evaluation of the noise level caused by different types of piling hammers.

Table 2. Noise caused by different piling hammers and its damping in open terrain.

Piling hammer	Noise level at 10 m distance [dB]	Estimated distance, where noise level is 80 dB [m]
Drop hammer	90...100	40...160
Vibration hammer	90...100	40...160
Diesel or hydraulic hammer	100...105	160...320
Franki piling hammer	80...90	10...40

3.3.3.2 Controlling and reducing noise level

Noise level can be reduced by isolating the noise source and using suitable cushioning in pile driving.

Noise levels have been reduced approximately 20 dB by installing noise shields to the piling rigs. The shield also prevents oil splashing and the spreading of exhaust gases.

Control of the noise level is performed with sound level measurements.

4 GEOTECHNICAL BEARING CAPACITY

The bearing capacity of a pile is determined as follows: the pile must sustain with sufficient certainty loadings in different loading cases after driving, and settlements and horizontal movements must be within the permissible structural tolerances.

The bearing capacity of the pile is determined either based on the structural or geotechnical bearing capacity, and the smaller one is chosen to the design capacity.

The geotechnical bearing capacity is determined according to the ground conditions, the stipulated requirements for the piling work and the checking procedures.

The geotechnical bearing capacity of the pile consists of the bearing capacity of the pile point, i.e. point resistance, and of the bearing capacity of the pile shaft, i.e. shaft resistance. The mobilization of the point resistance requires a considerably larger settlement than the mobilization of the shaft resistance. The effects of the negative shaft friction and plugging of the open-ended pile on the geotechnical bearing capacity are checked separately, when negative shaft friction is developed or the pile is plugged. The possible corrosion of the pile does not lower the geotechnical bearing capacity.

4.1 Determination of bearing capacity

This instruction is mainly concerned with driven piles, whose geotechnical bearing capacity is determined both on the basis of the penetration depth of the piles and final set. Concrete steel piles with a steel casing, where the steel casing is installed using driving energy, which is not sufficient from the standpoint of the determination of the pile bearing capacity, are treated as special cases. The bearing capacity of these piles is determined on the basis of the penetration depth or according to the instructions concerning cast-in-place piling.

The geotechnical bearing capacity of the pile can be determined in many different ways. The methods can be roughly divided into direct and indirect methods. Indirect methods include:

- static bearing capacity formulas,
- empiric methods based on the sounding resistance,
- dynamic pile driving formulas and
- stress wave analysis without stress wave measurements on the building site.

Direct methods include:
- dynamic test loadings based on stress wave theory and
- static test loadings.

In design stage the indirect methods are used in designing of the pile size, penetration depth and dimensions of the driving device. These are checked on the site using direct methods, usually with dynamic test loadings.

The bearing capacity of the close-ended pile consists of the point resistance and external shaft resistance. The bearing capacity of the open-ended pile consists of the point resistance, internal shaft resistance and external shaft resistance. The bearing capacity of the open-ended plugged pile consists of the point resistance of the plugged pile (paragraph 4.3) and external shaft resistance.

The bearing capacity of the pile group is the smallest of following:
- sum of bearing capacities of individual piles; typical for groups of point bearing piles,
- bearing capacity, which is obtained assuming the pile group a uniform pier foundation, or
- action, which causes the maximum permitted settlement of the pile group.

4.1.1 Static bearing capacity formulas

The geotechnical ultimate load of the pile P_u is derived from the formula:

$$P_u = \int_0^Z \pi \, d \, f_s \, dZ + A_{pk} \, q_p - W \qquad (2)$$

Z=the pile length inside the ground
d=diameter of the pile
f_s=shaft friction
A_{pk}=cross section area of the pile point
q_p=point resistance
W=weight of the pile

4.1.1.1 Point resistance in homogeneous cohesionless soil layer

Point resistance q_p is derived from the formula:

$$q_p = \sigma'_{vp} N_q \tag{3}$$

σ'_{vp} = *effective vertical stress on the level of the pile point. The effective vertical stress is calculated considering the weight of the soil layers locating maximum 10 d above the pile point.*

N_q = *bearing capacity factor (figure 6.). When determining the bearing capacity factor, the internal friction angle is taken as a medium value between 5 d above the pile point and 3 d below the pile point. Use of the friction angle ϕ > 40° requires performing of the triaxial tests in the laboratory or in situ tests.*

The maximum point resistance is $q_p \leq 20$ MPa, if the determination of the strength parameters is not based on the laboratory or in situ tests. The point resistance of the Franki pipe pile is determined according to the valid instructions concerning Franki piles.

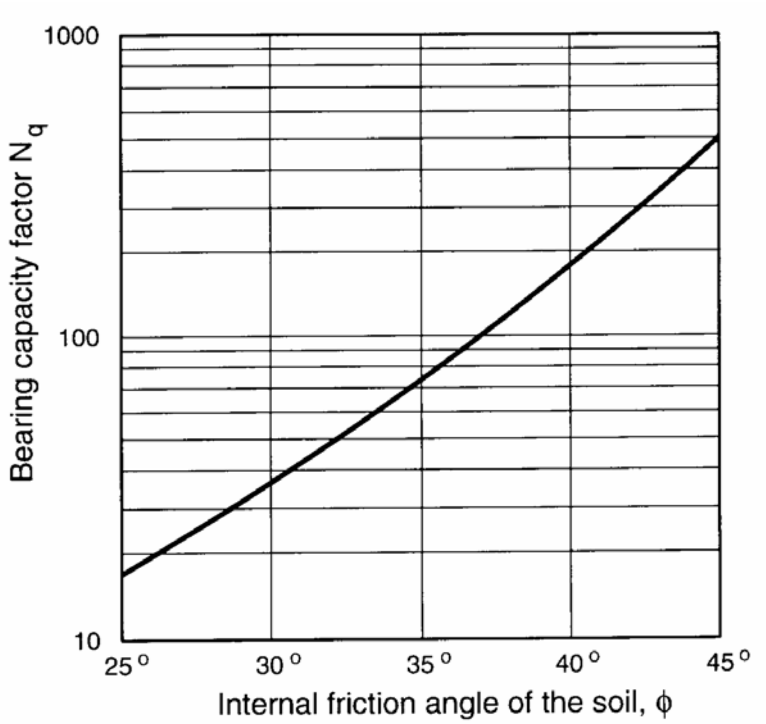

Figure 6: The bearing capacity factor N_q as a function of the internal friction angle of the soil ϕ /11/.

4.1.1.2 Point resistance in rock

The point resistance q_p for the pile resting on rock (cf. 3.2.1.1) is determined from *formula 4*.

After several thousand blows (5000...10000 blows) the point resistance is:

$$q_p \approx 2\, q_{cyl} \qquad (4)$$

q_{cyl} = *uniaxial compression strength of the rock, [MN/m²]*

The strength of the rock mass can be assessed from the ultimate uniaxial compression strength. The uniaxial compression strength of the rock is usually 100...300 MN/m^2. The uniaxial compression strength of rocks is handled in more detail in reference /19/.

4.1.1.3 Shaft resistance in cohesionless soil layer

The shaft resistance f_s of driven steel pipe piles is determined from formula:

$$f_s = K_s\, \sigma'_v \tan \phi_a \qquad (5)$$

K_s = *earth pressure coefficient (figure 7)*
ϕ_a = *friction angle between the steel pipe pile and the ground;*
tan ϕ_a = *0,7 tan* ϕ
σ'_v = *effective vertical stress in the ground*

The earth pressure coefficient K_s is determined as a function of internal friction angle of the soil, ϕ. In reality the shaft friction is affected by the compressibility of the soil, the original horizontal stress in the ground, and the size and shape of the pile. The exact determination of the shaft resistance requires test loadings.

The friction angle ϕ_a between the pile and the soil is smaller than the internal friction angle of the soil, ϕ. Reduction is made for the term tan ϕ. The term tan ϕ_a between cohesionless soil and steel is obtained by reducing the term tan ϕ with factor 0,7.

The maximum shaft resistance of the steel pipe piles can be $f_s \leq 0{,}15$ MPa.

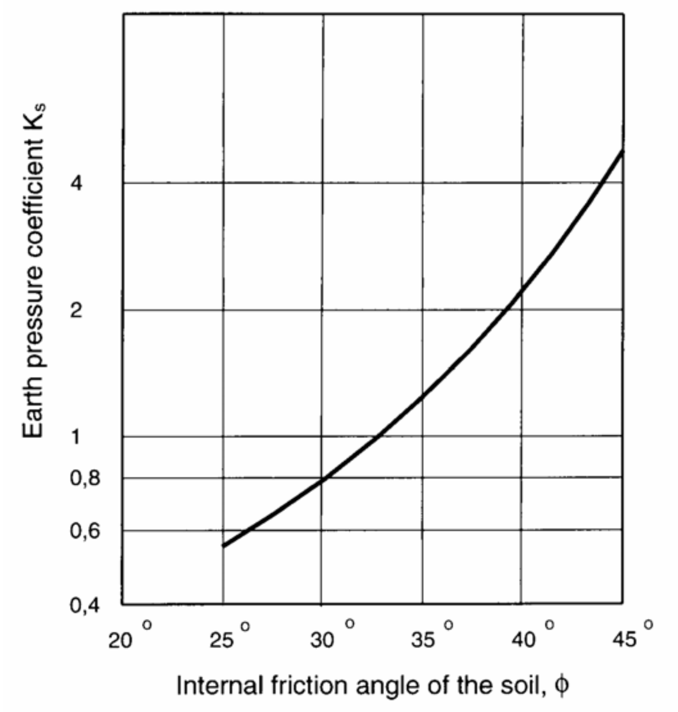

Figure 7: The earth pressure coefficient K_s as a function of the internal friction angle of the soil ϕ /5/.

If the pile point is strengthened, the reducing effect of the strengthening should be taken into consideration (paragraph 6.4).

4.1.1.4 Shaft resistance in cohesive soil layer

In cohesive soil layers the shaft resistance cannot be utilized for sustaining the permanent loadings of the steel pipe piles.

In saturated clay the shaft resistance is equal to adhesion, when the excessive pore pressure caused by the pile driving is mainly dissipated. The shaft resistance f_s is derived from formula:

$$f_s = s_a = \alpha \, s_u \qquad (6)$$

s_a = *adhesion*
α = *adhesion coefficient of the steel piles (figure 8)*
s_u = *undrained shear strength*

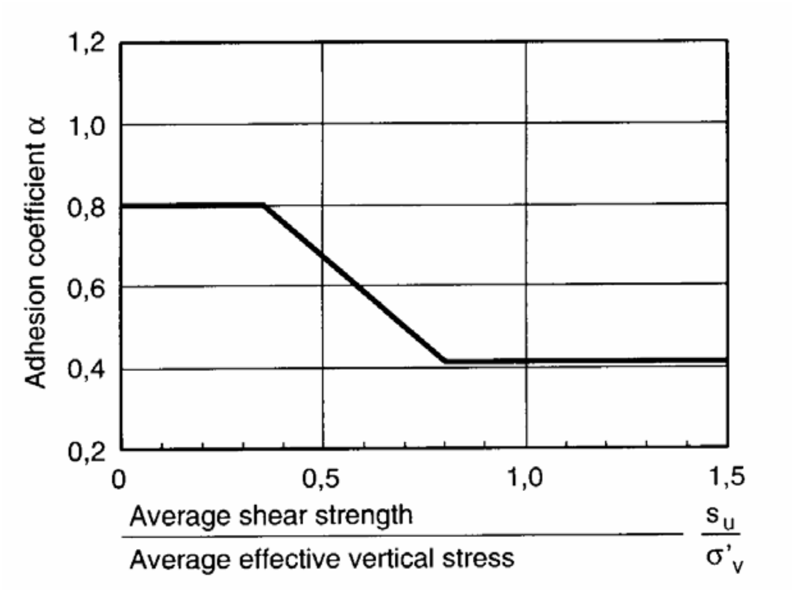

Figure 8: Adhesion coefficient, α, for steel piles in clay /10/.

4.1.2 Methods based on sounding resistance

The bearing capacity of the pile can be evaluated directly from the sounding resistance in dynamic probing, weight sounding or cone penetration test. The basis for coarse comparison between the sounding resistances obtained with different sounding methods are presented in Finnish foundation instructions for bridge design (TIEL 2172068) /22/.

4.1.2.1 Geotechnical ultimate limit load on the basis of sounding resistance in dynamic probing

The geotechnical ultimate load P_u is derived from *formula 2*.

The shaft resistance f_s and point resistance q_p are determined for steel piles from *figure 9*. The effect of the material on the shaft friction angle is taken into consideration in the shaft resistance curve presented in figure 9. In determination of the point resistance of the pile on the basis of figure 9 the average sounding resistance observed in dynamic probing is taken from 5 d above the pile point to the depth of 3 d below the pile point.

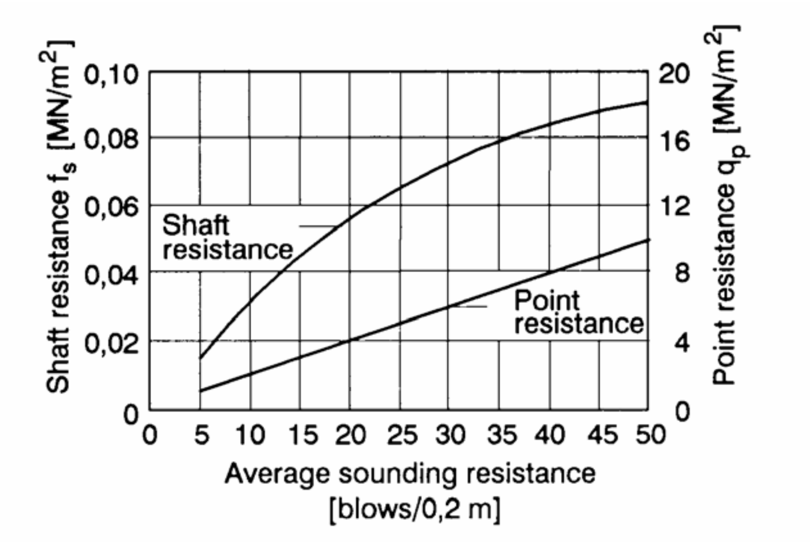

Figure 9: Shaft resistance f_s and point resistance q_p for steel piles from the average sounding resistance in dynamic probing.

The point resistance can be extrapolated linearly until the value $q_p \leq 20$ MPa. The shaft resistance does not exceed the value $f_s = 15$ MPa while the sounding resistance is increasing.

4.1.3 Dynamic pile driving formulas

The dynamic pile driving formulas are based on energy analysis, in which the static bearing capacity is evaluated using a simplified driving resistance. The bearing capacity of the pile is determined from the measured settlement using Gates' formula or from the measured settlement and temporary compression using Hiley's formula.

Gates´ formula:

$$P_u = 96\,(2{,}4 - \log s)\sqrt{e_f W_h H} \qquad (7)$$

e_f = *driving efficient coefficient*
W_h = *weight of the hammer, [kN]*
H = *drop height of the hammer, [m]*
s = *measured pile settlement/blow, [m]*

Hiley's formula:

$$P_u = \frac{e_f E_j}{s + \frac{1}{2}c} \frac{W_h + n^2 W_p}{W_h + W_p} \qquad (8)$$

E_j = *driving energy, [kNm]*
e_f = *driving efficient coefficient*
n = *factor, which is 0,5 for a hard wooden driving cap, 0,8 for a hard plastic driving cap and 1,0 for a steel dolly without a driving cap /12/*
s = *permeable settlement of the pile, [mm]*
c = *temporary compression, [mm]*
W_p = *weight of the pile, [kN]*

The driving effectiveness of the piling device should normally be checked on the site in connection with the dynamic test loadings.

4.1.4 Dynamic test loadings

The dynamic test loading is a direct method for the determination of the bearing capacity, being based on the stress wave measurements (PDA-measurements) performed on the building site. The static bearing capacity of the pile is calculated from the measurement results considering the effects of the rate of the dynamic loading.

Dynamic measurements can be analyzed in several different ways, e.g., using the CASE-method. The results obtained can be further processed with a computer. Adjusting the damping and temprorary compression coefficients used as soil parameters in such a way, that the calculated stress wave joins the measured wave, provides the loading distribution in the pile, and the bearing capacity can be divided to the shaft and point resistance. The most commonly used methods are CAPWAP and SIGNAL MATCHING analyses.

The determination of the bearing capacity can be performed using part of the PDA-measurement results with the CAPWAP or a corresponding analysis, if the primary determination of the ultimate load is performed using the CASE method.

4.2 Internal shaft resistance of open-ended pile

If no plugging is occurring in the pile, it can be assumed, that the internal shaft resistance is half of the external shaft resistance. The bearing capacity composing from the internal shaft resistance and the point resistance of the steel cross section area must not exceed the bearing capacity composing from the point resistance of a plugged pile of corresponding size.

4.3 Bearing capacity of plugged pile

A soil plug develops to the open-ended pile in cohesionless soil layer, if the plugging soil layer includes only small amounts of fines and is sufficiently well grained and at least medium dense and if the pile is installed sufficiently deep in the plugging soil layer. In addition to this, the pile driving should be performed using a slow driving hammer. The pile plugging becomes more effective, when the acceleration caused by the hammer blow to the pile is decreasing. If the pile is installed using a vibratory hammer, no plugging occurs. If the pile is designed to sustain the loading as plugged, a test pile drive should be performed to verify the plugging in a reliable way.

The geotechnical ultimate load P_u of the open-ended pile is derived from formula:

$$P_u = \int_0^Z \pi \, d \, f_s \, dZ + \eta \, A_{pk} \, q_p - W \tag{9}$$

Z	=	*the pile length inside the ground*
d	=	*external diameter of the pile*
f_s	=	*shaft friction on the external shaft*
η	=	*plugging coefficient*
A_{pk}	=	*cross section area of the close-ended pile point of the corresponding size*
q_p	=	*point resistance*
W	=	*weight of the pile*

The plugging coefficient η is determined in moraine with formula /1,2/:

$$\eta = 0,8 \qquad if \ \frac{z}{d} = 10 \tag{10}$$

And in sand or gravel with the formula:

$$\eta = 0,8 \qquad if \ \frac{z}{d} = 15 \tag{11}$$

z	=	*installation depth in the plugging soil layer*
d	=	*diameter of the pile*

When the relation z/d decreases, the plugging coefficient is reduced linearly.

Plugging can be verified, if the settlement of the soil surface inside the pipe in the plugging soil layer is at least half of the settlement of the pile. Plugging cannot be found from PDA-measurements performed during pile driving. Plugging can be found at the earliest two weeks after the pile driving by using dynamic probing performed through the pipe. If the plugging degree is evaluated using PDA-measurements, the measurement can not be performed from redriving until two weeks after the pile is driven.

4.4 Negative shaft friction

Negative shaft friction is determined according to the Finnish pile driving instructions LPO-87 /16/.

4.5 Tension capacity of piles

When the tension capacity of the piles for long term tension loading is designed, only the shaft resistance in cohesionless soil layers can be taken into account in addition to the effective weight of the pile. Then the shaft resistance of the compressed pile is divided by the factor k_v = 2. In transient loading the shaft resistance of the compressed pile is divided by the factor k_v = 1,5, and the shaft resistance can also be taken into account in cohesive soil layers.

4.6 Safety level requirements

The geotechnical ultimate load for steel pipe piles can be determined using the methods presented above (cf. 4.1...4.3). Safety level requirements depend on the reliability of the method used for the determination of the geotechnical ultimate load in relevant conditions.

The geotechnical bearing capacity of the pile is determined from the ultimate load of the pile by dividing it by the total safety factor. Recommended values for total safety factors are presented in *table 3*. If the factors presented in table 3 are used in the calculation of the bearing capacity, the effects of loadings are calculated using characteristic loads.

Table 3. Total safety factors for determination of the geotechnical bearing capacity.

Method for determination of the geotechnical ultimate load	Safety factor F
Static bearing capacity formulas	2,5...3,0
Methods based on sounding resistance	2,5...3,0
Dynamic pile driving formulas	2,5...3,0
Dynamic test loading	2,0
Static test loading	1,8

Usually dynamic test loadings should be performed for large diameter steel piles. If no dynamic test loadings are performed, the bearing capacity of the pile is determined both on the basis of the penetration depth and final set after the pile driving. Then if the bearing capacity is determined from two bases and using both static bearing capacity formulas or methods based on sounding resistance and pile driving formulas, the slightly reduced safety level $F \geq 2,2$ compared to table 3 can be used. For a concrete pile with a steel casing, which can be compared to the cast-in-piles and in which the determination of the bearing capacity is based on the penetration depth of the pile, the safety level requirement is $F \geq 2,5$. If a large amount of dynamic or static test loadings is performed on the site, the safety level requirement can be lower than the safety factors presented in table 3.

4.7 Pile settlement

Assessing the settlement of point bearing piles in the soil layer is inaccurate without test loadings. Static test loadings should be performed in such a way, that the settlement of the pile point can be observed. The settlement analysis for a foundation on point bearing piles is often performed on the basis of the settlement evaluations for individual piles.

The settlement of the close-ended point bearing pile, S_0, consists of the elastic compression of the pile, S_e, and deformation of the layers below the point, S_s. The settlement of the upper pile head, S_0, is:

$$S_0 = S_e + S_s = \frac{P\,L}{A_p\,E_p} + \frac{P\,d}{A_p\,E_s} \qquad (12)$$

P = *load of the pile*
L = *length of the pile*
d = *diameter of the pile*
A_p = *cross section area of the pile*
E_p = *Young's modulus of the pile*
E_s = *Young's modulus of the soil layer below the pile point*

Values presented in *table 4* are recommended for the modulus E_s below the point of the point bearing pile.

Table 4. Young's moduli E_s for the soil and rock below the point of an individual, close-ended pile after pile driving.

Soil type	E_s [MPa]
Sand, gravel	1000...2000
Moraine	2000...10000
Rock	50000...100000

Installation of the unplugging pile and cast-in-place pile with steel casing does not compact the soil below the pile point. The compression modulus of the soil in natural state is used for the soil below the point in the calculation of the pile settlement.

The settlement of the group of point bearing piles corresponds to the settlement of individual piles, if the piles are resting on rock or the moraine layer between the pile point and rock is thin and dense. The settlement of the individual pile can be assessed using the same principle taking into account the effect of the shaft friction.

The settlement of the group of friction piles can be calculated using the model presented in *figure 10*. The compression moduli of the soil layers are determined in the same way as the compression moduli of the soil layers in their natural state.

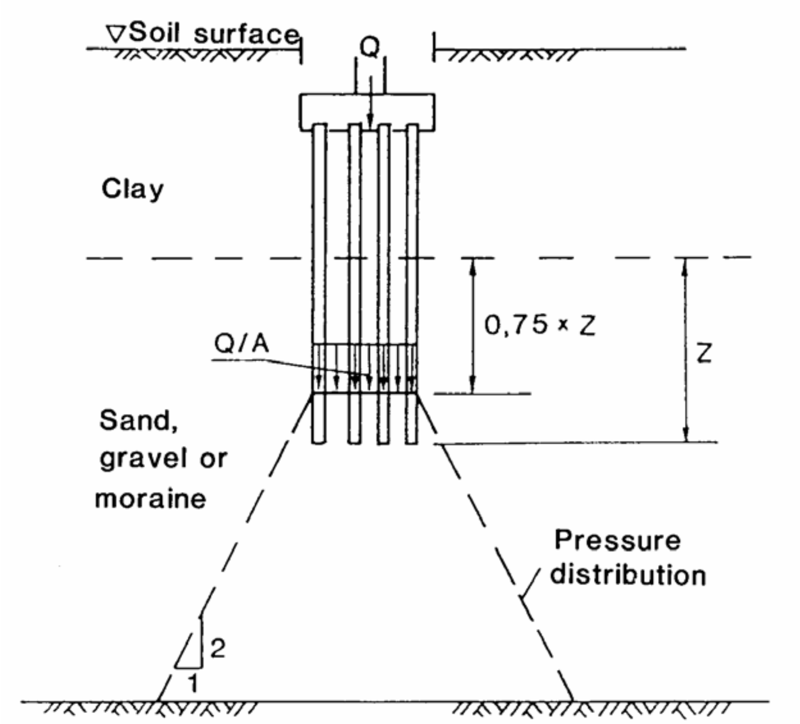

Figure 10: Settlement calculation for friction pile group /14/.

4.8 Loadings

In pile design the lateral loadings and moments should be considered in addition to the loadings parallel to the pile. The design should be performed for the most dangerous loading combination. Loadings should be determined according to reference 0 (TIEL 2172072) and other valid instructions. In pile actions, the additional actions caused by location and inclination deviations of the pile and by the location deviations and displacements of the supported structure should be taken into account.

The permitted location deviations of the piles should be determined smaller than values presented in paragraph 7.3, if the location deviations presented in paragraph 7.3 may cause exceeding of the permitted bearing capacity with 15%. If the actual deviations exceed the values permitted, check calculations are performed according to paragraph 10.5. Then a 15% excess in bearing capacity can be allowed for an individual pile. If necessary, a pile load smaller than the permitted bearing capacity can be used in design as a precaution against the location deviations.

4.9 Lateral geotechnical design

4.9.1 Lateral capacity of pile

The lateral capacity of the pile normally means the maximum lateral loading that the pile can tolerate, which corresponds to the ultimate load of the soil or in some cases the yielding moment of the pile. Thus the lateral capacity is composed of the resistance of the surrounding soil, the attachment degree of the upper head of the pile and the bending stiffness of the pile. Especially, if the loading is repeated or dynamic, the behaviour of the surrounding soil may become governing factor.

In bridge design, however, the lateral capacity is normally determined by the permitted lateral displacement, because the deformations may grow too large from the standpoint of the bridge structures before the lateral capacity is exceeded. The permitted lateral displacements are determined based on the displacements allowed by the other structures.

The lateral capacity of the pile group is usually smaller than the lateral capacity of the individual pile multiplied by the number of piles, if the mutual central distance in the direction of the lateral load is less than 6...8 times the diameter of the pile. The lateral resistance and capacity decrease further, when the number of successive pile rows increases in the effective direction of the load and when the central distance of the piles decreases.

The design of the pile group returns in principle to the design of the individual piles, when the loadings transmitting through the pile footing are determined using one of the generally accepted methods.

4.9.2 Lateral loading of pile

The lateral loading of the pile is caused by the imposed force due a displacement, supporting force due lateral loads or earth pressure against the pile.

Temperature changes in the supported structure, rotation, shrinkage and creep cause imposed forces.

The earth pressure results from the difference between soil levels or from the soil mass moving towards the pile due to the low stability of the excavation or inclined slope. When the safety against the sliding failure of the soil mass in piling area is $F < 1,8$, plastic movements develop in the ground and the earth pressure against the piles should be considered. At the higher safety level, loading differences cause only elastic displacements, which can be so large in soft layers, that it is necessary to take them into account as a bending loading the pile. Lateral pile loading follows also from ground settlement around inclined piles and from frost pressure, the development of which should normally be prevented.

The magnitude of the lateral load depends on the shear strength of the soil, the shape of the pile and the loading rate. The lateral load is at maximum, when the pile slips through the soil mass. The limit value of the lateral load is obtained according to paragraph 4.9.4. When the pile is designed for a lateral load caused by the imposed force or the earth pressure, the connection between the lateral pressure and displacement according to the modulus or subgrade reaction method can be used (cf. paragraph 4.9.5).

4.9.3 Failure mechanisms of piles

Failure mechanisms of the laterally loaded pile depend on the relative stiffness of the soil, the pile and the pile attachment to the structure.

The relative stiffness can be assessed with parameter R in cohesive soils and parameter T in cohesionless soils.

$$R = \sqrt[4]{\frac{E_p I_p}{E_s}} \quad (13)$$

$$T = \sqrt[5]{\frac{E_p I_p}{n_h}} \quad (14)$$

E_pI_p = *stiffness of the pile*
E_s = *lateral modulus of the cohesive soil*
n_h = *coefficient of the lateral subgrade reaction for a cohesionless soil*

When the relation between the installation depth of the pile and parameters mentioned above L/R or L/T, i.e. relation of stiffness, is at most two, the pile is treated as a body, which rotates rigidly in the ground, and the deformations of the pile can be ignored. Ground failure occurs then before the pile fails. The location of the rotation center is calculated assuming that the bending moments are in equilibrium. In homogeneous soil it can be assumed, that the rotation center is situated at a depth approximately equal to 70% of the installation depth. Then the lateral capacity can be evaluated also by manual calculations, determining first the ultimate value and distribution of the lateral capacity (cf. 4.9.4).

When the relation between the installation depth and parameters L/R and L/T is at least 4, deformations of the pile should also be considered in the calculations. The failure of the pile occurs at this level of stiffness before the ground fails. When the relation of stiffness is between 2...4, the intermediate values can be interpolated with a sufficient accuracy. The pile length corresponding to the value of the stiffness relation factor equal to 4 can be considered as an ultimate value for the functional length, after which, increasing the installation length does not affect the performance of the pile.

The influence of the stiffness relation on the failure mechanism of the pile is shown in *figure 11*. Figures 11 a and 11 c represent the values of the stiffness relation factor of less than or equal to 2. Figure 11 d represents the values between 2...4, and figures 11 b and 11 e represent values more than or equal to 4.

4.9.4 Ultimate values of lateral resistance and lateral pressure

Lateral resistance develops, when the pile moves towards the ground due to an external load. Lateral resistance means the resistance caused by the soil mass per unit area. Lateral resistance is exploited when calculating the lateral capacity of the pile.

The lateral pressure develops, when the soil moves towards the pile. This concept is used when calculating lateral loading.

The limit values of the lateral resistance and lateral pressure depend on the strength properties of the soil and in cohesionless soil also on the effective unit weight.

The limit values of the lateral resistance and lateral pressure are determined according to the failure state of the soil based on the earth pressure theory. Safety is considered in the calculation of the lateral resistance and lateral pressure according to paragraph 4.9.8 in such a way, that the most adverse effect to the lateral capacity or to the lateral loadings is considered.

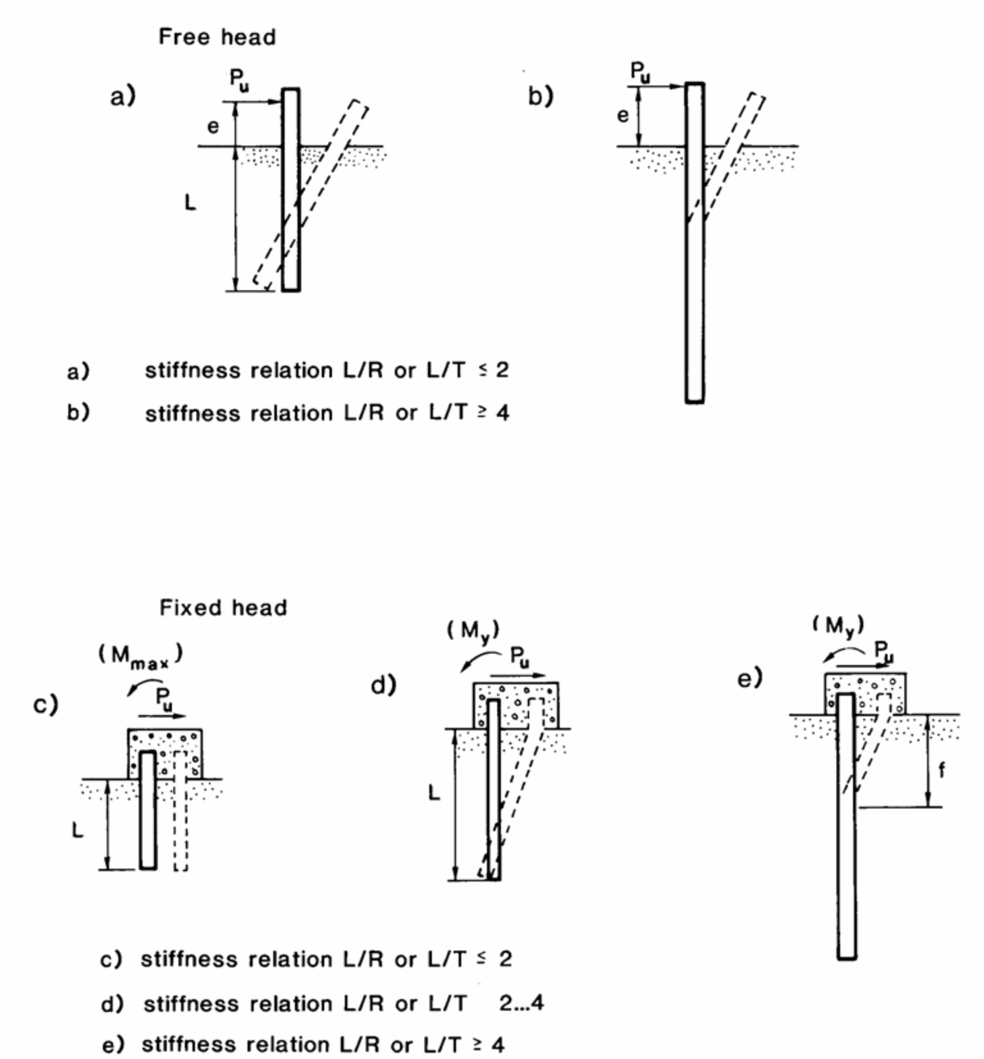

Figure 11: The influence of the stiffness relation on the failure mechanism of the pile /8/.

The ultimate values of the lateral resistance for cohesionless and cohesive soils according to the different references are presented in *figure 12 /3/*.

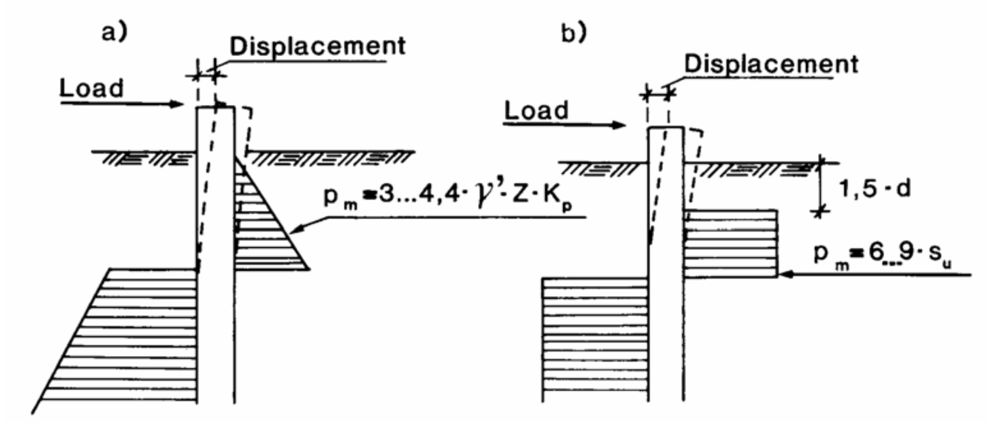

Figure 12: The ultimate values of the lateral resistance and pressure p_m
a) in cohesionless soil
b) in cohesive soil.

In cohesionless soil, the values of the lateral resistance and pressure should increase linearly with the depth. In cohesive soils the values are assumed constant regardless of the depth. The surface layer of the cohesive soil is taken into account to a depth of 1,5 d in the calculation of the lateral loading, but not in the calculation of the lateral capacity.

4.9.5 Calculation of displacements

Displacements are calculated using the subgrade reaction method. The interdependency between the lateral resistance or pressure and displacements is normally represented with moduli of subgrade reaction. Moduli of subgrade reaction are not material parameters of the soil, while they are also dependent on the dimensions of the structure. Displacements are calculated in the serviceability state using characteristic values of the material parameters and actions. The safety is considered in values of the permitted displacements.

In cohesionless soil the lateral subgrade reaction k_s is expected to increase linearly to the depth z = 10 d and thereafter to remain constant. The subgrade reaction of the cohesionless soil under static loading is determined with formula:

$$k_s = n_h \frac{z}{d} \tag{15}$$

where the coefficient of the subgrade reaction n_h is obtained from *figure 13* as a function of the internal friction angle.

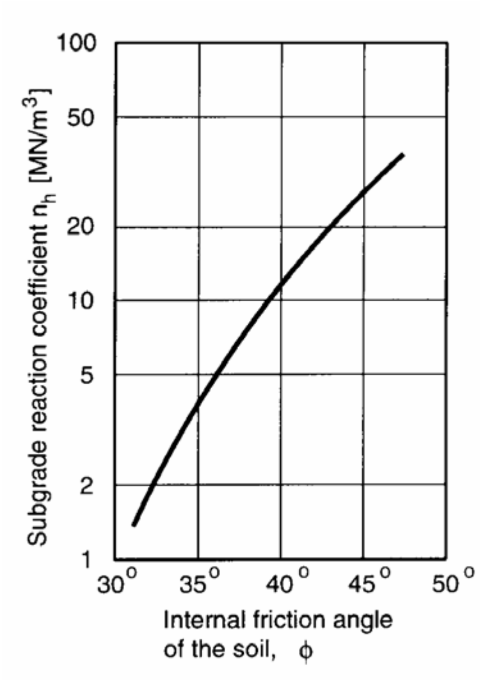

Figure 13: Assessment of coefficient of the subgrade reaction in cohesionless soil from the friction angle. Below the ground water level n_h is 60% of values presented in figure /16/.

The friction angle can be assessed according to the Finnish foundation instructions for bridge design (TIEL 2172068) /22/. The subgrade reaction in cyclic loading can be determined according to the table 5 in paragraph 4.9.7. The subgrade reaction in cyclic or dynamic loading can be determined from the static subgrade reaction based on the relation between the dynamic (depending on the deformation level) and static Young's moduli or shear moduli.

Figure 14 shows a method to represent the approximate lateral pressure-displacement connection of the subgrade reaction in cohesionless soil.

Yet because the subgrade reaction is directly dependent on the compressibility, it can be viewed from the compressibility modulus M of the soil or from the Young's modulus E_d in drained conditions, when the subgrade reaction is obtained from formula:

$$n_h = \alpha\,\beta\,\frac{M}{z} = \alpha\,\frac{E_d}{z} \qquad (16)$$

α = *0,74 (according to Terzaghi) /8/*

α = *1,0 (according to Poulos) /8/*

β = *0,83...0,95 for sand, while Poisson's ratio varies correspondingly 0,25...0,15.*

In temporary loading the lateral subgrade reaction of the cohesive soil varies between:

$$k_s = 50...150\,\frac{s_u}{d} \tag{17}$$

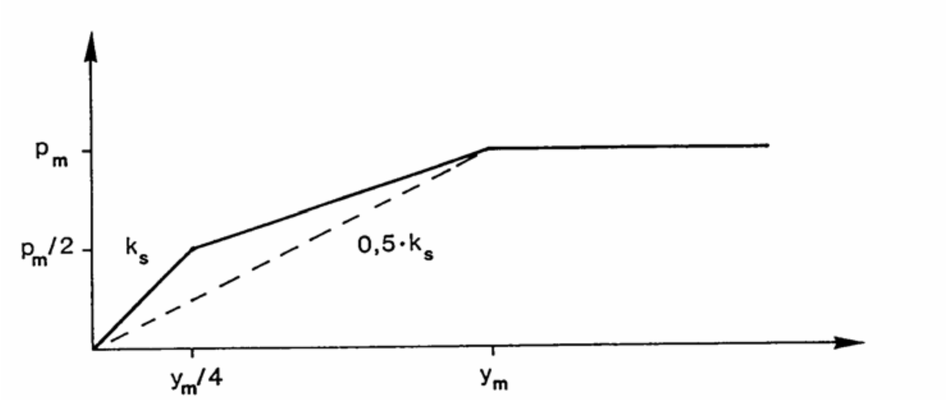

Figure 14: Determination of the subgrade reaction of cohesionless soil. p_m is the ultimate value of the lateral resistance and y_m is corresponding displacement /8/.

$$k_s = 20...50\,\frac{s_u}{d} \tag{18}$$

In long-term loading the subgrade reaction of the cohesive soil varies between:
The approximate lateral pressure-displacement connection of the subgrade reaction in cohesive soil for temporary loading is presented in *figure 15 a* and for long-term loading in *figure 15 b*. The note p_m in *figure 15* represents the lateral pressure corresponding to the ultimate load and the corresponding lateral displacement of the pile is presented by y_m. The subgrade reaction of the cohesive soil is assumed constant regardless of the depth.

In the case of long-term loading the subgrade reaction of the cohesive soil can be determined also from the compressibility modulus (M), when the subgrade reaction k_s is:

$$k_s = \beta\,\frac{M}{d} \tag{19}$$

β = 0,46...0,74 for clay, while Poisson's ratio varies correspondingly between 0,4...0,3
β = 0,62...0,83 for silt, while Poisson's ratio varies correspondingly between 0,35...0,25.

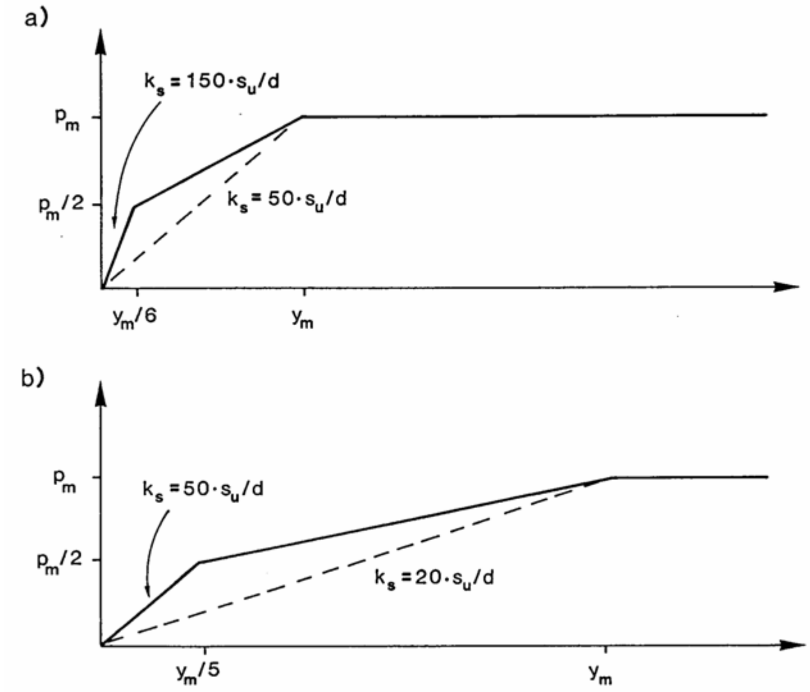

Figure 15: Determination of the subgrade reaction of cohesive soil
a) in temporary loading
b) in long-term loading /8/.

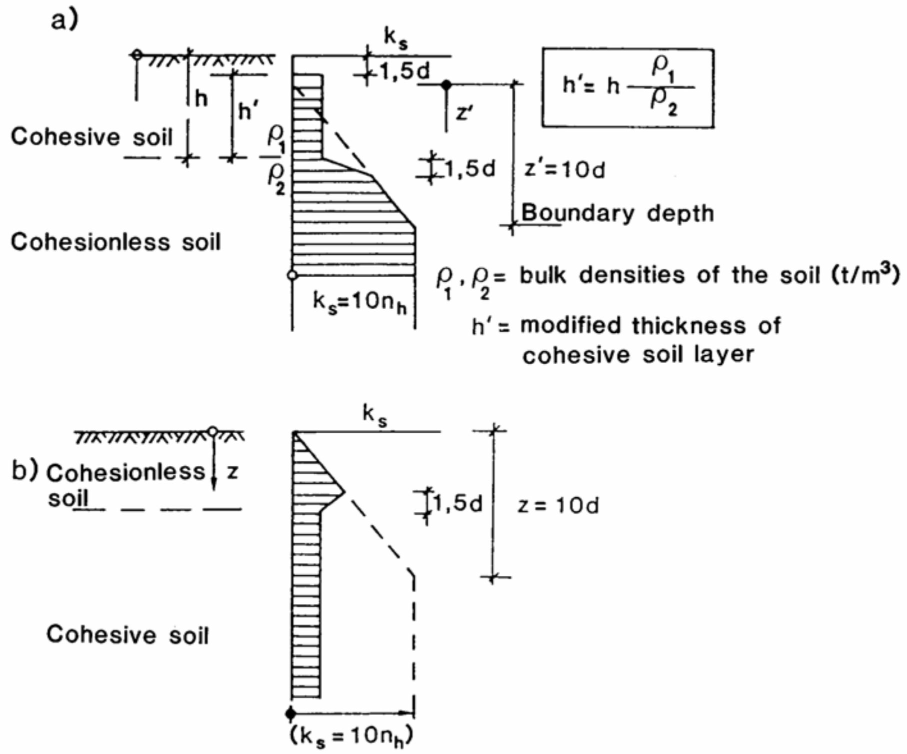

Figure 16: Subgrade reaction in a layer boundary
a) when the cohesionless soil is below the cohesive soil
b) when the cohesionless soil is above the cohesive soil /16/.

In the case of temporary loading the subgrade reaction of the cohesive soil can be determined from Young's modulus E_u in an undrained situation, when the determination can be realized using the undrained triaxial test. *Figure 16* shows the determination of the subgrade reaction in different layer boundaries.

4.9.6 Design for static lateral loading

Design for the static lateral loadings means analyzing of the pile in loading case, in which the load is constant with the time and in which the inertia forces do not affect the actions of the structure.

At present there are several methods, which have a reasonably good accuracy, for manual calculations for the analysis of the laterally loaded pile. Increasing demand for accuracy provide ample cause to perform the analyses with a computer, because the nonlinear behaviour of the soil leads to several iteration circles in calculations.
The following calculation model is suitable for personal computers. In this model the static properties of the ground are represented using nonlinear, horizontal springs and the pile is modelled using the element method (FEM). The reaction forces caused by the pile movements are centralized as springs to the node points of the element model. The values of the springs can be determined e.g., using the subgrade reaction method (cf. paragraph 4.9.5). From the standpoint of the calculation accuracy the exact determination of the values for the springs nearest to the soil surface is primarily important. When the displacement exceeds the corresponding ultimate load of the soil i.e. the ultimate value of the lateral resistance the elastic-plastic lateral pressure-displacement function is iterated. The element-spring model, which is dependent on the values of the stiffness relation L/R and L/T, used in the method is presented in *figure 17*. The boundary conditions of the pile head and pile point are also modelled using springs and they should correspond as near as possible to the performance of the upper structure and conditions at the point.

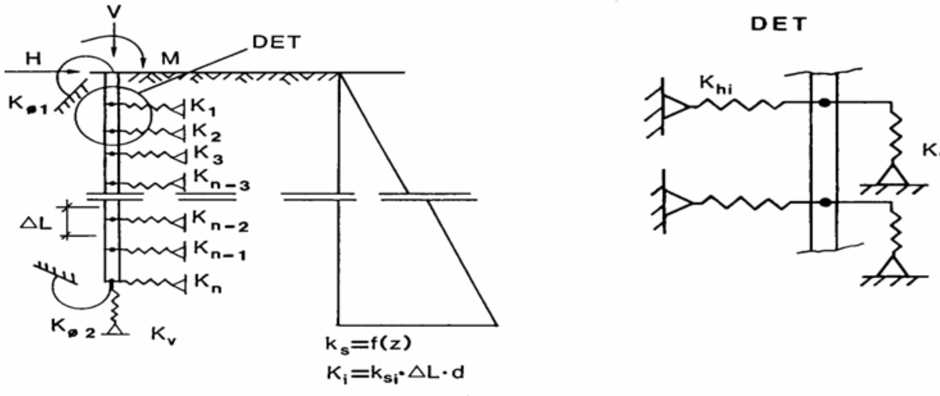

Figure 17: The element-spring model depicting the interaction between the pile and the ground.

4.9.7 Design for cyclic lateral loading

Design for the cyclic lateral loading means analyzing of the pile in a loading case, in which the loading changes with time and the inertia forces do not normally affect on the actions of the structure. Loading can be unidirectional or varidirectional. Cyclic, i.e. repeated, loading weakens the strength and deformation properties of the soil and thus increases the lateral displacement compared to the static loading situation. The looser the soil is, the greater the increase. The upper boundary value for the frequency of the cyclic loading is normally assumed f = 1 Hz. If the density of sand is below the critical value, which corresponds sounding resistance 15 blows/0,2 m in dynamic probing, the risk of soil liquefaction should be taken into account.
Wave loadings may be included in cyclic loadings, but they are of minor importance, when they are directly loading the pile or pier of the bridge. However, the waves affecting through caissons, vessels or pack ice should be considered.

When the laterally loaded pile is analyzed for cyclic loads, the static subgrade reaction method, in which the modified subgrade reactions given in *table 5* are used, can be applied. The values presented in table ditto are valid for cohesionless soil and they are classified according to the density of the soil.

Table 5. The lateral subgrade reaction of the cohesionless soil k_{ss} for cyclic loading /8/. k_s = static subgrade reaction

Subgrade reaction for cyclic loading	Relative density D_r		
	< 0,35 Loose	0,35...0,65 Medium dense	> 0,65 Dense
k_{ss}	0,25 k_s	0,33 k_s	0,5 k_s

The liquefaction risk of loose cohesionless soil is checked and taken into account, if necessary.

4.9.8 Safety considerations in design

In lateral design of the pile the limit state method is recommended. In the limit state method partial safety factors are located in material parameters and to the lateral load, and in some cases to the settlement, also. Total safety factors can also be used.

Designing the lateral capacity of the pile, the material parameters of the soil are divided by the partial safety factor. If the safety of the lateral capacity is assessed on the basis of the displacement, the safety can be included in the values of permitted displacement. The actions are then calculated using characteristic values.

When the pile is designed for the lateral load caused by the imposed force or the earth pressure, the material parameters are multiplied by the partial safety factors, this ensures the safety of the lateral load.

When the total safety factor method is used, the minimum total safety factor should be $F \geq 2{,}2$ based on calculations and $F \geq 1{,}8$ based on static test loadings. When partial safety factors are used, it is sometimes necessary to also check the reasonable safety level utilizing the total safety factors.

4.9.9 Increasing of lateral capacity

The lateral capacity of the pile can be increased by reinforcing the soil or other structural measures, which primarily enlarge the cross section area of the upper end of the pile.

The strength of the soil can be improved by compaction or injection.

Structural measures include e.g. wings welded to the pile or a concrete collar. These are presented in *figures 18 a* and *18 b*. Considerable increase in lateral capacity can be achieved utilizing the steel frame solution presented in *figure 18 c*. Another measure is additional filling on the soil surface (*figure 18 d*).

Inverse measures can include lightening excavations to reduce lateral loading and light fillings.

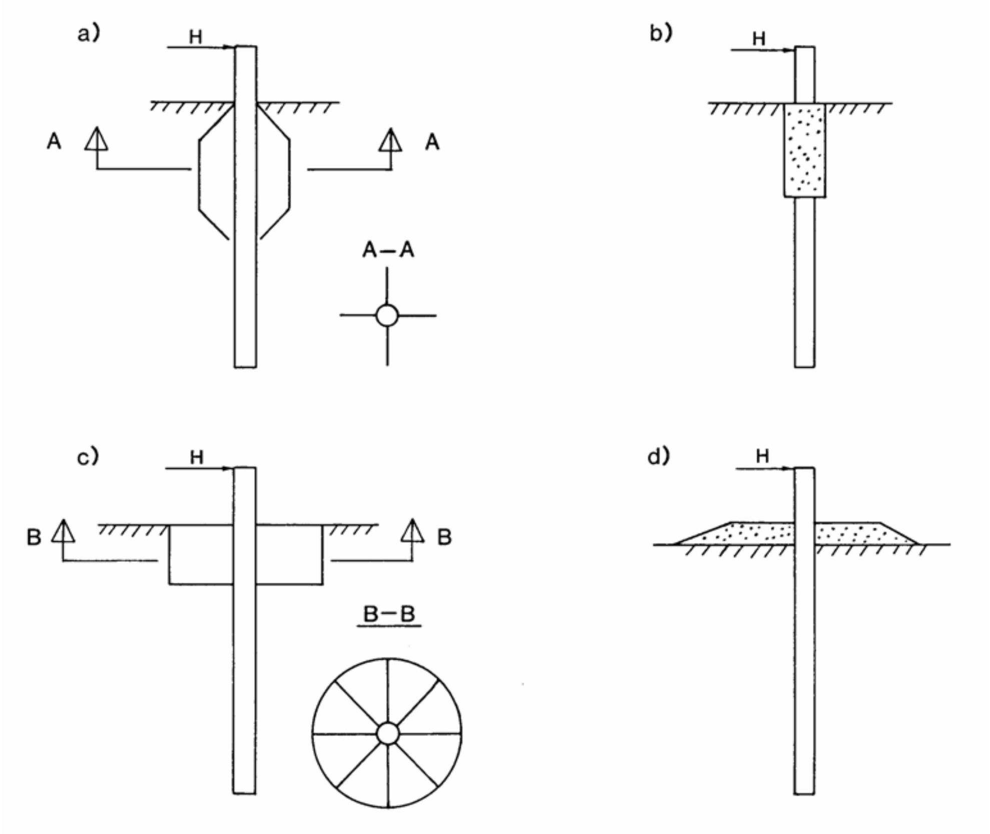

Figure 18: Possible solutions for increasing lateral capacity /6/.

5 STRUCTURAL BEARING CAPACITY OF PILE

The structural bearing capacity of the pile is determined by the strength of the pile structure. The structural bearing capacity is checked against the actions coming from the supported structure. Also bending moments from the horizontal loads, eccentricities or fixing moments may load the pile. In addition to the requirements of the supported structure the bearing capacity of the pile should be considered for buckling, additional loads, such as negative shaft friction and bending of the inclined piles due to the ground settlements or bending caused by one-sided soil pressure or lateral resistance. The recommended minimum wall thickness of the steel pipe pile driven from the upper head is 10 mm. For pipes with a diameter less than 600 mm the wall thickness may be 8 mm. The corrosion of the pile should be considered when determining the long-term structural bearing capacity of the pile.

5.1 Design of structural bearing capacity

1) Permitted material stresses of the pile are determined on the basis of the pile material and soil conditions. In bouldery soil conditions it may be appropriate to reduce the material stresses permitted in normal situations.
2) The structural capacity of the pile is checked in governing loading situations considering the corrosion reduction.
3) The driving power required for the permitted geotechnical bearing capacity is determined and the driving stresses are checked considering the safety factor. The corrosion of the pile can be ignored.

5.2 Structural design

5.2.1 Unconcreted steel pipe pile

In a completed structure the steel pipe pile is usually filled with soil. The upper part of the open-ended piles is sometimes empty and in close-ended piles the hole pipe is empty. The structural capacity of the pile is formed by the bearing capacity of the steel pipe.

5.2.1.1 Driving stresses

The maximum permitted driving stress of the pile is σ_{dall} = 0,9 σ_{sa}, when σ_{sa} is the lower yield strength of the pile material. The corrosion of the pile is ignored.

Pile driving induces both compression and tension stresses into the pile. The force caused by the hammer blow can be derived by multiplying the stress by the area of the pile. The preliminary estimate of the driving stress can be calculated using the following formula:

$$\sigma_{\max} = f_w\, f_0\, v_0 \frac{E}{c} = f_w\, f_0 \sqrt{\gamma\, H\, E} \qquad (20)$$

f_w	=	*reflection factor for the stress wave depending on the site soil conditions*
f_0	=	*coefficient depending on the pile driving rig* $f_0 = e_f \sqrt{2}$*, where* e_f *is the effectiveness factor of the pile driving hammer*
v_0	=	*velocity of the hammer, [m/s]*
E	=	*Young´s modulus for steel = 2,1 x* 10^8 *kN/m*^{2}
c	=	*velocity of the stress wave in steel* $\approx$ *5100 m/s*
γ	=	*unit weight of the steel = 77 kN/m*^{3}
H	=	*drop height of the hammer, [m]*

Unless more accurate information is available regarding the pile driving hammer, the range of factor f_0 can be 0,7...0,85. When a steel pipe pile is driven against the rock, the reflection factor of the stress wave f_w ranges between 1,5...1,7. When the pile point is in dense soil, f_w = 1,3...1,5.

The effectiveness of the pile driving hammer and the driving stresses are determined with stress wave measurements, e.g. PDA-measurements.

5.2.1.2 Service state stresses

The permitted central compression stress of the steel pipe pile is determined according to the pile driving category. In general, large diameter steel pipe piles are designed in pile driving categories IA or IB, where the permitted maximum central compressive stress is 0,58 σ_{sa} in category IA and 0,50 σ_{sa} in category IB.
If the steel pipe pile is subjected to bending or shear loadings, the structural bearing capacity of the pile is determined for normal force, bending and shearing according to the instructions of "Suomen rakentamismääräys-kokoelma" relating to steel structures and supplementing steel structure instructions of the Finnish National Road Administration (TVH 723449) /32/.

Corrosion reduction of the pile should be considered, when determining the stresses in service state.

5.2.1.3 Buckling

The structural resistance of the steel pipe pile against buckling is determined according to the Finnish pile driving instructions LPO-87 /16/, paragraph 3.475, allowing for the corrosion reduction of the pile.

5.2.2 Composite pile

The completed composite pile is a steel pipe pile filled with concrete in such a way the adhesion between steel and concrete is sufficient to ensure interaction. The composite pile is structurally designed according to the instructions of "Suomen Rakentamismääräyskokoelma" concerning composite structures.

5.2.2.1 Driving stresses

The permitted driving stresses for the steel pipe of the composite piles are determined according to paragraph 5.2.1.1.

5.2.2.2 Service state stresses

The structural resistances of the composite pile against compression, normal force, bending and shearing are determined according to the reference /15/ (Composite Structures, Design Instructions, paragraph 3.1) and supplementing instructions of the Finnish National Road Administration concerning steel structures (TVH 723449) /32/ allowing for the corrosion reduction of the pile.

5.2.2.3 Buckling

The structural resistance of the composite pile against buckling is determined according to the Finnish pile driving instructions LPO-87 /16/, paragraph 3.475 allowing for the possible corrosion reduction of the pile. The bending strength of the pile is derived from formula:

$$EI = E_{cd}\, I_c + E_s\, I_s + E_r\, I_r \tag{21}$$

E_{cd} = *Young's modulus for concrete*
I_c = *moment of inertia for concrete cross section*
E_s = *Young's modulus for reinforcement steel*
I_s = *moment of inertia for reinforcement*
E_r = *Young' s modulus for steel pipe*
I_r = *moment of inertia for steel pipe*

5.2.3 Concrete pile with steel casing

The steel casing of this type of concrete pile acts as a mould for excavation and casting. The structural bearing capacity of the pile is determined by the bearing capacity of the reinforced concrete cast within the steel casing.

5.2.3.1 Driving stresses

The permitted driving stresses for a steel casing are verified according to paragraph 5.2.1.1.

5.2.3.2 Service state stresses

The structural resistance of the concrete pile with a steel casing against compression, normal force, bending and shearing is determined according to the instructions of "Suomen Rakentamismääräyskokoelma" concerning concrete structures and supplementing instructions (TIEL 2172073-2000) /30/ and Bridge loads (TIEL 2172072) /23/ published by the Finnish National Road Administration.

> When casting underwater the concrete strength class is normally K 30, which requires a cement content of 350...400 kg/m^3 while the relation between water and cement is at maximum 0,6.

5.2.3.3 Buckling

The structural resistance of the concrete pile with a steel casing against buckling is determined according to the instructions mentioned in paragraph 5.2.3.2 and valid instructions for large diameter piles.

5.3 Consideration of corrosion

The corrosion of the steel piles should be allowed for when determining the structural resistance of the pile in service state. In close-ended steel pipes, significant corrosion occurs normally only to the external surface of the pile. In open-ended steel pipe piles internal corrosion should also be taken into consideration. If the point of the open-ended pile is permanently below the ground water level and the upper head of the pile is hermetically sealed, the pile forms a closed, air-tight casing and corrosion of the internal surface of the pile becomes negligible.

The corrosion protection methods for steel pipe piles are:

- alloys,
- cathodic protection,
- organic and inorganic coatings,
- concrete coating or concreting.

The estimated effects of the corrosion can be taken into account by overdimensioning.

5.3.1 Overdimensioning

By overdimensioning the wall thickness of the pile is increased to such extent, that the wall thickness of the structure will be sufficient to bear the designed loading even after the corrosion during the planned service life has occurred. The corrosion allowance necessary depends on the planned service life of the structure and the aggressiveness of the ground or water surrounding the pile.

The corrosion allowance for steel pipe piles is designed for 100 years service life. In natural soil, where no humus, sulphide, impurities or considerably low pH or specific resistivity values occurs, the corrosion allowance is 2 mm/100 years for each corrodible surface below the governing lowest ground water level or 1,5 m below the bottom of the waterway.

The use of overdimensioning as the only preventive measure against corrosion above the governing lowest ground water level or above the bottom layer of the waterway (bottom of the waterway -1,5 m) normally requires corrosion investigations to determine the underground corrosion (cf. 2.2.6). In small bridge projects and in soil or water areas, where the corrosiveness can be with good reason assessed to be minor, overdimensioning according to *table 6* can be applied.

Table 6. Recommended corrosion allowances in normal corrosion conditions [mm] for each corrodible surface in 100 service years.

WATER AREA			SOIL AREA	
Zone	Sea	Inland	Zone	
> HW + 1,5	4	3	Ground level + 1,0	3*
HW + 1,5 ... NW - 1,5	10	6	Ground level+1,0...HW+1,0	4*
NW – 1,5 ... Bottom - 1,5	4	3	HW + 1,0 ... NW - 1,0	4
< Bottom - 1,5	2	2	< NW - 1,0	2

* If the piles are exposed to the effect of de-icing salts, the corrosion protection or the corrosion allowance should be reviewed.

5.3.2 Special steels

Improving of the corrosion resistance with steel alloys is not actually a corrosion protection method. Small amounts of copper, nickel, manganese or chromium improve the corrosion resistance of steel in air. In structures embedded underground these conventional small amounts have no effect. Considerable protection is achieved only by using acid-proof molybdenum alloyed stainless steel, with a price ten times that of normal carbon steel.

5.3.3 Cathodic protection

Cathodic protection is a corrosion protection method, where the corrosion current is compensated with a protective reverse current. The electrode potential of a metal falls to the immunity area, when in practice the corrosion reaction is significantly retarded. The potential reduction can be achieved either with a contact with a base metal, using so-called sacrificing anodes or an external direct current supply.

The requirement for the application of the cathodic protection is, that the protected steel pipe piles are surrounded by conductive medium, such as water or moist soil.

The method applied depends on the protected structure, the magnitude of the protective current, the electrical resistance of the environment, availability of the direct current and economical factors. Application of cathodic protection requires an approved plan.

In the design of cathodic protection other metal structures, with ground contact, are considered. They should also be protected to prevent stray current corrosion.

5.3.4 Organic and inorganic coatings

The activity of the corrosion pair is prevented or retarded with organic coatings. The most common organic coating materials used on steel pipe piles are epox, tar epox, bitumen, polyurethane and polyethylene.
Steel pipe piles are usually applied as driven piles, when the application of coatings is problematic. During the pile driving coatings are easily scratched or torn. Lifting mechanisms, guiding structures and coarse grained soil layers cause scratches to the coating, which concentrate the corrosion. The protection of the scratched coating can be supplemented with cathodic protection. If the coating should be used, the risk of damaging can be decreased e.g. by placing the fill

layers after the pile driving is completed or by coating only the upper part of the pile in a trench.

The most common inorganic coating is zinc. Zinc protects steel also cathodically. If the zinc layers are damaged and the steel layer becomes exposed, zinc corrodes. The protective effect of the zinc layer terminates, when the zinc layer has corroded away. The galvanization of the steel pipe piles is normally difficult to put into practice due to the large dimensions.

5.3.5 Concrete coating

The steel pipe can also be separated from the surrounding ground with a concrete mantle (*figure 19*). In addition, the alkalinity of the concrete creates a protective oxide layer on the surface of the steel. The concrete mantle is necessary to protect only the upper part of the pile, which is often in the most aggressive environment. Concrete can be cast into a predrilled hole or a working pipe, which is driven around the pile after pile driving. There is a end plate fitted to the shape of the pile in the working pipe.

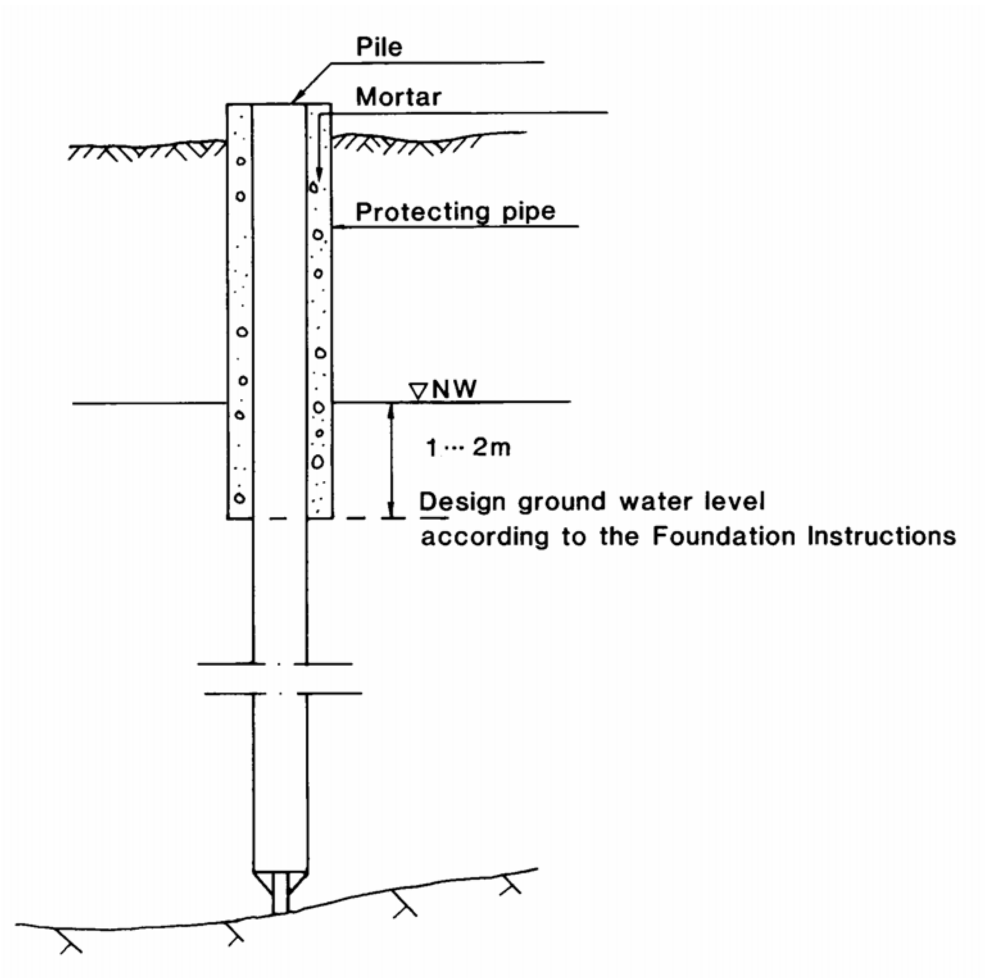

Figure 19: Corrosion protection with concrete mantle /9/.

Also internal casting can be applied in steel pipe piles, where the concrete and reinforcement is designed to carry, if necessary, all loadings resting on the pile. The steel pipe pile acts as a casting mould, which can corrode away completely. In spite of that the pipe pile should be designed for driving stresses, if the bearing capacity is verified during driving. The concrete casting is necessary only in the most aggressive area of the upper part of the pile (*figure 18*).

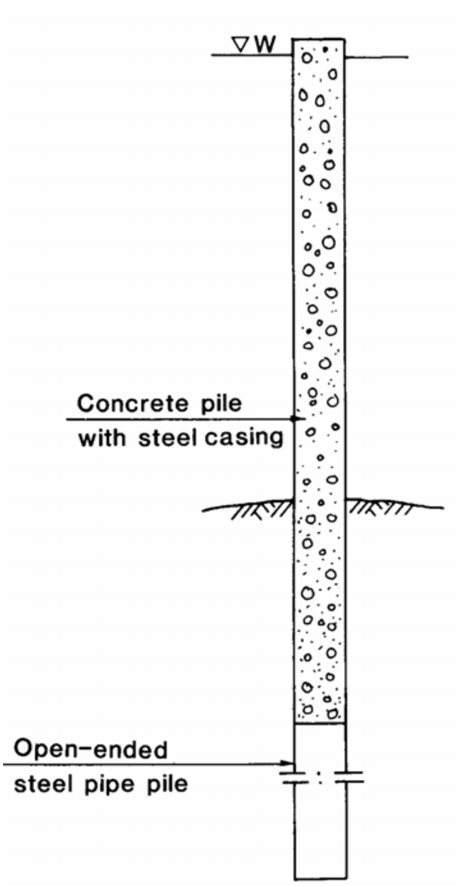

Figure 20: Preparing for corrosion by designing the upper part of the pile as a concrete pile with a steel casing /9/.

6 PILES AND PILING EQUIPMENT

6.1 Pile pipe material and quality requirements

Steel pipe piles used in bridges are usually welded steel tubes. The tubes are produced from hot-rolled steel sheet using either longitudinal or screw welding joints. Also steel pipes produced using other methods can be used providing they satisfy the quality requirements given in this instruction.

6.1.1 Steel grades

Materials used for steel pipe piles are general structural steels that are in accordance with SFS 200. A standard steel grade is Fe 510 and a quality class is C or D depending on the quality class required by the structure or the pile driving (cf. 6.1.2). Where a high structural capacity of the pile is required the amount of steel in pipes can be reduced by using a high strength steel, as pipe steels X 60 and X 70 in accordance with standard API 5L.

6.1.2 Selection of quality specification

The most commonly used quality specification for steel pipe piles is D. The use of quality specification D is especially recommended if the lowest service temperature of the structure is below -20°C and if supported structures are connected directly by welding to the piles or the pile driving is performed in temperatures below -20 °C.

The selection of quality specification is discussed in more detail in instructions concerning steel structures published by the Finnish National Road Administration (TIEL 2173449) /32/.

6.1.3 Dimensions and technical terms of delivery

In regard to the technical delivery terms for measurements, the standard for pipe beams SFS 5001 is applied for the pipe piles with following specifications concerning tolerances:

External diameter	± 0,5%	calculated from the circumference
Wall thickness	+ 10%	
	- 5%	
Length -	0	
	+ 50 mm	
Linearity	< 0,1%	of the length
Irregularities of the head		< 2 mm

The irregularities of the head means the accuracy of the cutting of the pile (*figure 21*). Any irregularity in the shape of the pipe edged to be welded can be detected with a plate installed to the head of the pile. Gaps between the plate and the pile head indicate irregularities.

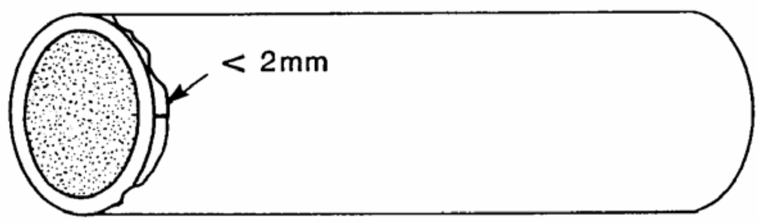

Figure 21: Irregularity of the pile head /13/.

Right-angle accuracy of a head < 0,5% of the external diameter d
or ≤ 4 mm

The right-angle accuracy of the pile head means the right-angle of the cutting compared to the axis (*figure 22*). The right-angle of the pipe head can be measured using a square or a wide steel band installed around the pile head.

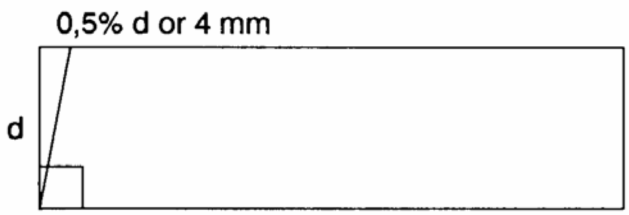

Figure 22: The right-angle accuracy of the pipe head /13/.

Circularity

$$R \leq 2\,\%\,; \quad R = 200\,\frac{d_{amax} - d_{amin}}{d_{amax} + d_{amin}}\,\%$$

d_{amax} = *measured maximum external diameter*
d_{amin} = *measured minimum external diameter*

6.2 Rock shoe and its attachment to the pipe pile

In steel pipe piles resting on rock, whose points are intended to penetrate into the rock, rock shoes equipped with a piece of hard metal are used to prevent sliding. A rock shoe is designed for the compression load so, that a point piece and a point peg sustain at least as much loading as the pile itself. The attachment of the point peg and the point piece are designed for the tension load in order to prevent loosening of the point peg or the point piece due to the tension waves caused by pile driving. The bending load of the rock shoe before the piece of hard metal penetrates into the rock depends on the inclination of the rock surface and on the driving force. The correct attachment of the point peg to the hard Finnish rock may require thousands of blows with low driving energy.

A pile is subjected to the compression loading both during pile installation and in a completed structure. The point piece between the pile and the point peg is designed as a steel structure to transmit loadings during pile driving and in a completed structure between the point peg and the pile. Design of the stresses during pile driving is performed according to the paragraph 5.2.1.1.

The rock shoe is subjected to bending when a pile is driven through a bouldery or stony soil layer and when the shoe contacts an inclined rock surface, in which case the magnitude of the moment depends on the driving energy and the inclination of the rock surface. Rock shoe is designed against bending to ensure that possible sliding on the rock surface does not cause exceeding of the bending capacity of the shoe before exceeding of the capacity of the pile pipe, while supporting earth pressure resultant is situated in distance of 1.5 d from the lower end of the pile pipe.

The rock shoe is subjected to tension during pile driving when the shoe is still in a loose layer. When large diameter steel pipe piles are driven by free fall or diesel hammer, acceleration may be 6000 m/s^2.

Design of the point peg in rock shoe is checked in service state according to the table 4.3225 in LPO-87 in piling category IA using permitted stresses.

The rock shoe is welded to a pile.

Rock shoes equipped with a piece of hard metal (*figure 23*) are made according to a standardized drawing approved by the Finnish National Road Administration.

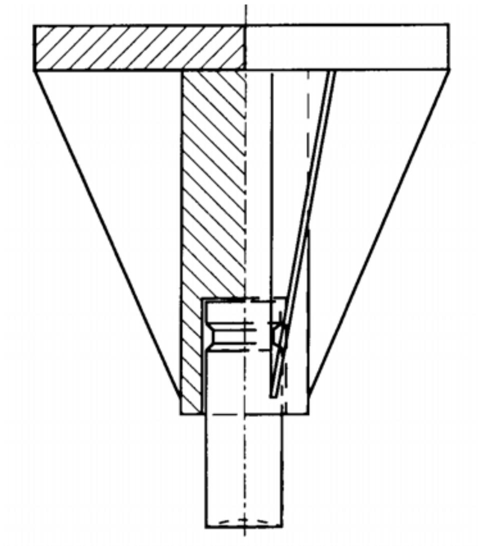

Figure 23: Scheme of a rock shoe equipped with a piece of hard metal.

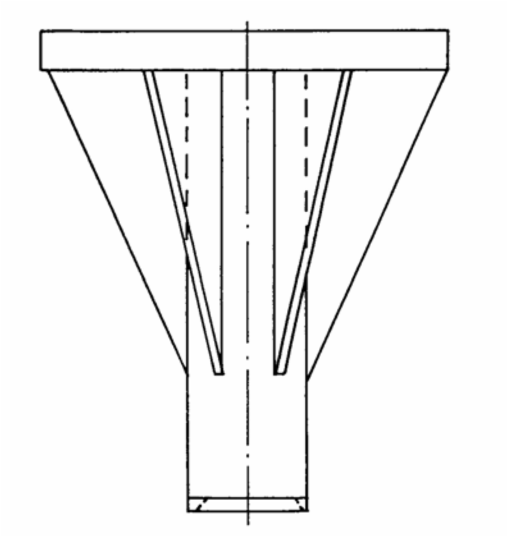

Figure 24: Scheme of a rock shoe made from structural steel.

Rock shoes made of structural steel approved by the Finnish National Road Administration (*figure 23*) can be used to protect the pile point against damage caused by stones and boulders or to centralize loading to prevent the formation of bending action. There is no risk of sliding on the rock surface, if the rock surface is sufficiently even or the supporting soil layer is sufficiently dense and thick to prevent sliding.

In exceptionally demanding situations (cf. 3.2.1.1) sliding can be prevented with a steel peg drilled through the rock shoe. Then rock shoes, that have a hole closed with concrete to enable drilling, are used. Consequently the cross section area of the point grows so large, that driving of the point to the rock is not possible without breakage.

6.3 Bottom plate

The purpose of a bottom plate is to enable a pile to act as a close-ended pile. A bottom plate is designed to sustain a point resistance both during pile driving and in a completed structure. The attachment of a bottom plate is designed to tolerate tension forces caused by pile driving.

The risk of damage to a bottom plate and point may be considerable when a pile is driven in stony and/or bouldery moraine. In which case it is recommended to use a rock shoe.

A pile is subjected to compression both during pile driving and in a completed structure. A bottom plate is designed as a steel structure in order to sustain actions due a resistance of the ground both during pile driving and in a completed structure. Stresses during pile driving are designed according to the paragraph 5.2.1.1.

A bottom plate is subjected to tension during pile driving if a pile point is in a loose soil layer. When large diameter steel pipe piles are driven by free fall, hydraulic or diesel hammer, acceleration may be 6000 m/s^2.

A bottom plate is usually made from a steel plate, which is attached to a pile by welding. A bottom plate can be reinforced with various steel stiffeners, as shown in *figure 25*.

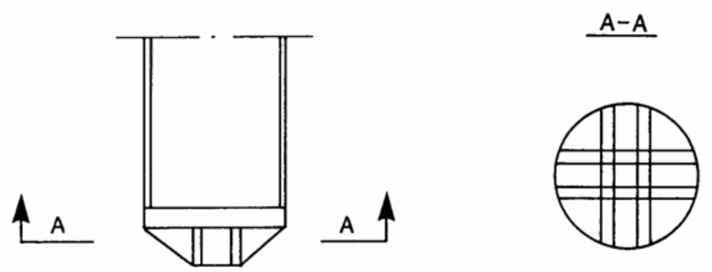

Figure 25: A bottom plate reinforced with steel ribs.

6.4 Tip reinforcement

A purpose of a tip reinforcement is to strengthen the point of a pile driven as open-ended. A tip reinforcement is used in open-ended piles in such ground conditions, where a risk of damage to the point occurs during pile installation. A tip reinforcement increases the geotechnical point resistance while the cross section area of the point enlarges. External shaft resistance decreases 50% in a dense coarse grained soil layer or moraine layer and 25% in a loose soil layer.

A tip reinforcement is typically a steel band with a minimum width of 100 mm, which is welded to the pile point on the external shaft surface (*figure 26*).

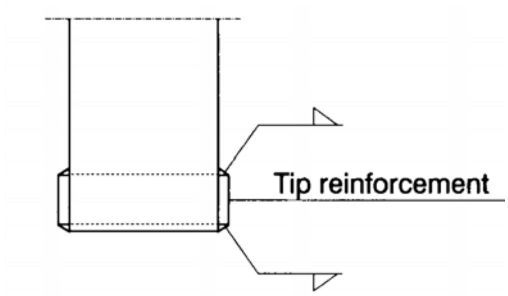

Figure 26: Steel pipe pile with a tip reinforcement.

6.5 Welding joints

Steel pipe piles are extended on site either by arc welding or gas arc welding. The recommended welding process on site is arc welding, due gas arc welding being susceptible to wind and airflow disturbances.

6.5.1 Welding plan

A detailed plan for welding is compiled as a part of a pile driving execution plan. As a minimum, following items should be considered in that plan:

- account of weldability of basic materials,
- welding conditions,
- possible need for preheating during point manufacture and attachment,
- welding sequence,
- groove forms and finishing of the groove,
- positions in welding,
- supporting of an extended pile during welding,
- welding methods and apparatus,
- welding additives to be used: rods, bars, powders and protection gases,
- justifications for selection of welding additives, welding technique and welding values with a method test, if necessary,
- number and order of layers presented as a drawing,
- qualification of welders,
- possible after-treatment of the welds,
- possible heat treatment of the welded parts and
- controlling of welds.

A method test is not usually required, when welding is based on the recommendations of basic material or additive manufacturers. A method test should be made, if the pipe material or additive is such, that on the basis of welding test performed by manufacturers of pipes or additives it cannot be demonstrated that welds fulfill the quality requirements given. A method test is performed according to the standard SFS 3326, as appropriate.

6.5.2 Welding grooves

Steel pipe piles are usually welded from one side, in which case groove form must facilitate effective weld penetration, thus providing the inside part of the weld to be flat and unobtrusive. Groove forms are selected according to the standard SFS 2143 or SFS 4594.

Most commonly used groove forms are ½V- and V-groove. Use of V-groove is recommended in welding of horizontal welds and ½V-groove for vertical welds.

6.5.3 Selection of the welding additive

A welding additive is selected according to a raw material of a pipe following recommendations of a pipe or additive manufacturer.

6.5.4 Fitting and bridging of pipes

After a groove is made and cleaned, pipes are fitted for welding. Special attention should be paid to accuracy of fitting and correct and durable bridging. Shape of a pipe deviating from a circle and different internal diameters cause fitting problems, which can affect welding time and quality. Differences in height of a radial surface must be repaired before fitting. Due to nonradial shape and differences in diameters, edges are not level. By rotating the pipe good fitting can be produced. Due to the welding specifications, 1,6 mm is the maximum permitted gap.

The best way to bridge is to weld a long tack weld as penetration welding carefully and leave it as a part of a bottom layer (*figure 27*). Edges of the tack weld should be made thinner in order to ensure faultless penetration welding in joints. Welding of short tack welds directly to a groove is not recommended.

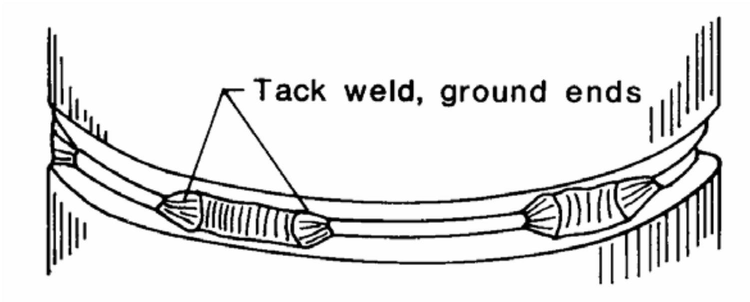

Figure 27: Tack welding /18/.

It is often advantageous to use connecting pieces notched at the groove. An advantage of such bridging is, that the pieces assist the fitting of heavy pipes and groove remains open. Bridging or removal of pieces should not cause damage to the pipe (*figure 28*).

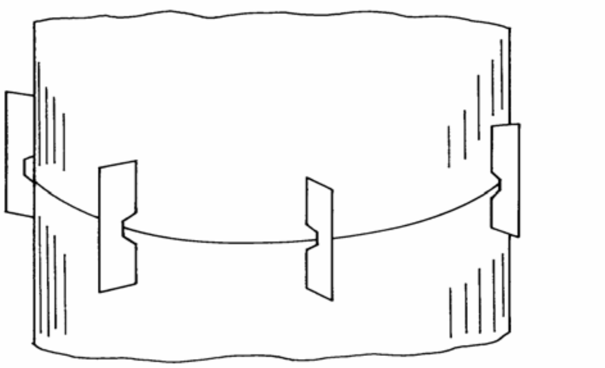

Figure 28: Bridging with connective pieces /18/.

6.5.5 Welding

Welders should be qualified by the qualification certificate in accordance with the standard SFS-EN 287-1. Welders´ qualification certificates or copies of them should be presented to the supervisor of the welding work before the welding work starts. Welding conditions should be organized to provide every possibility to achieve the quality level required in the plan. Special attention should be given to maintain grooves and additives dry and clean.

Welding instructions for horizontal and vertical welding are given in reference /18/.

A steel backing can be used in extension welding of pipes. A material of the backing is chosen from the same steel quality category as the extended pipe. Width of the backing strip should be at least 60 mm. The backing is installed symmetrically with respect to the weld. The backing is attached to the extension pipe. An internal weld is ground to the level of the basic material before the backing is attached. The steel backing is attached to the pipe with welds, the length of which is 50 mm and mutual distance about 50 mm. The a-measurement of the intermittent welds is same as the material thickness of the backing. There should not be air passage between the backing and the pipe wall. The material thickness of the backing is chosen on the basis of the welding technique and welding values, defined in the welding plan. The material thickness of the backing should be at least 4 mm.

The structural design of the weld should be checked according to the standard SFS 2378 relating to the groove form in question.

6.5.6 Quality requirements and control of welded joints

In piles, where a pipe is not acting as a bearing structure, a quality category requirement for joint weld is WC.

In piles, where a pipe acts as a bearing structure, totally or partly, welding category requirement is WB.

Welder's qualification is tested in approval test according to the standard SFS-EN 287-1.

In category WC the quality of welds is checked visually for the total length of the weld.

In category WB in addition to the visual inspection at least 10% of the welds are check with ultra sonic scanning and results are documented to the instruction record for each weld. Ultra sonic scanning is started from the first welding seam.

When backing plate is used the quality of the weld is checked with a method test in accordance with the standard SFS-EN 288-3 corresponding to the pile extensions. No impact strength nor hardness tests are required in testing. At the installation site extension welds are inspected for the total length.

Design of welds for service stresses of a pile and actions during pile driving is performed according to the fatigue factor of the standard SFS 2378 relating to the groove form in question or according to the standard 2373, when the load interchange number determined in the table 3 in the standard 2378 corresponding to the domain of the stress is lowered. Strength calculations for the welding seam are presented in design documents and the quality category of the weld is marked onto the drawings.

6.6 Attachment of the pile head to the concrete structures

A joint between the upper head of the pile and a concrete structure should be designed to be able to transfer all actions subjected to the pile from the concrete structure.

A joint of a composite pile or a concrete pile with a steel casing to a concrete footing is made using connection steels between concrete structures. Connection of an open-ended steel pipe pile is made in a same way, when the upper part of the pile is secured against corrosion with an internal concrete or when the steel pipe pile continues straight as a column without a footing.

Typical joints between an open-ended steel pipe pile and a concrete footing are shown in *figure 29*.

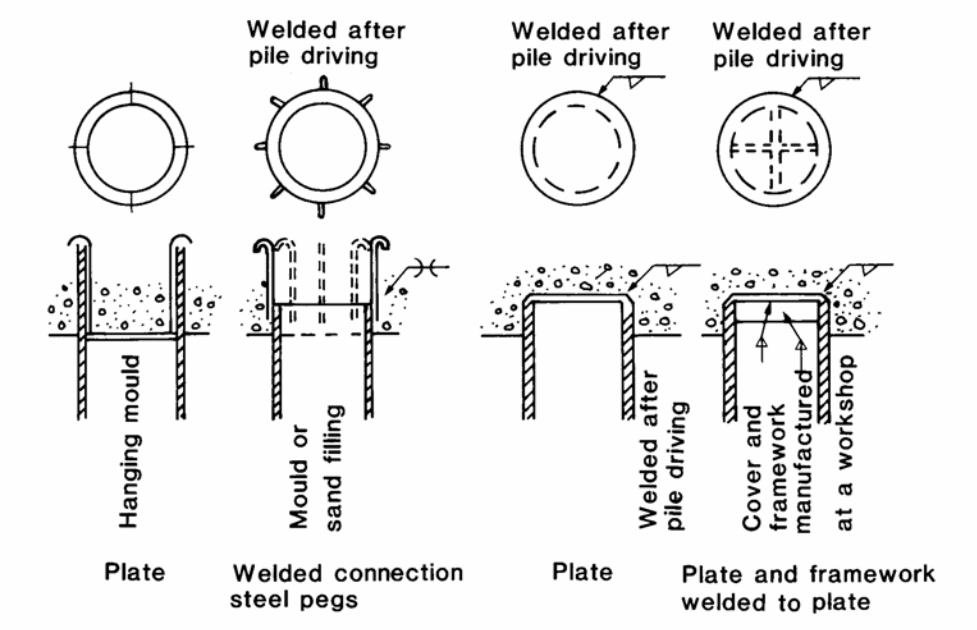

Figure 29: Typical joints between an open-ended steel pipe pile and concrete footing.

7 PLACEMENT OF PILES

7.1 Mutual distances in pile groups

The minimum centralized distances of piles are mutually selected to eliminate any possibility of reduction in bearing capacity or damage during pile driving.

In a pile group the minimum distance between shaft surfaces of parallel piles is normally derived according to the formula:

$$e_l = 300 + 0{,}7\ d \tag{23}$$

e_l = *minimum distance between shaft surfaces of parallel piles*
d = *diameter of the pile, [mm]*

In severe ground conditions the distances should be greater. If the piles are "steering" each other, e.g. in pipe pile walls for coffer foundations, the minimum distances displayed above can be reduced. Then the piles should normally be driven open-ended.

If a pile group disperses downwards, the upper end of the piles can be located within smaller distances than the above mentioned.

If there are crossing piles in the pile group, the crossing point should be designed as near the ground surface as possible. The mutual distance of crossing piles depends on the depth of the crossing point of the piles. The minimum distance [mm/m] is derived from formula:

$$e_r = 50\ l_r \geq e_l \tag{24}$$

e_r = *the mutual minimum distance of the crossing piles, [mm]*
l_r = *depth of the crossing point, [m]*

7.2 Distances from other structures

The minimum distance from adjacent structures is determined separately for each case. It depends on the state of the adjacent structure, the structure, the foundation method and presumed displacements of the ground.

In the vicinity of sensitive structures open-ended piles should be used. If plugging does not take place, the piles can be driven using distances given in paragraph 7.1. In driving a close-ended or plugging pile near large diameter piles the minimum distance between shaft surfaces of the piles should be at minimum 2 d, where d refers to the diameter of the driven pile. Correspondingly the minimum distance between shaft surfaces of the piles should be next to concrete pile groups 3 d, individual concrete piles 4 d, wooden piles, slender steel piles or piles in bad condition 5 d. If a new pile is driven deeper than previous piling, the settlements caused in the old piling should be assessed 65-67 separately.

7.3 Deviations

Permissible deviations for the location and orientation of a pile should be considered individually for each case, because extremely exacting precision requirements slow the pile driving. Permitted deviations (cf. 4.8) are indicated in the pile location drawings. Permissible deviations in location, direction and inclination for conventional pipe piling work used if otherwise unmentioned in design are given in *table 7*.

Table 7. Permissible deviations in location, direction and inclination for piles. /17/

Object of deviation	Permitted deviation
Location of an individual pile	±80 mm
Location of a pile in a pile group	±100 mm
Center of gravity of a pile group	±80 mm
Direction deviation, deviation in a horizontally projected direction of an inclined pile	±5° (±87 mm/m)
Inclination deviation, deviation of an individual pile from vertical or inclined direction	±2% (±20 mm/m)
Deviation of an upper head after levelling	±50 mm

8 FOUNDATION PLAN FOR STEEL PIPE PILE DRIVING

A geotechnical plan is represented in the geotechnical part of the specification for bridge construction, which refers to bridge drawings. The general working specification for bridge construction (SYT) is referred, when matters including to the geotechnical plan are appropriately represented in it.

The following minimum requirements for a geotechnical work plan should be addressed:

- general sequence of building,
- pile types and sizes,
- target levels of the piles,
- permitted bearing capacity of the piles,
- permitted location and inclination deviations,
- permitted curving,
- pile driving sequence,
- extending of the piles,
- use of rock shoes, bottom plates, tip reinforcements etc.,
- preliminary pile driving instructions and control procedures (cf. 9.4),
- pile driving hammer and required effective driving energy transferred to the pile,
- planned test loadings,
- instructions for penetration of a stony and/or bouldery filling,
- control of displacements caused by pile driving and preventative measures,
- instructions for observing possible pile rising and preliminary instructions for control and redriving,
- instructions for keeping the pile driving record (model of the pile driving record is in appendix 1),
- possible concreting, quality of the concrete, method for concreting, concrete reinforcement needed and application and location of the control pipes,
- requirements for drawings of the completed piling as build (SYL, TIEL 2210004) /27/ and
- possible after-monitoring measurements.

9 PILE DRIVING

Appropriate sections of the general specification for bridge construction (SYT) are followed in piling work, and in Quality Controlled Construction -contracts the general quality requirements (SYL) are followed.

9.1 Suitable piling rigs

Hammers, which have sufficient driving energy compared to the required bearing capacity of the piles, are suitable for driving large diameter steel pipe piles, while the bearing capacity of the piles can be verified on the basis of the piling work. The drop height or the piston stroke speed should be sufficiently large as to facilitate the exploitation of the structural capacity of the pile in pile driving, thus facilitating effective piling.

The drop height and the piston stroke speed should be adjustable to enable control of the driving energy in all driving stages. The drop height of the hammer should be determinable during the piling work with accuracy of 0,1 m.

The relation between the cross section areas of the hammer and the steel should be as small as possible, but not less than 1. This ensures, that the driving energy transmitted to the pile is as large as possible.

The installation of Franki pipe piles and the installation of the working pipe of the Franki pile are performed using the Franki piling hammer.

The final set of the steel pipe piles cannot be driven using a vibratory hammer, because the bearing capacity of the piles cannot then be evaluated on the basis of the installation work and the bearing capacity remains lower than the bearing capacity of a corresponding driven pile. However, steel pipe piles can be installed with a vibratory hammer before the final set in such soil layers, where the shaft friction is not utilized in geotechnical design.

The structure of the piling rig should enable sufficiently close monitoring of the pile penetration into the ground and the interruption of the driving if necessary.

The guide bars for the mast of the piling rig should be installed sufficiently rigidly to the required inclination. When a piling hammer hanging from a crane is used, the pile should be sufficiently supported by other supporting structures. The whole piling rig should be supported and assembled in such a way that no swinging occurs during piling.

The piling rig should meet all aspects relating to the occupational safety.

> Suitable piling devices for driving steel pipe piles include piling rigs with free fall hammers, free falling diesel hammers, hydraulic hammers and diesel hammers with accelerated hammer blow. The drop height

potential should be 1,5...2,0 m, if the hammer falls freely, and 2...2,5m, if the hammer is hanging from a cable. If the hammer blow is accelerated, the maximum speed may be 7 m/s. The exploitation rate of the steelstress in pile material should be checked on the basis of the estimated speed of the hammer or the piston. A more recommendable way is to measure the driving stresses in connection of the dynamic test loadings. The steel stresses should remain in range presented in paragraph 5.2.1.1.

The driving stresses of Franki pipe piles, which are driven from the lower end, have no noticeable effect to the upper part of the pile pipe. Thus the drop height of the Franki piling hammer may be as much as 6 m. Strengthening of the lower part of the Franki pipe piles may be necessary, if the pile is driven into a stony or bouldery soil layer.

9.2 Centralizing the blow

The impact of the hammer or the piston to the pile point should be central and parallel to the axis of the pile.

When slow driving hammers are used, a driving cap or a dolly should be used to centralize the impact and to protect the pile head. When steel pipe piles are driven no driving cap is needed, if the pile and especially the pile head sustain the maximum stresses during pile driving. If no driving cap is used, a dolly manufactured from a thick steel plate should be used between the hammer or the piston and the pile head. The structure of the driving cap or the dolly should allow centralizing in relation to the hammer or piston and the pile.

The driving cap or the dolly is centralized with steel plate guides installed directly to the pile head. The driving cap or dolly is firmly attached to the frame, which should be centrally attached to the steel plate.

A suitable material for the driving cap is azobe. The cross section area of the driving cap should be equal to the cross section area of the hammer. A suitable length of the driving cap is 300...800 mm. The driving cap is changed before the wooden part is so worn, that the blow meets the frames. The driving cap should be as light as possible.

The material of the dolly should sustain the stresses caused by driving. The dimensions of the dolly are determined on the same basis as for the driving cap.

When a vibratory hammer is used, the hammer is attached centrally to the pile head.

9.3 Verification of the driving energy

The effective driving energy transmitted to the pile from the piling rig, including the driving cap or the dolly, can be verified by utilizing the dynamic test loadings with stress wave measurements.

Verification of the driving energy transmitting to the pile is performed utilizing the dynamic test loadings. Even for the same driving rig the effective driving energy can be remarkably lower when driving inclined piles compared to the vertical piles. It is recommended to check the driving energy transmitting to the pile in the start of the piling work and it should be rechecked, if the piling rig or parts of it are changed during piling work, e.g. the driving cap or the dolly.

When hydraulic hammers are used, the losses of the driving energy are minimal. When the free fall hammer is used, the driving energy depends on the inclination of the pile and on the driving cap. The energy losses of the hammers hanging on a cable are large. The effective driving energy of old diesel hammers may differ considerably to the values given by the manufacturer.

Pile pipes equipped with a bottom plate or a rock shoe, which are driven from the lower end of the pile, or Franki pile pipes, the bearing capacity which is determined with PDA-measurements, must be designed to sustain test driving to the upper head of the pile.

9.4 Compiling and verification of pile driving instruction

The designer complies driving instructions for each combination of a pile type and ground conditions. Instructions are included in the foundation plan (cf. 8) and they are defined on the basis of the information obtained regarding the piling rig and after the test piling and loading.

The procedure of compiling is as follows:

1) On the basis of the ground conditions, the selected pile type and size and the target level, the designer complies the pile driving instruction, where the stages of the piling work are preliminarily presented and the effective driving energy required from the driving rig is determined. Using the piling formulas presented in paragraph 4.1.3, a reasonable requirement for final set can be obtained, when the target ultimate load is chosen to be 2,5...3,0 times the permitted geotechnical bearing capacity. The maximum permitted drop height H can be calculated with formula 20 using $\sigma_{max} \leq 0{,}9\ \sigma_{sa}$, where σ_{sa} is the yielding strength of the pile material.

2) The contractor selects the appropriate piling rig in order to reach the required driving energy using the drop height or the piston stroke speed within the permitted values and reports information about the piling rig to the designer.
3) The designer defines the driving instructions according to the piling rig used, if necessary.
4) If no test piling is performed prior to the actual piling, the piling work is started in a test piling nature. The designer complies the final driving instructions on the basis of dynamic test loadings.

   As a minimum, the following items should be presented in the driving instructions:
   - instruction for drop height or for driving energy used in different stages of driving,
   - instructions for filling close-ended piles driven from the upper head with water to ease penetration, The pile should not be filled, not even partly, with concrete before the geotechnical designer has verified the geotechnical bearing capacity and the structural designer has checked the actual pile loads on the basis of location, inclination and linearity measurements (cf. paragraph 10.5),
   - instructions for a careful driving of the point peg in the rock shoe to the rock,
   - instructions both for final driving and for final set,
   - method instructions for the appearance of expected special features during pile driving,
   - instructions appertaining to reporting duties and defined instructions for keeping pile driving record and
   - possible dynamic test loading.

9.5 Support and control of the pile during driving

A pile should be supported in the beginning of driving or installation to keep in the designed location and in the designed inclination. During the pile driving or installation the pile should be continuously supported to its place at the cutting level. If it is unreasonably difficult to support the upper head of the pile, the location and inclination of the upper head should be monitored during piling work. If the pile is observed to deviate from the designed location or inclination, the location and inclination correction should be attempted.

If the pile point meets a stone or a boulder in the ground and intends to change its direction, it should be possible to loosen steering to enable avoidance of the obstacle without the pile bending. If there is a need to loosen the steering more than the tolerances given for the location and inclination of the pile would permit, the structural design of the pile foundation should be checked (cf. 10.5).

10 SUPERVISION OF PILE DRIVING

In supervision of pile driving the supervision instructions for bridge construction (TIEL 2220001 and TIEL 2210002) are followed, as appropriate /24, 25/.

10.1 Expert supervision

The geotechnical designer of the steel pipe piling is responsible for the expert supervision, the purpose of which is to verify and approve the geotechnical bearing capacity of piles and pile groups. The structural designer is responsible for the expert supervision for piles, the purpose of which is to verify and approve the structural bearing capacity of piles and pile groups.

10.2 Site supervision

The builder nominates a piling supervisor for the building site. The piling supervisor ensures, that the piling work is performed according to the foundation plan and the piling work plan accepted on the site. Tasks of the piling supervisor include:

1) checking that material certificates of the pile material have been delivered to the builder and that the pipe materials are in accordance with the specifications,
2) verifying that the piles and their piling equipment delivered to the site correspond to the plans,
3) verifying that the working plans of the contractor are applicable to the conditions prevailing on site, and obtaining the geotechnical and structural designer's approval of the working plans,
4) verifying the qualification of the contractor's piling foreman and qualifications of the welders welding pile extensions (SYL 4, TIEL 2210006) /29/,
5) taking care that the piling foreman is aware of the pile driving instructions valid at each time,
6) supervising that the pile material is handled and stored as appropriate,
7) supervising that the contractor performs and reports without delay the measurements relating to the pile driving work he is ordered to perform,
8) checking visually the pile welding done on the working site, ordering welds to be checked with ultra sonic measurements according to the specifications, and supervising checks,
9) checking the angle deviations in the pile extension welds, and reporting the angle deviations exceeding the permitted deviation to the structural designer,
10) ordering and supervising pile test loadings according to the specification, and attaining that the geotechnical designer obtains the results of the performed test loadings as soon as possible,

11) supervising final driving of piles, verifying filling of the piling records according to the performed pile driving, and delivering the filled piling records to the geotechnical designer for checking,
12) supervising the measurements of pile location, inclination and linearity, checking the reporting of these measurements, and attaining that the measurement results are delivered to the structural designer as soon as possible,
13) supervising cleaning and casting of the piles to be filled with concrete after the geotechnical and structural bearing capacity has been verified (cf. paragraphs 10.4 and 10.5),
14) bringing to the attention of the contractor any neglected occupational safety,
15) supervising that the working methods of the contractor are appropriate.

In a Quality Controlled Construction -contract the nature of the supervision is spot-checking, and the supervision concentrates especially on the checking of the quality and the stage plans, and on the acceptance of the competed structure.

10.3 Tasks of the pile driving supervisor

In general, tasks of the pile driving supervisor is to attain that:

1) the piling work plan is compiled and approved,
2) piles fulfill the requirements concerning manufacturing tolerances and perpendicularity of the pile heads,
3) a rock shoe corresponds to the plan and it is attached as planned,
4) piles are handled and lifted as appropriately,
5) piles are placed at the designed location vertically or at the designed inclination and are supported according to the pile driving plan,
6) welding grooves of the pile extension are within the limitations of the plan and that welding is performed according to the supervision instructions for welding works and the planned checks of the extension are performed using ultra sonic measurements,
7) planned test pile driving and dynamic pile tests (PDA) are performed and approved driving instructions are obtained from the geotechnical designer,
8) planned measurements of elasticity are performed,
9) the hammer blow remains central and parallel with the pile,
10) the wooden part of the driving cap is changed appropriately,
11) the level of the previously driven piles is monitored by levelling when piles are driven in the vicinity, and that the geotechnical designer is contacted in order to obtain instructions for check driving and redriving, if heaving is observed,
12) final set is driven using the drop height as stipulated in the driving instructions,

13) final set is not interrupted before the settlement of the pile fulfills the final set requirement given in the driving instructions,
14) settlements occurring during final set are measured and recorded immediately in the piling record, which is filled continuously during the pile driving and presented daily to the geotechnical designer (appendix 1),
15) check driving and redriving corresponding to the plan and ordered by the geotechnical designer during the work are performed,
16) pile driving is interrupted, if the temperature is below directive values presented in paragraph 6.1.2,
17) the pile location, inclination and linearity are measured and that results are presented as clear measurement drawings to the structural designer, if stipulated tolerances are exceeded,
18) piles to be filled with concrete are cleaned and cast after the geotechnical and structural bearing capacity has been verified (cf. paragraphs 10.4 and 10.5),
19) necessary checks are performed in cast piles in order to ensure underwater casting,
20) the need for additional piles is checked with the designers according to paragraphs 10.4 and 10.5.

List of measures presented above should be complemented, if necessary, corresponding to the pile type and driving hammer used and to the conditions prevailing on the building site in order to ensure that the required pile bearing capacity is achieved.

10.4 Verification of the geotechnical bearing capacity

In order to verify the geotechnical bearing capacity the piles must reach their target levels, to satisfy the requirements given in the instructions for final set, and stipulated test loadings should be performed to them. Information regarding the penetration levels of piles and pile settlements during final set should be presented daily, in form of piling records, to the geotechnical designer. Also results of elasticity measurements and dynamic test loadings are to be presented to the geotechnical designer. Based on these data the geotechnical designer determines the actual bearing capacity of the piles and decides as to the possible need for additional piles or other measures to reach the geotechnical bearing capacity.

The more precise method to determine the geotechnical bearing capacity is to perform test loadings, of which dynamic test loadings are most suitable for large diameter piles. Dynamic test loadings should be performed at the start of the pile driving for each different soil condition and pile length, in order to adjust the pile driving plan, and especially the instructions for final set, thus attaining the designed bearing capacity of piles.

10.5 Verification of actual pile loads and structural bearing capacity

The inclination and location of the piles are measured in order to verify the actual loading of piles and the structural bearing capacity of the pile group.

The linearity of the pile is measured in order to verify the structural bearing capacity of the pile. In addition, the integrity of the pile is clarified and that the joints are welded and checked appropriately.

If the location and inclination tolerances required in the plans are exceeded, clear and unambiguously presented results of location, inclination and linearity measurements should be delivered immediately to the structural designer. On the basis of the measurement results the structural designer calculates the actual loadings of the piles by check calculation for the pile group and redesigns the structural bearing capacity of the piles and pile groups and decides on the possible need for additional piles or other measures to reach the required structural bearing capacity. In general, only loadings of the piles, that exceed the permitted deviations of location, should be checked after the pile driving, However, the pile groups should normally be analyzed as a whole according to the realized piling.

The pile cannot be cut and cast until the geotechnical and structural bearing capacity is approved. To finally verify the structural bearing capacity it should be clarified that pile casting is performed according to the plans.

> Piles are extended by welding. Checking measures for welding joints are introduced in paragraph 6.5.6.
>
> Also, the quality of the underwater casting should be verified as required in the plans. If no checking method is introduced in the plan, the checking method for cast-in-place piles according to the general specification for bridge construction (SYT) is followed, i.e. ultra sonic measurements and injection with special hoses. It is not recommended to cast piles, the diameter of which is less than 600 mm, as underwater work. If the casting is, however, performed under the water, the quality control measures should be presented separately in the plan.

10.6 Documentation of the approval of pile driving

When piling is completed, its approval is reported and a realization drawing according to the supervision instructions for bridge construction (TIEL 2220001) is drawn /24/. The realization drawing and all measurement data are filed as appendices to the quality report of the bridge.

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RECORD FOR STEEL PIPE PILE DRIVING

APPENDIX 1

Building site:	Pile No	
Location:	Record No	Page No /

STEEL PIPE

External diameter: mm Wall thickness: mm Length: mm Weight: kN

Part	Steel grade	Manufacturer	Date of manufact.	Certifications and remarks	
- pipe					
- point					
- extension					

TOLERANCES OF THE PIPE

External diameter	______ % of circumference	Wall thickness	______ %
Linearity	______ % of length	Length	______ mm
Circularity	______ %	Irregularities of the head	______ mm
Right-angle accuracy of the head	______ % of external diameter or ______ mm		

INSTALLATION OF STEEL PIPE STARTED: DATE TIME ENDED: DATE TIME

Piling rig:

Part	Material	Mass (kg)	Cross section (mm^2)	Length (m)
- hammer				
- driving cap				

Designed inclination		______________
Measuring level	__________ +	__________
Level of the pile point	+	__________
Pile length (m)	=	__________
Pile point, reached level	+	__________
Pile point, target level	+	__________

Drop height (m)	Blows (number/s)	Settlement (mm)	Settlement (mm/blow)	Elasticity (mm)	Depth of the point from the measuring level (m)	Remarks (extension of the pile, interrupts, etc.)

ADDITIONAL PAGE FOR PIPE INSTALLATION

APPENDIX 2

Building site:	Pile No
Location:	Record No Page No /

INSTALLATION OF THE STEEL PIPE (CONTINUING)

Drop height (m)	Blows (number/s)	Settlement		Temporary compression (mm)	Depth of the point from the measuring level (m)	Remarks (extension of the pile, interrupts, etc.)
		(mm)	(mm/blow)			

Layer thickness	Soil type boundaries	Soil type	Effective working time	Remarks (excavation device, samples, boulders, interrupts, etc.)
	Measuring level		Work started:	
m			h	
m			h	
m			h	
m			h	
m			h	
m	Base level		h	
Thickness of the plug m			Work ended:	Treatment of the base:
Total effective working time			h	Total working time: h

REINFORCEMENT AND CASTING STARTED: DATE TIME ENDED: DATE TIME

Check of the base: date	time	Approval of the base, controller:	
Performed cleaning measures:			
Reinforcement, main steels:	hooks:	Check pipes:	
Concrete manufacturer:			Strength class K
Amount of cement: kg/m^3		Maximum grain size: mm	Viskocity: sVB
Water-cement relation:	Additives:		
Concrete consumption: m^3,	theoretical: m^3	Test blocks (identifying mark):	
Casting method:			Temperature: °C
Casting interrupted: time		at level +	
Reason for interruption:			

SPECIAL AND CORRECTIVE MEASURES

CHECK MEASUREMENTS

Checks	Designed	Deviation / measured value	Level of the pile point after driving - ___________
Location	☐	ΔX =	Check levelling Date level - ___________
Curvature	< 1 :	ΔY =	Check / redriving Date - ___________
Direction	☐	___________________	Pile cutting level - ___________
Inclination	☐	_______ :1 ________	Complied: ___ / ___ 19 ___

Checked: Date: Number of appendices: