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Connection of timber foundation piles to concrete extension piles

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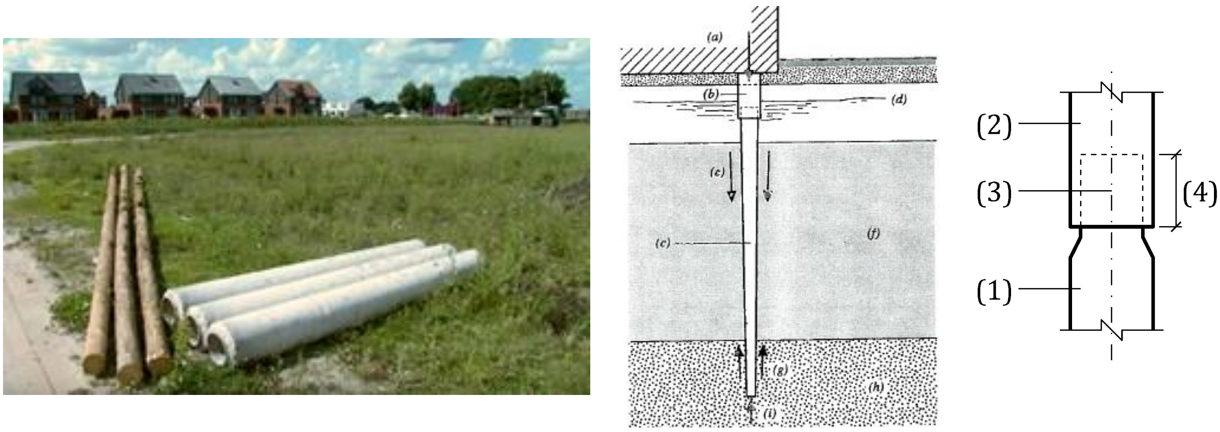
1 Introduction

In many regions of the world, timber piles have been used for centuries for the foundation of buildings and bridges. For historical buildings, it is common that masonry foundation walls are placed upon timber piles and cross beams that remain below the water level. With the introduction of concrete foundation beams in the 20th century, the distance between the head of the timber pile and the concrete foundation beam was bridged by concrete extension piles. Originally made in situ, currently only prefabricated extension piles are used. In the Netherlands foundations of houses were widely realized with timber piles until the 1980s (Van de Kuilen, 1995). Currently, timber piles are mostly used for the foundation of industrial buildings like greenhouses. However, the lacking of relevant mechanical properties of timber piles and connections of parts of piles and of piles with pile extensions together with respective design rules, hinder the wider application of timber foundation piles. For the verification, the strength properties of timber piles and the connections have to be specified in standards or determined by tests, but still test data is available only to a very limited extent.

In the future revised version of Eurocode 5, rules for the design of timber foundation piles will be included. The new Clause to prEN 1995-1-1 drafted by the Project Team SC5.T3 (Project team SC5.T3 ,2020) and by CEN/TC 250/SC 5/WG 3 (Working Group CEN/TC 250/SC 5/WG 3, 2021) covers general rules for timber piles, materials (wood species, grading rules), properties (compressive strength and MOE parallel to the grain in the fibre saturated state, shrinkage, shaft friction), durability, ultimate limit states (axial compression, stability) and execution.

In order to guarantee for sufficient durability throughout the design service life of the foundation, the tops of softwood piles should be permanently located at least 500 mm below the lowest ground water table to be expected at the location of the foundation. If this cannot be met, the timber pile is driven to a level reliably beyond the lowest ground water table and extended by a pile extension made of material exhibiting sufficient durability when exposed to varying ground water levels (Figure 1).

Extension piles are frequently made out of a concrete, either totally out of concrete or with a steel tube on the concrete. In tests in the Netherlands the strength and failure mechanisms of the connection of such a concrete extension pile was investigated, together with the influence of the stiffness of the connection on the overall behaviour of the pile in the soil. This paper presents elements of the draft section for the new EC 5 on timber foundation piles related to this connection and test results on a connection type. The failure mechanisms are analysed and the effect of the found strength and stiffness on the behaviour of the pile in the soil. Specific calculation rules are necessary because with the available rules according to the current version of EN 1995-1-1 [Eurocode 5, 2004)], engineers in practice have concluded that the timber would not fulfil the requirements for strength at the location of the connection.



a) b) c)

*Figure 1 **a)**: Timber piles and concrete extension piles. **b)**: Timber pile with concrete extension pile (b) on top of the timber pile (c). (d) is the ground water level. **c)**: Detail view of the top of the timber pile (1) and the concrete extension pile (2) with a socket (3) of a certain depth (4).*

2 New design rules for timber piles in future EC 5

2.1 Introduction

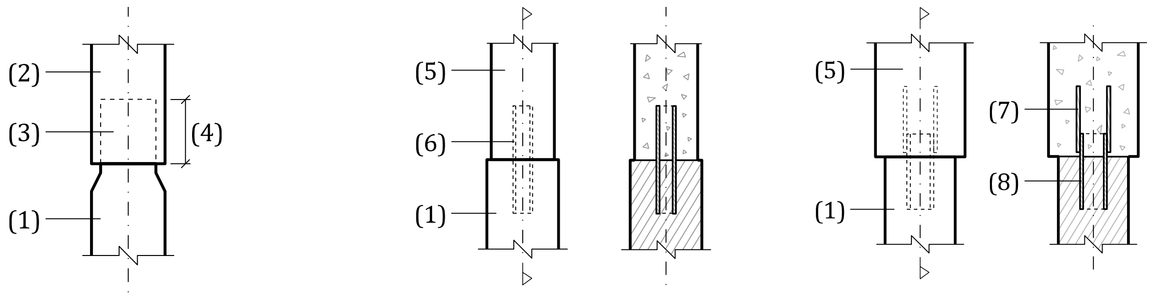
On request by several CEN member states a set of new design rules for timber foundation piles were drafted for inclusion in the future version of EC 5. The scope is on piles, used as foundation for new buildings, subjected to axial compression loading. The objective is to verify the timber pile only; the geotechnical loadbearing capacity has to be determined according to prEN 1997 (Project teams SC7.T4 and SC7.T5, 2019). This standard states that methods used to determine the geotechnical bearing capacity of the piles have to be based on results of in situ load tests or on analytical formulas, of which the validity is priorly demonstrated, using the results of similar load tests. Several National Application Documents supply such analytical formulas in combination with parameters to be used.

One of the main differences with other use of timber in structures is that timber piles under buildings are fully saturated. In the new design rules, this is addressed by applying directly the saturated values for the compressive strength and the MOE determined from test in the verification rules.

2.2 Durability aspects

To fulfil the structural requirements for the entire design service life, special care has to be taken in the design of foundations with timber piles. When piles are used in waterworks or temporary structures where parts are not permanently fully submerged in ground water, durable wood species should be used. However, when piles are used fully submerged under the ground water level during the entire service life, softwood species with a low natural durability like Norway spruce (*picea abies*) can be used without chemical treatment. This is because in that case there is not enough oxygen to activate fungal decay (bacterial decay can also take place under the water level, but this is a much slower process). However, currently in the Netherlands, pine (*pinus sylvestris*) piles are not used for new foundations, because of bad experiences with fungal decay of the sapwood (which has a larger amount for pine than spruce) in situations where the ground water level was temporarily lowered and the pile heads occasionally came above that level.

To guarantee for sufficient durability, it is advised to permanently keep the pile head 500 mm below the lowest ground water level. In that case, a concrete extension pile has to be applied to bridge the gap between pile heads and foundation beams. Types of connections between timber pile and extension pile that are currently used are shown in Figure 2.



a) Socket-type joint

b) Pen-type joint with steel pen (tube)

c) Pen-type joint with steel cylinder / rebars

Figure 2. Different connection types for concrete extension piles with timber piles. Taken from (Project team SC5.T3 ,2020) and (Working Group CEN/TC 250/SC 5/WG 3, 2021).

The principle of type a) in Figure 2 is that a socket at the end of the concrete extension pile is driven over the pile head, clamping the pile head. A steel ring is placed at the bottom of the socket, allowing the pile head to be oversized and enabling removing of split parts.

For types b) and c) a steel pen (tube) is partly integrated in the concrete extension pile and the part outside the extension pile is driven into the pile head. In this case the diameter of the pen is smaller, so the clamping is provided by a smaller part of the pile.

The extension piles as well as their connections with the timber piles have to be able to resist the actions from the structure. The connection strength and stiffness will have an influence on the stability of the pile. This paper will focus on the resistance against tangential (usually horizontal) loads. prEN 1997 describes that the resistance against horizontal loads has to be determined by taking the interaction between pile and surrounding soil into account, but gives no procedure on how to determine this interaction. It is stated that properties like the strength and stiffness of the piles should be based on the material standards. In this paper, a procedure for timber piles with an extension pile is proposed. This paper will focus on the influence of the connections' strength and stiffness on horizontal loads due to asymmetric soil addition, applied to counter effect the lowering of the ground level caused by settlements.

3 Connection between timber piles and concrete extension piles

3.1 Background of the investigated connection type

The investigated connection between timber the pile and the concrete extension piles is of so called socket type which is frequently used in the Netherlands. However, only one calculation model where the maximum depends on the axial loads, which gives a very low capacity was available (Rensman, 1997).

The motivation for this research was the outcome from practice in the Netherlands where the connection between the timber pile and the concrete extension pile was verified for cases where extra horizontal load on the piles was acting due to asymmetric support by the soil because of settlements and soil additions. In that case, the engineers concluded that the pile would fail at the connection due to a combination of compression parallel to the grain and bending stresses. Thereby the engineers had started from the following assumptions:

- The verification rule for bending and compression according to clause 6.19 of EN 1995-1-1was used.
- A linear elastic and completely stiff behaviour of the connection was assumed.
- For the bending properties, a strength class C18 for the pile was assumed, because for this strength class the characteristic compressive strength was closest to that of strength class C18. (The characteristic compressive strength for softwood foundation piles according to the Dutch NAD is 19.8 N/mm^2, related to a moisture content of 12%)

Based on these assumptions the failure mechanism as shown in the bottom graph of Figure 3 b) develops, leading to brittle failure in the pile just below the connection.

The following considerations can be made regarding this outcome:

- It is questionable if the verification rules for a combination of bending and compression according the current version of EC 5 are applicable. Experimental research on combined compression and bending always involves stability issues, and does not focus on very local stress interactions (Buchanan, A.H., 1984). Anyhow, the verification is mainly governed by the bending part, and therefore the assumption for the bending strength plays an important role.
- When the bending moment capacity of the pile is higher than the flexural capacity due to compression perpendicular to the grain as a result of the force couple transferring the moment from the timber pile to the concrete extension pile, a plastic failure can occur (Figure 3b) top graph). Previous research on small diameter roundwood indicate different relationships for the bending

strength and compression strength than for sawn timber (Ranta-Maunus, A. (2000).
- The stiffness of the connection can influence the stress distribution in the pile.
- In Germany there is an approval document for clamping glulam timber columns in a concrete foundation, where the clamping is achieved by casting mortar(DIBt, 2016). However, this approval only covers the use of timber in service class 1 and 2, and it does not give a calculation method to derive the stiffness of the connection. The correction factors to be used for compression perpendicular to the grain and for shear give an indication that higher values for these properties can be expected for confined timber. For compression perpendicular to the grain a value for $k_{c,90}$ of 2.0 is given and a factor $k_{v,c}$ = 2.4 for shear in combination with compression perpendicular to the grain is introduced, where the shear strength may be multiplied with. It can be expected that these strength values also increase for the piles investigated in this study in the fully saturated state.

To get more insight in the consideration given above these aspects, a testing program was performed on a socket type extension pile-timber pile connection (type a) according to Figure 2). With the outcomes of the experiments, the stress distribution in the pile regarding its interaction with the soil under horizontal loads is investigated.

Parallel to this investigation, in situ inspection of 7 piles under the foundation under 3 different houses built in the 1980s was carried out (Lobbe, 2019). There the dimensions of the concrete extension pile were confirmed to be equal to those declared by producers. The inspection showed that the pile heads were fully clamped in the concrete extension pile. The piles showed no significant decay.

The outer diameter of the concrete extension pile was 280 mm and the socket had an inner diameter of 180 mm and a depth of 230 mm (distance 4 in Figure 2a).

For the test series, 12 new fresh Norway spruce piles were ordered with a length of 2 meters and a head diameter of 200 mm. The piles were stored in a mist chamber (at 100 % relative humidity of surrounding air) to ensure that the high moisture content of the piles was maintained until they were tested.

3.2 Test set-up

It was decided to use a steel tube of 193.7 mm x 8 mm to simulate the socket of the concrete extension pile. The steel tube has approximately the same stiffness as the concrete socket and it could be reused for all 12 tests. To represent a socket of 230 mm long, a thick steel plate was welded, to transfer the axial load on the pile (see Figure 3a, top figure). The adopted mechanical scheme of the test set-up, enabling to apply a combination of a normal force and a bending moment on the connection, is shown in Figure 3.

In practice, at the end of the socket (which is smaller in diameter than the pile head), a steel ring integrated, to drive the socket over the pile head to provide complete clamping. In the laboratory, the diameter of the pile head was reduced manually to fit into the socket. For the evaluation of the stresses, the location where the pile head enters the socket is looked at (parameter M_A, in the Figure 3a). There, the diameter was 178 mm.

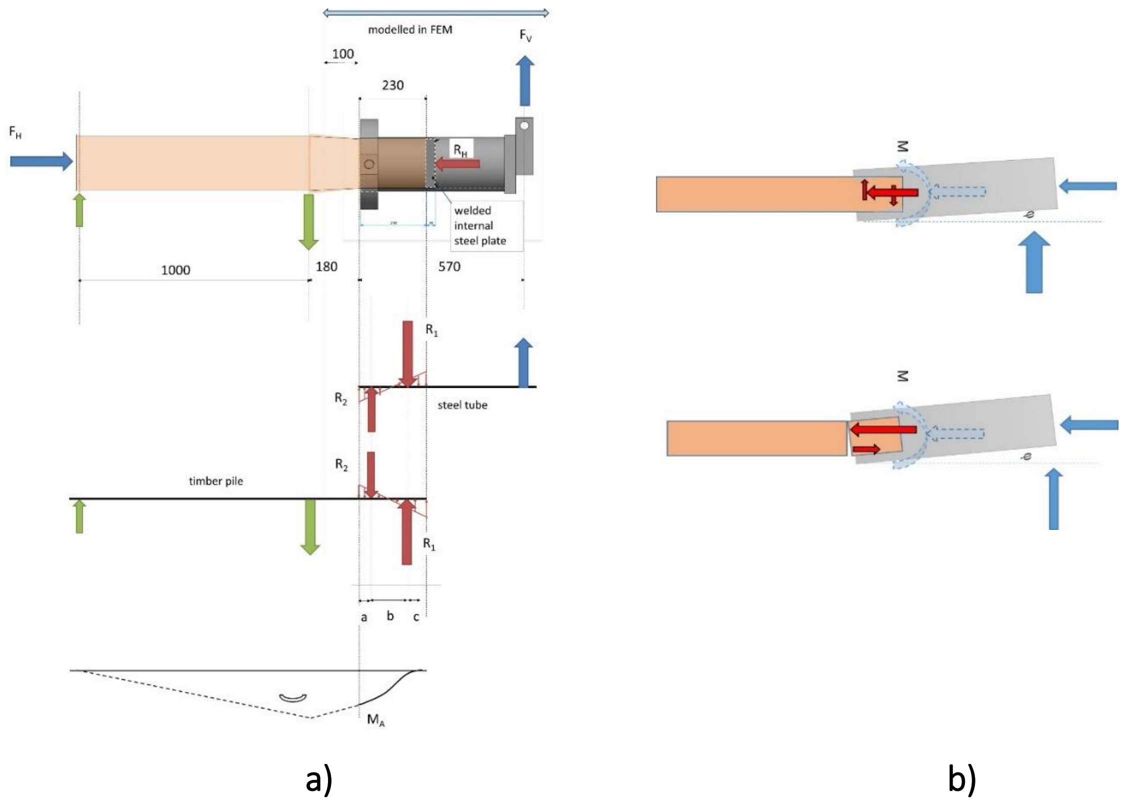


Figure 3 a) Adopted mechanical scheme, with forces on the total system and the steel tube and the timber pile head, assuming the failure mechanism according to the top of figure b). In the bottom of a) a graph with the location of paramter M_A. b) Possible failure mechanisms. Above ductile failure behaviour due to compression perpencidular to the grain, below brittle failure behaviour due to combination of compression and bending parallel to the grain.

After a preliminary test on one specimen, the following loading protocol was adopted:

- At the start of the test an axial force F_H = 50 kN was applied.
- The actuator that measured the vertical load F_V was moved up vertically with a speed of 5 mm/min until a displacement of 50 mm was reached.
- Then the axial force F_H = 50 kN was increased to 170 kN. This force was kept constant for 1 hour, and the change in force F_V due to relaxation was measured.
- After 1 hour, the vertical actuator was moved up with a speed of 5 mm/min until a displacement of 90 mm was reached.
- The vertical displacement of the steel tube was measured at 2 positions spaced 420 mm apart. These measurements were used to determine the rotation angle of the connection, where no bending deformation of the steel tube was assumed.

3.3 Results

Table 1 shows the material properties of the 12 piles at the time of testing.

Table 1. Material properties of the 12 tested piles.

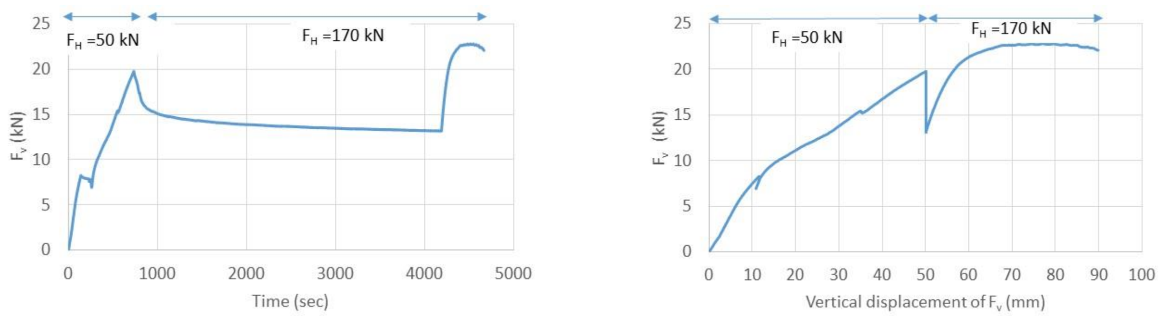
	Density [kg/m³]	MOE_{dyn}* [N/mm²]	m.c. [%]
Mean value	850	10'100	83
Coefficient of variation (%)	7	14	27

*MOE_{dyn} was calculated with $MOE_{dyn} = 4f^2l^2\rho$, where the first natural frequency f (Hz) was measured with the Brookhuis MTG 960 handheld

Figure 4 shows the measured force F_V (see Figure 3) in the vertical actuator for a typical test result against time (a) and vertical displacement of F_V.

The figure shows an initial linear branch, after which plasticisation occurs. After the axial force is increased to 170 kN and kept constant for an hour, the vertical force relaxes by approximately 1/3. When after an hour the displacement of the vertical actuator is increased again, the actuator force returns to the same level as in the first stage and then full plastic behaviour is visible. After 90 mm vertical displacement the test was stopped.

The mean values for the bending stiffness and the bending moment in the elastic phase are C_{el} = 275 kNm/rad and $M_{A,el}$ = 7.7 kNm, respectively. The mean value for the plastic moment is $M_{A,pl}$ = 12.3 kNm. At that point, for all specimen, a rotation angle of 0.1 rad was reached. The behaviour of the connection can therefore be described with a tri-linear approximation (see Figure 5). The elastic-plastic phase is caused by the softening of the wood subjected to compression perpendicular to the grain.



a) b)

Figure 4 a). Vertical force F_V (see Figure 3a) against time. b) Vertical force F_V (see Figure 3a) against the vertical displacement of F_V (right figure). In both a) and b) the applied axial force in that stage of the test are shown in the top of the figures.

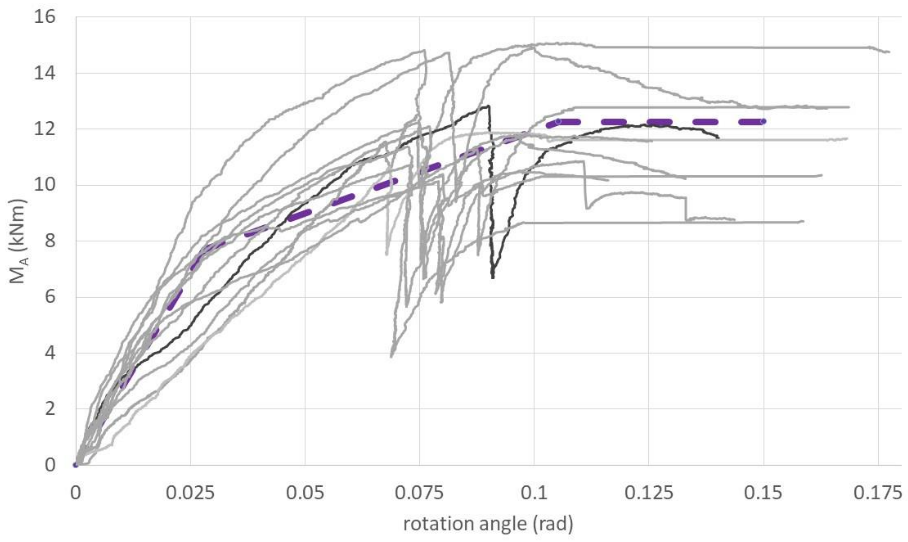


Figure 5. Moment M_A against rotation for all 12 test. The dotted line represents the mean tri-linear representation based on the tests.

The compressive stresses parallel to the grain were 2.7 N/mm^2 in the first phase and 6.9 N/mm^2 in the second phase. The maximum measured bending stresses reached a mean value of 21.9 N/mm^2 (cov = 14.1%). The combination of normal and bending stresses did not cause any (brittle) failure.

It can be concluded that the bending strength of the timber at the connection will not be governing since failure due to compression perpendicular to the grain occurs first. Then plastic deformations can develop.

3.4 Analysis

The compressive strength perpendicular to the grain is a property that is very dependent on the configuration of the material in the structure and the moisture content. Nobel(2014) showed that for the standardised test specimen according to EN 408 the mean compressive strength is reduced by more than 50 % with the m.c. increasing from 12% to 45%, but that for more stocky specimens, this reduction is much less. In the investigated situation the timber pile head is confined by a circular tube. An FEM model was used to investigate the occurring stresses in the elastic phase. Nobel (2014) showed, that the modulus of elasticity (MOE) for compression perpendicular to the grain is much lower at high levels of moisture content (m.c.) for every configuration and for Norway spruce timber could vary between 50 N/mm^2 and 150 N/mm^2. In the FEM model a value of 70 N/mm^2 was used.

The FEM model was programmed in DIANA FEA 10.4 (Ferreira, D and Manie, J, 2020). Timber was modelled with 8-noded linear elastic orthotropic solid elements with a mesh size of 5 mm. For the MOE parallel to the grain a value of 9600 N/mm^2 was adopted, and for the MOE perpendicular to the grain 70 N/mm^2. For the steel tube 8-noded linear elastic isotropic solid elements were used, with a mesh size of 5 mm.

The contact area between steel tube and timber pile in the socket, was modelled with connection elements that could only transfer compressive forces and no tension. Linear elastic behaviour was assumed.

Only a vertical force causing bending moments was applied in the model. The axial compression was not taken into account, because in the experiments no influence of the axial load on the failure mechanism for bending was observed. See Figure 4b, where the loading path does not seem to be influenced by the axial load.

The pile was modelled until 100 mm outside the socket, where it was clamped. The deformations are shown in Figure 6. The model confirms that rotation is governed by the elastic properties compression perpendicular to the grain of the timber.

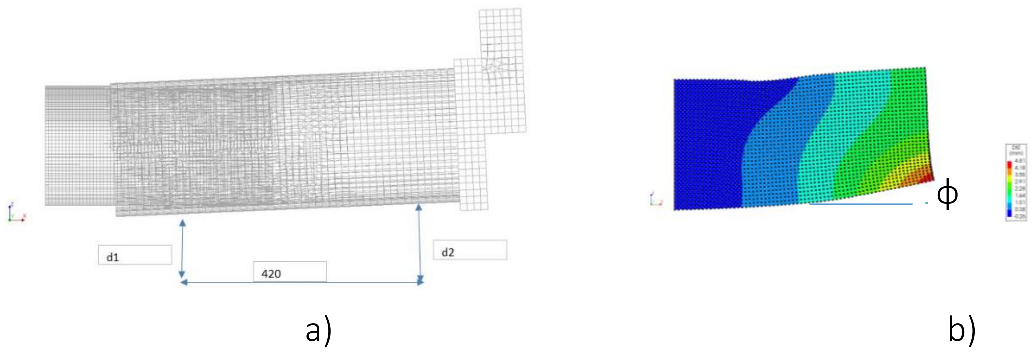


Figure 6. Deformation of the model for a load F_V = 13.7 kN, corresponding to M_A=7.7 kNm. a) Deformation of the timber pile head and the steel tube. b) Deformation of the pile head only.

Figure 7 shows a triangular distribution of compressive stresses perpendicular to the grain at the contact areas at the top and bottom of the timber pile head in the socket. The maximum compressive stress found at the bottom edge was $\sigma_{c,90,max,1}$ = 2.8 N/mm^2 and at the top edge $\sigma_{c,90,max,2}$ = 2.3 N/mm^2.

The length of the contact areas can be taken from Figure 7 (for the bottom edge l_1 = 125 mm and for the top edge l_2 = 105 mm). Based on that the forces R_1 and R_2 according to Figure 4 can be calculated with Formula (1):

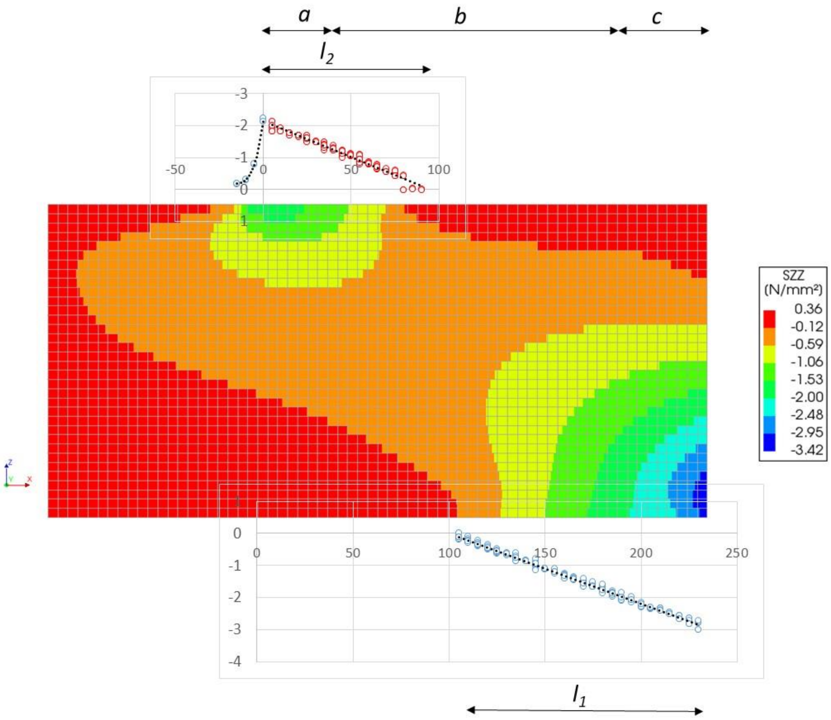


Figure 7. Stress distribution perpendicular to the grain from the FEM model. Top and bottom edge show triangular stress distrubutions for compression perpendicular to the grain where the load couple is intorduced. Dimension a,b and c relate to those defined in Figure 3. l_1 and l_2 *are the contact lengths where the forces* R_1 and R_2 *are introduced (see Figure 3).*

$$R_\mathrm{i} = 0.5\ \sigma_{\mathrm{c,90,max,i}}\ l_\mathrm{i}\ D\ d \qquad (1)$$

In Formula (1) D is the pile head diameter in mm and d accounts for the irregular stress distribution because of the circular contact area. A value of 1.5 gives values for R_1 and R_2 that are consistent and predicts a moment M_A of 7.7 kNm calculated with Formula (2):

$$M_{\mathrm{A,el}} = R_{\mathrm{1,el}}(a + b) - R_{\mathrm{2,el}}a \qquad (2)$$

The calculated moment M_A of 7.7 kNm is similar to the mean elastic moment found in the experiments.

The shear stress found in the FEM model was τ_v = 2.4 N/mm^2, where a value of 2.9 N/mm^2 can be calculated according to elastic theory with the value of $R_{1,el}$ determined with equation (1).

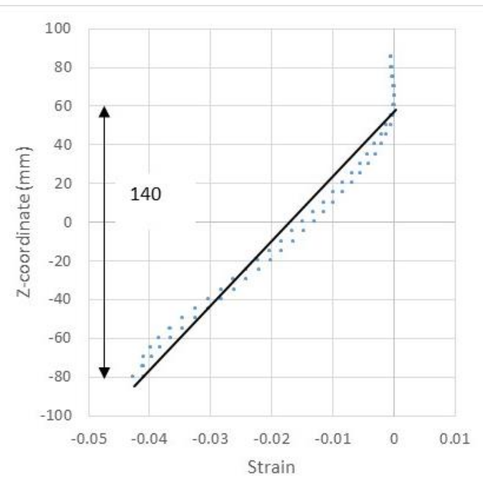
When for calculation of the plastic moment the same contact lengths are assumed but instead of a triangular stress distribution assumed a constant value of $f_{c,90,max}$ =2.8 N/mm^2 both at the top and bottom edge is assumed, the plastic moment can be calculated with Formula (3):

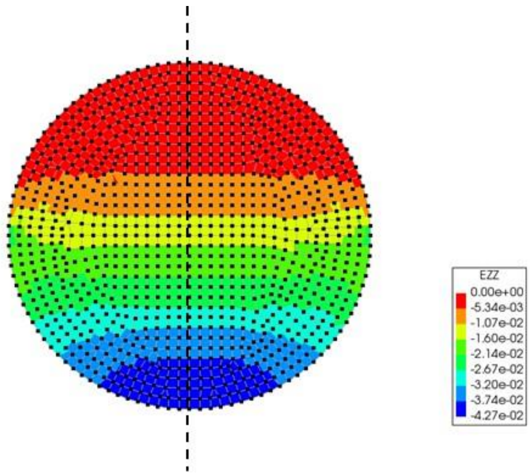
$$M_{\mathrm{A,pl}} = R_{\mathrm{1,pl}}\left(a + b + c - \frac{l_1}{2}\right) - R_{\mathrm{2,pl}}\frac{l_2}{2} \qquad (3)$$

With Formula (3) a plastic moment of 11.5 kNm is predicted, which slightly underestimates the mean value of 12.3 kNm found in the experiments.

The shear stress found according to elastic theory with the calculated value of $R_{1,pl}$ is τ_v = 5.8 N/mm^2. No shear failure occurred.

The rotation of the connection can be determined from the deformation at the bottom egde. The contribution of the steel tube in the deformation can be neglected. The rotation determined by means of the FEM model for $M_{A,el}$ =7.7 kNm was 0.025 rad , close to the mean value of 0.028 rad found in the experiments.





a) b)

Figure 8. a) Strain distribution perpendicular to the grain over the center line of the cross section of the pile at $\sigma_{c,90,max,1}$. b)Strain distribution plot over the cross section with the center line indicated

Figure 8 shows the strains over the cross section at the position of $\sigma_{c,90,max,1}$ at the bottom edge. This shows a linear strain decrease from the bottom outside to zero at 140 mm inwards. Over this length, the average strain can be used. The rotation due to the deformation of the pile at the bottom side due to force R_1 can then be calculated with Formula (4):

$$\theta_{el} = 0.5 \frac{\sigma_{c,90,max,1}}{MOE_{90}} \frac{140}{l_1} \tag{4}$$

This gives a value of 0.022, slightly less than the value of 0.028 found in the experiments.

The results of the FEM analysis was confirmed by hand calculations and underpin that the strength and stiffness of the connection is governed by the strength and stiffness of the timber pile head subjected to compression perpendicular to the grain stresses.

3.5 Material properties for soil modelling

EN 1995-1-1 does not give reduction factors for the MOE for high moisture content for short-term loads. The creep factors given in EN 1995-1-1 only relate to the load duration. For service class 3 this a value of k_{def} = 2 is given. However, the influence of high moisture content and load duration should be accounted for separately. The set of new clauses for the design of timber piles advises to use a value of 80% for the MOE parallel to the grain for saturated piles, compared to the values for service class 1, if no information about the saturated value is available. Form the study presented in this paper, the saturated value is directly available. A value of k_{def} = 1 is proposed for both creep and relaxation. The high value of 2 in the current version of EC 5 (EN 1995-1-1/A1/A2 (2014)) applies to situations with repetitive wetting and drying, which this is not the case for timber piles that are constantly submerged.

In Table 2 the assumptions regarding input to the soil interaction model are listed. Because the main objective is to study which element is governing in case of horizontal loads, mean values are used for both strength and stiffness.

Table 2. Input values for the soil-pile interaction model.

Property	Value
$MOE_{0,mean,sat}$ (N/mm^2)	0.95 $MOE_{dyn,sat,0,mean}$ = 0.95 10'100 = 9'600
$MOE_{0,mean,fin,sat}$ (N/mm^2)	$MOE_{sat,0,mean}$ */(1+1)* =4'800
Pile head diameter (mm)	200
Pile length (mm)	10'000
Taper (mm/m)	7.5
Length of concrete extension pile (mm)	1'000
Ground water level	500 mm below the pile head
Ground level	at the top of the concrete extension pile
Connection properties	Trilinear moment – rotation diagram according to Figure 5.
$f_{m,0,mean,sat}$ (N/mm^2)	30*

* In Ranta-Maunus, A. (2000) a ratio of 2 is found between the bending strength and compression strength of small diameter roundwood. The compression strength of roundwood piles in the Dutch NAD were based on compression tests with a mean value of 20 N/mm^2 for saturated pieces. In the table a ratio of 1.5 is used.

4 Mechanical behaviour of a timber pile with a concrete extension piles in the soil

4.1 Strength and stability for vertical loads

The stability of a foundation pile in the soil under axial loading is influenced by the resistance of the soil. Tangential displacement of the pile will lead to reaction stresses in the opposite direction, which stabilises the pile. A new appendix in prEN 1997 gives a method for determining the stability of the pile including that effect. In EN 1993-5 a verification method for buckling of steel foundation piles is given, together with estimations of the buckling lengths of piles in the soil. The new appendix of prEN 1997 might be aligned with this approach. However, both approaches are valid for prismatic piles and therefore the applicability for wooden piles with a varying diameter in combination with concrete extension piles has yet to be demonstrated.

4.2 Strength and stability for horizontal loads

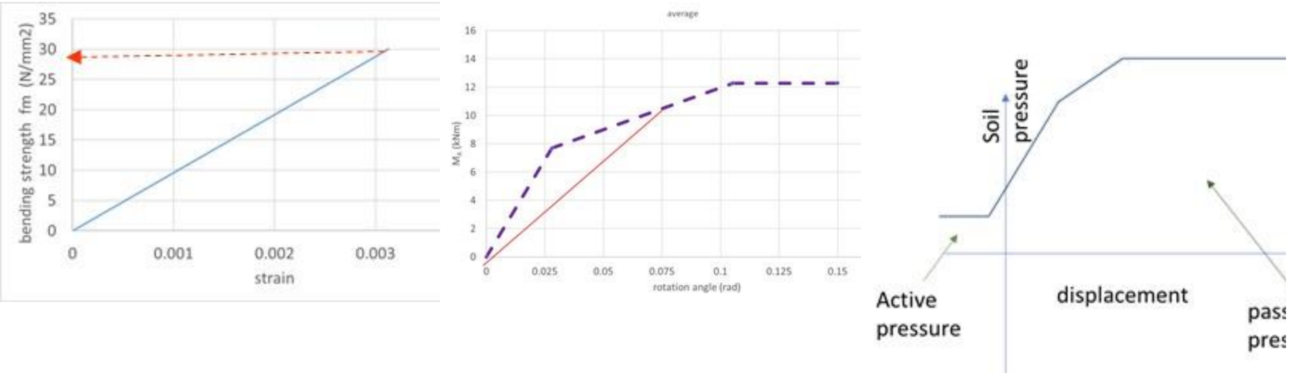
Two kinds of horizontal loads were examined:

1. Horizontal soil displacements, which for example can be the result of differences in ground level on both sides of the foundation pile.
2. Horizontal loads on the head of a vertical foundation pile, for example caused by wind load on a construction.

For both cases 1 and 2, the interaction between pile and surrounding soil can be determined using the model of an elastic beam (the pile) on a plasto-elastic bed (the soil). This kind of model is in engineering practice also used for the calculation of retaining walls.

For some examples the effects have been determined using that method. The calculations were performed with the Dutch Deltares model D-sheetpiling v.19.3. This program allows assigning physical non-linear material properties to the soil and geometric non-linear calculations. The pile properties addressed in 3.5 were used, with the short-term (higher) *MOE* value. The connection between pile and concrete extension is modelled by assuming the average relation between bending moment and rotation as found in the laboratory tests (Figure 5). For reasons of simplicity, the head of the concrete extension (=the connection of the extension with the construction) was in this case assumed at ground level. The soil is represented by the Brich-Hansen reaction model.

Figure 9 gives an overview of the material properties that are used for the modelling. In the modelling, linear behaviour was assigned to both timber pile, concrete extension pile and the connection of the concrete extension pile with the timber pile. However, the latter was manually iterated until the output for the chosen stiffness coincided with a point on the tri-linear moment-rotation diagram.



Timber pile | Connection concrete extension pile-timber pile | Soil (values changing with depth)

Figure 9. Material properties used for the different elements in the modelling.

Table 3 gives the soil profiles used in the analyses. Profiles A and B represent subsequently a lower and upper boundary for the strength and stiffness of the soil. It is debatable if foundation piles would even be applied in soil profile B.

Table 3. Soil parameters used

soil profile	Soil layer	Top side of the layer	Saturated volumetric weight [kN/m^3]	Stiffness of the soil [kN/m^3]	Angle of internal friction / Cohesion [deg] / [kN/m^2]
A (weak)	Peat	ground level (+1m)	11	1'000	15 / 5
	Sand	-9 m	20	10'000	33 / 0
B (strong)	Sand	ground level (+1m)	20	10'000	33 / 0

Case 1: soil displacements

For this case, the tangential soil displacements are assumed to decrease linearly with depth with the maximum at ground level, as shown in Figure 10. The displacement of the pile head itself is assumed zero, i.e. the situation in which there is no displacement of the construction.

Calculations have been made for the case in which the connection between concrete extension and foundation beam is fixed or can rotate freely. In general, the reality is somewhere in between those two extremes.

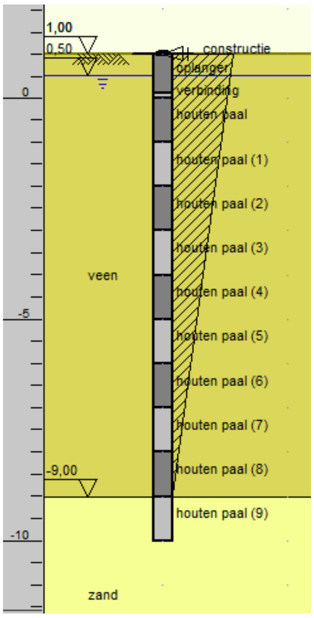


Figure 10. Schematisation of the pile. As indicated, the pile is divided into parts, to account for the difference in cross section. The shaded part indicates the horizontal soil displacement.

In the calculations, the maximum soil displacement at the pile head for which collapse of the pile would occur has been examined.

Table 4. Calculation results for the effect of soil displacements

connection extension and foundation beam	soil profile	Max. soil deformation at ground level near collapse of the pile [mm]	Delimiting mechanism	Max. rotation of the connection pile/extension [rad]
Fixed	A (weak) *)	230	Bending moment deeper in the pile reaches maximum	0.05
	B (strong)	80		0.08
Hinged	A (weak)	250	Max. rotation examined in the lab tests reached **)	0.15
	B (strong)	250		0.15

*) *The calculation result for the indicated case is shown in Figure 11.*

**) *For the 'hinged' cases the maximum rotation of the connection between pile and extension for which the tests were performed is assumed to be the point of collapse.*

It can be concluded that the connection between extension and pile could be delimiting in the case there is a hinge between extension and foundation beam. However, this is only reached at an extreme soil deformation of 250 mm.

If the soil deformation is a slow, long-term effect, so that the low long term value for *MOE* (see Table 2) should be used, it is likely that the values found for the maximum soil deformation would be larger.

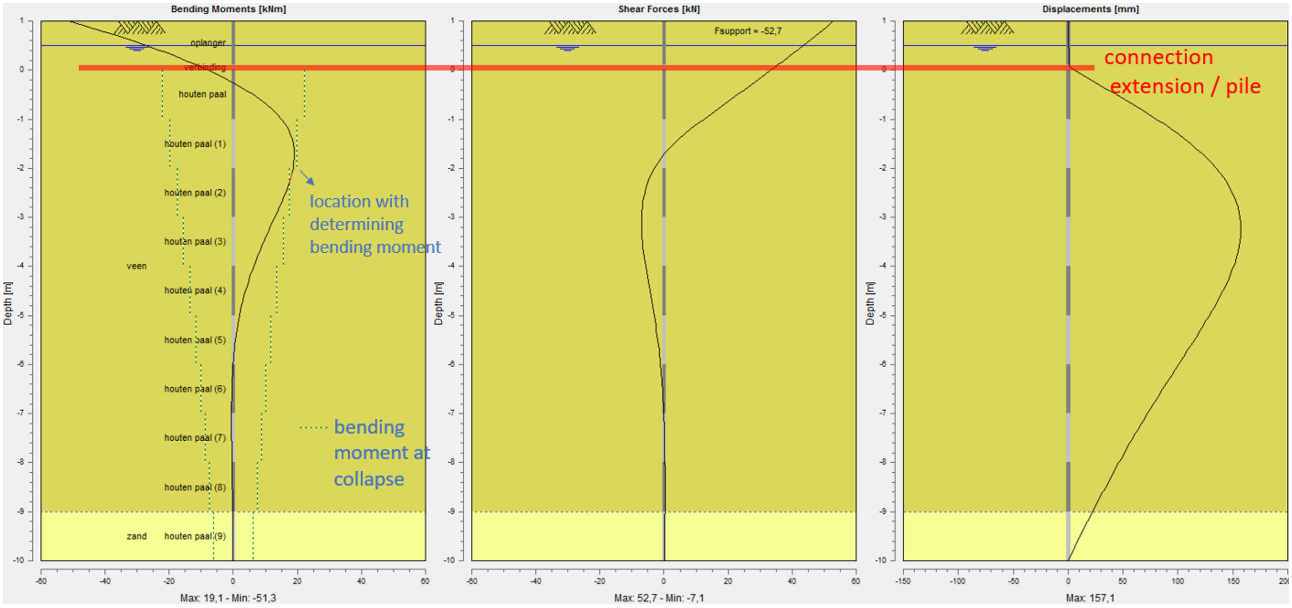


Figure 11. Example of the caculation result of case soil profile A, fixed connection between extension and foundation beam and max. soil displacement of 230 mm.

The procedure used to examine the effects of soil displacements on the foundation pile is well applicable in engineering situations and can be used as an example how the interaction analyses as requested in prEN 1997 can be performed.

Case 2: horizontal loads

For this case, the pile head (head of the extension) is assumed to be able to translate freely, i.e. the situation in which all piles under the construction are loaded equally and respond equally to the horizontal load. Calculations have been made for the case in which the pile head is fixed against rotation or can rotate freely. The maximum applicable pile load at collapse is determined.

Table 5. Calculation results effect of horizontal pile loads

Connection extension and foundation beam	Soil profile	Max. short-term horizontal load on the pile at collapse [kN]	Delimiting mechanism	Rotation of the connection pile/ extension [rad]	Horizontal deformation pile head [mm]
Fixed	A (weak)	53	Bending moment deeper in the pile reaches maximum	0.10	215
	B (strong)	90		0.05	68
Hinged	A (weak)	21.5	Max. bending moment in the connection (12.5 kNm) reached	0.15	200
	B (strong)	21.5		0.15	101

In the case of the fixed connection, the largest values for the horizontal load at collapse were found. This is caused by the fact that the horizontal resistance over a larger part of the pile is mobilised than in the case of a hinge at the pile head. The maximum horizontal force at collapse is for the hinged cases determined by the maximum moment capacity of the connection between pile and extension. Since this moment capacity is strongly time dependant, the results can only be applied for short-term loads. From the tests, it is known that the long term strength of the connection is much lower; a reduction in bending moment in time was observed when the deformation was kept equal. For the fixed cases, the bending moment deeper in the pile was determining for the result, however it would be reasonable to assume that if the bending moment capacity of the connection decreases, it will become determining as well.

5 Conclusions

The set of new clauses to prEN 1995-1-1 on the design of timber foundation piles and on logs to be used as timber foundation piles provides the opportunity of an increase in use of timber for this kind of application. The new clauses describe the verification rules for piles subjected to axial compression and give guidance related to the grading of timber logs, the connection of the timber pile with extension piles and to the evaluation of stability of timber piles.

In this paper, the relationship for one connection type (socket-type) was investigated and a trilinear moment-rotation diagram was determined by means of experiment.

FEM and analytical models confirmed that the material properties for compression perpendicular to the grain are governing for the connections' strength and stiffness properties. This was similar to the behaviour found in the experiment, where ductile behaviour was observed, which can be explained from the plasticization after the compression strength perpendicular was reached. Although because of the specific configuration (confined in a circular tube) the found strength perpendicular to the grain cannot be assigned to other configurations, a good estimation of the stiffness of the connection can be made with FEM models, assuming an MOE_{90} of 70 N/mm^2 for a fully saturated pile. With a conservative assumption for the strength perpendicular to the grain the influence of the connection in a pile-soil model can be investigated also for other configurations than the tested one.

A procedure to study the influence of tangential (usually horizontal) soil displacements on timber piles with concrete extensions piles on top is proposed.

The outcome of a specific situation from practice with high horizontal loads showed that the bending capacity of the timber pile will in most cases be governing and not the investigated socket connection.

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