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WOODEN PILE FOUNDATIONS: STRUCTURAL ANALYSIS AND ASSESSMENT OF REMAINING LOAD CARRYING CAPACITY

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ABSTRACT
Wooden pile foundations are present in many historic towns in Europe and beyond. The technology of making such foundations was developed primarily in southern Europe, but rapidly spread to other countries. The foundations themselves are made with wooden piles of up to 15m, on top of which horizontal wood members are placed acting as interface between structure and piles. Assessment of the state of wood and its mechanical properties is fundamental for a thorough structural analysis, whether for an existing structure or for re-use. In both cases, the type and amount of degradation needs to be addressed. For wooden structural elements, a fundamental analysis is required regarding the mechanical degradation because of long term loading (duration of load effect), in combination with an assessment of the size and severity of biological or physical decay. These effects are responsible for the remaining load carrying capacity and consequently, also for the decision-making process whether the foundation can be re-used. The assessment of this remaining load carrying capacity is done using an integral damage accumulation model, taking into account the severity and type of degradation, combined with the mechanical load components causing the duration of load effect in wood. As such, the structural analysis approach is different from current design standards for new timber structures, and in-line with the principles laid out in ISO standard 13822 for the assessment of existing structures.

Keywords: timber, piles, strength, assessment, decay modelling, service life, re-use

INTRODUCTION

Describing the long-term behaviour of wood in structural applications covers various aspects of structural design as well as the mechanical-physical behaviour of wood in the specific applications. It also relates to the environment in which the structure is placed, as the environment influences the mechanical properties of wood that need to be quantified for the design. The environment may also lead to mechanical, physical, chemical and or biological degradation, which needs to be addressed during the design stage. Current design specifications for timber structures, such as EN 1995-1-1 Eurocode 5, contain specific rules for timber structures, depending on the materials used, the expected loads and their duration, as well as the environment in which the structure will be placed. However, it must be kept in mind that design codes, such as Eurocode 5, account for the more common cases, and are not always applicable on special cases. Specific cases of structures with non-standard environments may require a different or modified approach for which guidance can be found in p.e. EN 1990 or ISO 2394.

For existing structures, like in the case of timber pile foundations, more advanced and elaborated work might be needed when assessing structural safety. A general framework for the assessment of existing structures is laid down in ISO Standard ISO 13822 – Bases for design of structures — Assessment of existing structures. In this standard, explicit attention is given to the differences in designing new structures and assessing existing structures. More specifically, the following statements are given:

- The continued use of existing structures is of great importance because the built environment is a huge economic and political asset, growing larger every year.

- The assessment of existing structures is now a major engineering task. The structural engineer is increasingly called upon to devise ways for extending the life of structures whilst observing tight cost constraints.
- The establishment of principles for the assessment of existing structures is required because it is based on an approach that is substantially different from design of new structures and requires knowledge beyond the scope of design codes.

As ageing and remaining service life are closely related, an assessment of an existing timber structure might involve the analysis of many different factors. On the one hand, the current status of the structure and its age must be addressed, which includes an assessment of the materials of which the structure is made with respect to strength and stiffness, and their geometry. On the other hand, the structural safety and reliability must be addressed, which involves an analysis of historical and envisaged future load conditions, as the structures' functions may change, but also because modern society has different (generally more demanding) safety and reliability requirements.

On the basis of these standards and considerations, analysis of an existing timber structure requires a number of steps. An important component is the analysis of the current state of the material, as wood may suffer for instance from (biological) decay or drying cracks as is often found in foundations and historic timber structures (Van de Kuilen and Gard, 2012). In order to be able to make decisions on possible structural interventions, damage models for timber structures that take the load history into account allow for service life calculations on partly deteriorated structures. However, as often these kinds of assessments need to be performed in accordance to current design rules and code regulations, an introduction to current code rules for the design of timber structures and their background is therefore given.

LONG TERM STRENGTH IN STRUCTURAL DESIGN CALCULATION OF TIMBER

The long-term strength is related to creep, in the sense that elements that have been deforming under long term loading, may reach a state that the creep rate starts to increase, leading to structural failure. The time span from the beginning of loading until the failure is called the 'time to failure' (T_f). This time to failure is related to the load level, where the load level is defined as the ratio between the applied stress and the timber strength, whereas this strength is generally taken as the mean short-term strength. The latter is the strength as found in standardized tests for the determination of wood properties. For the timber piles in Amsterdam, the current method of strength verification is based on full scale tests on sections of timber piles. For the piles in Amsterdam, the characteristic value of the short-term compression strength has been derived on the basis of a limited set of data on new piles. All piles were tested in water-submerged condition and only minor differences between spruce, larch or Douglas fir were noticed, so for practical reasons no distinction between wood species was made. However, for reasons of consistency of use in structural design codes, the result has been modified and published as a *dry characteristic compression strength value,* namely 19.8 MPa.

Equations relating the load level to the time to failure are generally straight lines on a logarithmic time-scale:

$$\frac{\sigma(t)}{f_s} = C_1 - C_2 logT_f \tag{1}$$

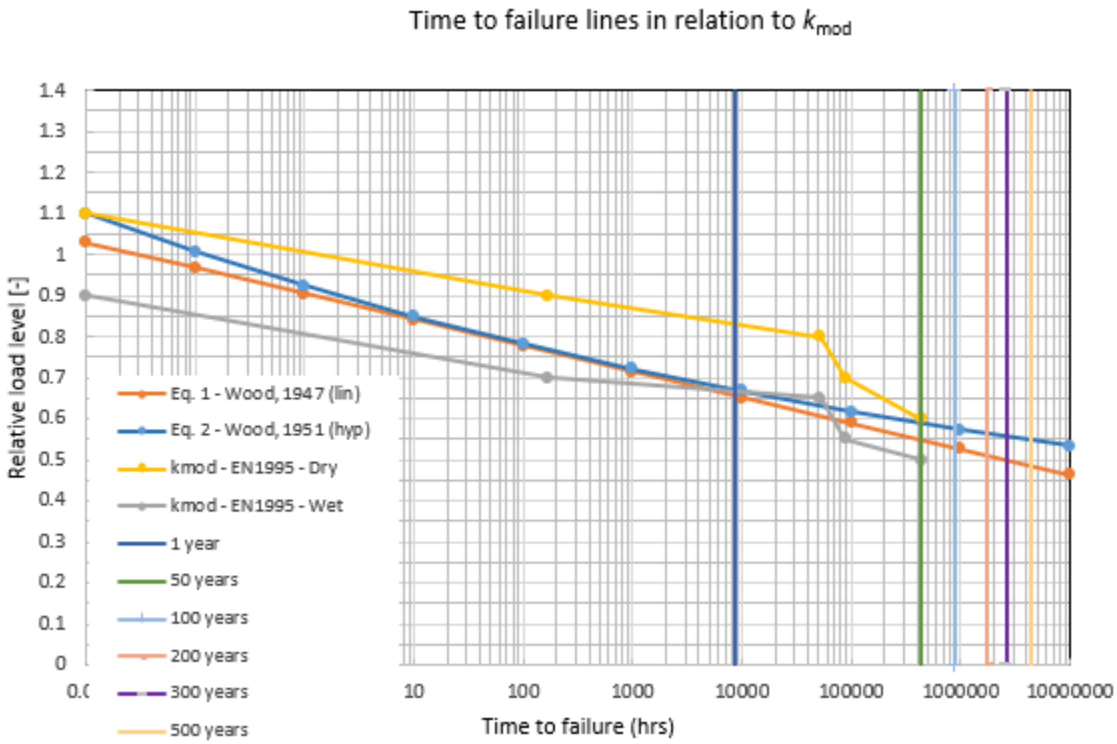
in which:
$\sigma(t)$= the stress in time, generally taken constant in a time to failure test
f_s = the short term strength of the material
T_f = the time to failure
C_1, C_2 = constants obtained from a regression analysis of test data

Values for C_1, C_2 can be found in literature. With respect to time to failure, sometimes reference is made to what is called the 'Madison equation,' reading:

$$\frac{\sigma(t)}{f_s} = 0.7416 T_f^{-0.4635} + 0.183 \qquad (2)$$

This is a hyperbolic curve, but leading to a similar result with respect to the time of failure in relation to the applied load level. A major difference is that this equation predicts that a stress level below 0.183 will lead to an infinite time to failure, far beyond the life for which normal timber structures are designed.

These two equations, together with the k_{mod} value as given in EN 1995, are graphically shown in Fig. 1



From the graph, it can be observed that the long-term strength values (k_{mod}), as given in EC5, coincide reasonably well with the derived time-to-failure lines by Wood (1947, 1951). However, the reference period in EC5 is restricted to 50 years (permanent loading), whereas foundation piles in Amsterdam have been in service for up to around 400 years. Due to the logarithmic nature of the time to failure process, the relative load level is quite sensitive for a given required period of service life. As an example, for a wet condition and a reference period of 50 years, EN 1995 specifies a value of 0.5. At this 0.5 level, however, the theoretical prediction of the mean time to failure based on the regression equation of Wood (1947), gives an equivalent estimated mean time to failure (service life) of about 300 years. A further point of attention is the fact that long term strength factors in design codes are derived on the basis of bending tests, whereas the prime loading case for piles in foundations is compression, or compression and bending. Shear stresses as a result of skin friction are considered negligible.

For pile foundations, but also old structures in general that need to be assessed for structural safety, this shows that it is very important to have an understanding of the ratio (the load level) between historical and actual permanent loads and material strength respectively. As can be seen from Fig. 1, if the load level is around 0.5 (or 50%), the expected failure time is around 50 years, but if the load level is about 0.4 (40%), the expected failure time by extrapolation would be more than 1,100 years. Consequently, for pile foundations, a good and accurate estimate of the load level, its history and the material state, is essential. In Van de Kuilen (1999), a more comprehensive overview of different time-to-failure lines is reported, where also the moisture content influence has been studied.

The use of this long-term strength phenomenon in the verification of the strength of timber is regulated in design standards such as NEN-EN 1995 - Eurocode 5 Design of Timber structures, through the factor k_{mod}, relating the characteristic strength to the design strength:

$$\sigma_{c,0,d} \leq k_{mod} \frac{f_{c,0,k}}{\gamma_M} \tag{3}$$

This equation can be used when structures are designed with new wood. It can be rewritten as:

$$\gamma_M \frac{\sigma_{c,0,d}}{f_{c,0,k}} \leq k_{mod} \tag{4}$$

This equation is equal to equation Eq. 1, and consequently k_{mod} in Eq. 4 can be seen as a representative value for the time-to-failure equation, apart from any safety factors. For applications such as timber piles that are saturated the k_{mod} value relates to Service Class 3 in Eurocode 5 (wet conditions).

In EN 1995, tables (see Table 1) are specified with values for k_{mod} as a function of the load duration and the service class. Each load acting on a structure has to be assigned to a 'load duration class.' Foundations are generally considered to be loaded by permanent loads, for which EC5 specifies load duration class permanent (self weight), when the accumulated duration of the characteristic load exceeds 10 years. Added to the self weight is a small component of the variable load that is considered to be permanent as well.

Table 1. Load-duration classes according to Eurocode 5 Design of Timber Structures and loading examples

Load-duration class	Order of accumulated duration of characteristic load	Examples of loading
Permanent	more than 10 years	self-weight
Long-term	6 months – 10 years	storage
Medium-term	1 week – 6 months	imposed floor load, snow
Short-term	less than one week	snow, wind
Instantaneous		wind, accidental load

As the reference period for structural safety levels is generally taken as 50 years, pile foundations aged 100 or more years clearly fall outside of this generally accepted time range. This means that for existing structures, when analysed for structural safety and remaining service life, the standard approaches as given in design codes have to be handled with care, and sometimes are not even applicable.

MODELLING APPROACH OF THE LONG-TERM STRENGTH USING A DAMAGE ACCUMULATION CONCEPT

The time dependent behavior of the material can be described by a system of serial and parallel energy barriers, where the overall rate of deformation is determined as a weighted mean of the deformation rate of the individual barriers. The resulting mathematical descriptions of this approach can be used not only to predict the time to failure (Eq. 2), but also the development of the strength in time, until this time to failure is reached. Models that describe the strength development over time are called damage accumulation models, and are in use for a variety of applications, including fatigue of bridges, airplanes and wind turbines.

As an example, a straightforward damage model is presented here, based on the theory of reaction kinetics (Caulfield, 1985), (Kuipers, 1986), (Van der Put, 1986). It explains damage accumulation models from Gerhards (1979) and Foschi and Yao (1986), the latter two are also referred to in the literature as the 'American' and 'Canadian' damage model respectively.

The principle of a damage model works as follows. A damage parameter α is defined: α is equal to 0 when the structure is new, there is no damage yet. Over time when the structure is subjected to a mechanical load, α grows. As soon as α reaches 1, the structure has failed and consequently the time *t* has reached the time to failure T_f. All the loads that are applied on a structural member lead to damage, and thus a (slowly) growing value α. The higher and longer duration of the load, the higher the α per unit of time. A simple and understandable approach is used as an example (Gerhards, 1979). It assumes that there is a linear exponential damage accumulation equation which gives the time to failure when integrated for α from 0 to 1 and *t* from $t = 0$ (new structure) to $t = T_f$ (structure has failed), and reads as follows:

$$\frac{d\alpha}{dt} = \exp\left(-C_1 + C_2 \frac{\sigma(t)}{f_s}\right) \tag{5}$$

Consequently, integration over the time-load history gives:

$$\int_0^{\alpha} \alpha = \int_0^{T_f} \exp\left(-C_1 + C_2 \frac{\sigma(t)}{f_s}\right) dt \tag{6}$$

If the load is specified as block loads σ in time periods Δt, it is possible to add up the individual σ-Δt contributions to the damage α as $\sigma(t)/f_s$ = constant:

$$\sum_{\alpha_i}^{\alpha_{i+1}} \alpha = \sum_{t_i}^{t_{i+1}} \exp\left(-C_1 + C_2 \frac{\sigma(t)}{f_s}\right) \Delta t \tag{7}$$

Alternatively, a more advanced time-integration has to be performed (using numerical analysis tools):

$$[\alpha]_0^1 = \int_0^t \exp\left(-C_1 + C_2 \frac{\sigma(t)}{f_s}\right) dt \tag{8}$$

For a constant stress as applied in most creep tests until failure: $\sigma(t) = constant, so\ \frac{\sigma(t)}{f_s} = constant.$

This leads to:

$$\alpha = t \cdot \exp\left(-C_1 + C_2 \frac{\sigma}{f_s}\right) \tag{9}$$

As defined for the failure: $when\ \alpha = 1\ then\ t = T_f$, so:

$$\alpha = t \cdot \exp\left(-C_1 + C_2 \frac{\sigma}{f_s}\right) \Rightarrow 1 = T_f \cdot \exp\left(-C_1 + C_2 \frac{\sigma}{f_s}\right)$$

This can be rewritten as:

$\frac{1}{T_f} = \exp\left(-C_1 + C_2 \frac{\sigma}{f_s}\right) \Rightarrow -lnT_f = -C_1 + C_2 \frac{\sigma}{f_s}$ and consequently:

$$\frac{\sigma}{f_s} = C_1' - C_2' lnT_f \tag{10}$$

in which σ(t) is the stress over time and f_s the strength.

Fitting of the equation to test data where the long-term strength of members has been measured gives the values of the constants. The equation derived by L.W. Wood in 1947 reads:

$$\frac{\sigma}{f_s} = 0.904 - 0.063 log T_f \tag{11}$$

with T_f in hours. The original publication for the long-term strength effects was published as Wood, L.W. Behaviour of wood under continued loading, Eng. News-record 139(24):108-111. (1947, 1951). This equation is derived for mean bending strength values and a constant load. In practice, a constant load does not normally occur, but modelling the time variant loading on a real structure as a time sequence of constant loads with varying amplitude and time gives similar results. A fixed value for the short-term strength f_s may be assumed, but a strength value can also be selected out of the known statistical distribution of the compression strength of (sound) piles. As a consequence, the important item in the traditional damage accumulation models is the ratio between applied load and short-term strength. The latter can be estimated from short term tests and in case of structures where the prime load component is dead weight, also the stress level can be estimated with sufficient accuracy, to be able to perform sensitivity analysis and failure studies.

Important to note is that from equation 5, it follows that damage accumulation is exponential to the load level. This means that low load levels contribute relatively little to the damage α per unit of time Δt. However, the ratio of stress σ over strength f_s contributes more than linearly to the damage development if that ratio increases. Thus, a higher load over a certain time span contributes to more damage than a lower load over a similar time span and this is exponential. In Fig. 2, the concept of strength development over time is shown for a relative load level of 0.5, showing hardly any load carrying capacity will be left after 200 years, see also Fig. 1 for the time to failure at a relative load level of 50%.

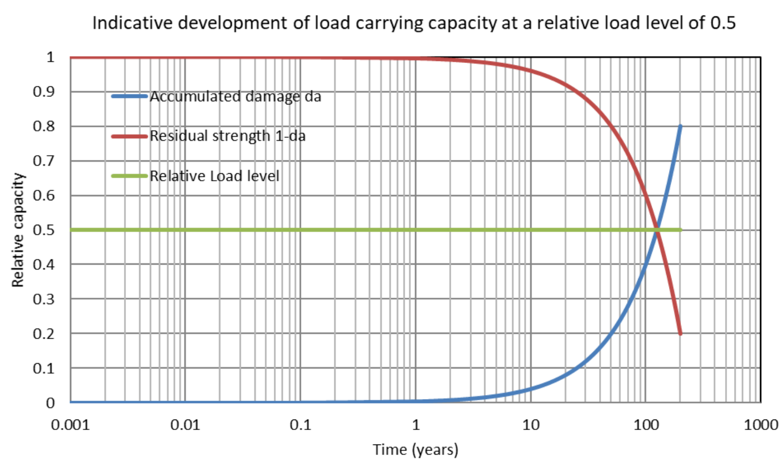


Fig. 2. The concept of strength development over time for a relative load level of 0.5 using a linear damage accumulation model.

For the damage model solution, a Ferry-Borges Castanheta model (FBC) load model is used, as a square wave load is considered to be sufficient for pile foundation loading and it gives a direct solution of the differential equation in terms of damage per unit of time.

INFLUENCE OF BIOLOGICAL DECAY ON DAMAGE DEVELOPMENT

Previously, the long-term strength assessment has been presented without any effects of biological degradation. In the case of timber piles in Amsterdam, however, even though the load may remain almost constant over longer periods, the biological degradation of the wood will lead to a stress redistribution within the cross section, so a higher stress ratio on the sound parts as the strength decreases over time with relatively constant load. The (slow) continuous increase of biological degradation influences the expected service life negatively. Due to the biological decay, assumingly degrading piles from the outer layers to the inside over time, the material properties of the wood will be affected. The outer layers will change its mechanical properties, and consequently there will be a redistribution of stresses from the degraded outer layers to the non-degraded inner layers.

An example is given in Fig. 3, where the cross-sectional variation in compression strength was determined in an old recovered pile from a foundation (Van de Kuilen, 2007).

The reduced mechanical properties on the outside of the pile will lead to an increase in stress on the remaining sound cross section, see Fig. 4. In Fig. 4, the upper part of a partly decayed timber pile has been CT-scanned, showing the reduced density of the outside (light color), whereas the inner part (dark color) is still sound. Whether the outer layers lose their capacity completely is still part of ongoing research. A safe approach would be that degraded layers have zero capacity, and consequently the design load would have to rely on the sound inner part of the pile. In either case, it will lead to an increased load ratio on that part, and consequently the time to failure as indicated from equation 11 and Fig. 2 would be reduced. Establishing how much this reduction is, is part of ongoing research. Literature on remaining strength after biological deterioration is limited, especially as biological deterioration studies generally focus on the relation between fungal decay and mass loss. Mass loss is extremely difficult to relate to strength loss as the mass loss is generally not evenly distributed over the volume and consequently understanding strength values based on prismatic cross sections is challenging.

ASSESSMENT OF DEGRADATION AND WOOD QUALITY

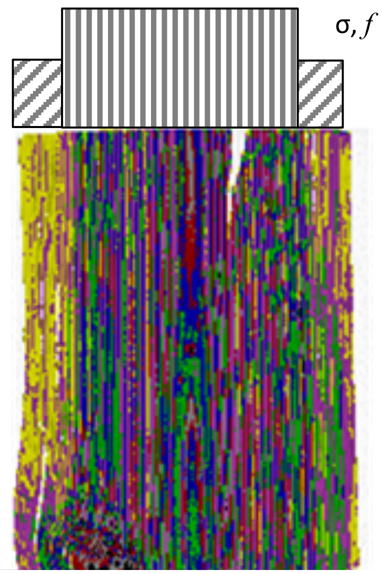


Fig. 4. Indicative idealized stress and strength distribution over a cross section of pile with decayed outer ring (Van de Kuilen et al., 2023)

With regard to piles and the measurements needed to assess their current state, a number of issues need to be addressed. It has to be realized that a timber pile is basically an upside down tree. This means that the material properties within the pile depend on a number of factors that are generally referred to as 'wood quality' and vary in both radial and longitudinal direction. In radial direction, juvenile wood, heartwood and sapwood have to be identified. In the longitudinal direction the ratio between sapwood and heartwood will increase as heartwood formation will take place only 10–15 years after wood formation. In addition, the juvenile wood is characterized by a lower compression strength parallel to grain, whereas the compression strength at the tip is considered to be about 10–20% lower than at the pile head. Furthermore, the compression strength of piles is generally governed by the knottiness and the whorls in particular, which is slightly different for spruce and pine respectively. In the case of bacterial decay of saturated piles, the extension and rate of decay in radial direction remains a major point of attention. It is not clear whether there is a delay time of the onset of decay after installation for sap- or heartwood respectively (Van de Kuilen, 2007), as well as the rate of destruction of the wood cells (Varossieau, 1949) and consequently strength loss. CT-scan images from decayed piles seem to indicate that the bacterial decay is primarily related to the sapwood, see Fig. 5. (Schreurs, 2017) but a lower decay rate and continuous decay process cannot be excluded.

In order to assess this decay in practice, the underwater microdrilling method can be applied (Ravenshorst et al. 2024). The resistance profile as measured with the microdrill, allows for an assessment of the decayed part within a cross section, see Fig. 6. The strength profile as measured in Fig. 6, is based on a similar analysis of the cross section as shown in Fig. 3. However, the microdrill output can be directly related to the expected compression strength as a function over the cross section. It can actually be seen within Fig. 6 that when the microdrilling signal is low and thus indicating decay, the decayed material still has a compression strength of around 4–8 N/mm^2 compared to a mean value of around 13 MPa for the centre part of the pile. The lower compression strength of the juvenile wood in the centre of the pile is also clearly visible in this case.

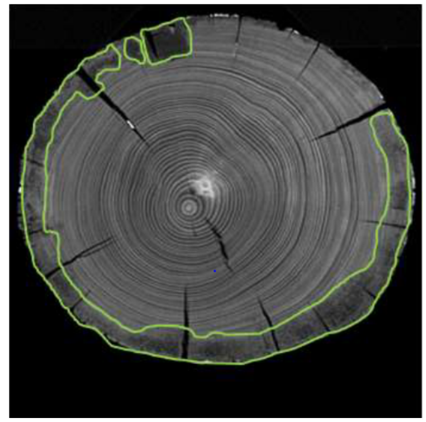


Fig. 5. Decay identification based on CT-scanning of a pile segment (Schreurs, 2017)

Consequently, measurement results like this can be used as input for the damage equation (eq. 5), which can be plotted in a time dependent load carrying capacity form as shown in Fig. 2. The stress ratio in equation 5 can be written in terms of load ratio F (kN) and load carrying capacity F_u as follows:

$$F_u = f_{c,0}A_{rem}(t) + f_{c,0,dec}A_{deg}(t) \qquad (12)$$

using the measured values for sound wood compression strength $f_{c,0}$ and decayed wood $f_{c,0,dec}$ compression strength respectively. Time integration with an estimated historical load on bridges (primarily dead load) will consequently give the estimated damage state in a similar way as indicated in Fig. 2. The following conditions have been used as input for hypothetical calculation of a sudden decay after 30 years. The output is shown in Fig. 7. The following data is used in this example. The load level as indicated by the community of Amsterdam is taken as 0.3. This is the estimated ratio between the dead weight of the upper structure of bridges in Amsterdam and the load carrying capacity of an equivalent new pile foundation. Further, a delay time of 30 years for degradation initiation is taken, covering on the one hand a possible delay time for heartwood degradation initiation as found in Van de Kuilen (2007), and on the other hand to account for the various stages of decay progress as indicated by Varossieau (1949), leading to a certain level of decayed wood. This is also related to the ratio b between the strength of decayed wood and non-decayed wood, which is taken as 0.4, on the basis of Fig. 6. The outer decayed ring has a remaining compression strength between 4–8 MPa, whereas the inner sound part of the pile has a compression strength between 9–16 MPa). The cross section d is reduced to an

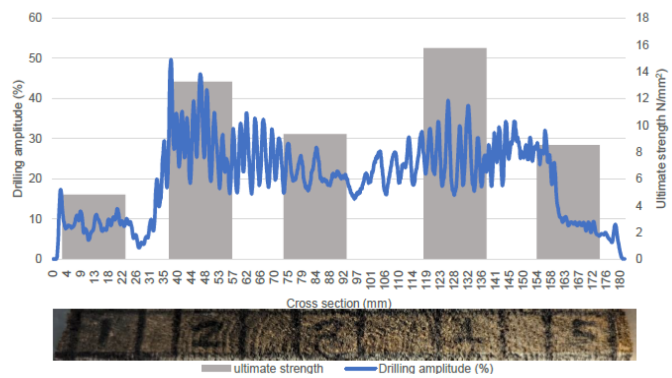


Fig. 6. Microdrilling profile and compression strength of cut-out small specimens

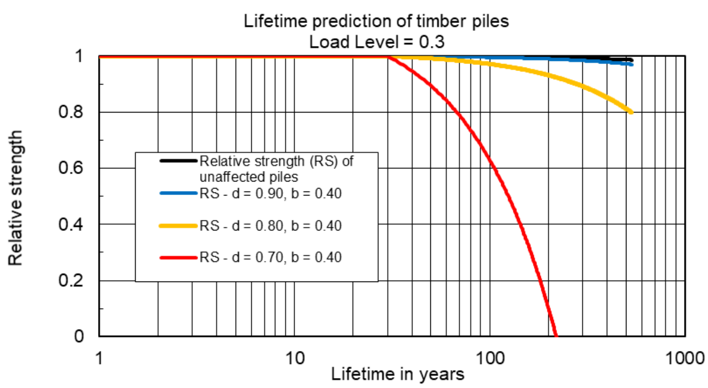


Fig. 7. Example of a lifetime simulation based on selected decay depths and non-zero strength of decayed wood.

exemplary value of 0.9, 0.8 and 0.7 respectively. No further time aspects have been taken into account for this example. By varying values for *b* and *d*, scenario and sensitivity studies can be performed with respect to the current state, as well as for the remaining service life as a function of the load. This allows for evaluations for foundation re-use as well.

OUTLOOK AND FUTURE WORK

The derived relative capacity model allows for extensive sensitivity analysis with regard to the current state and future service life estimation of a wooden pile foundation. Validation and application of the model requires an assessment of the historical loads in relation to the observed compression strength of recovered or in-service piles, as well as an estimate on how the biological degradation has progressed over the cross section over time. The fundamental concepts and theoretical model presented in this paper represent a viable approach to not only evaluate the current load carrying capacity, but to also evaluate the remaining service life as a function of future loading conditions in conjunction with progressing decay.

Not surprisingly, many parameters of pile foundations as well as the wood decay processes are still not well understood and require further attention. For instance, the aspect of skin friction along tapered piles as well as the transfer of skin friction shear stresses through a decayed wood layer with reduced shear rigidity should be further evaluated.

In addition, the biological degradation will progress with respect to time. However, there is uncertainty whether the biological degradation starts from day 1 when a pile is installed or whether there is a delay time to be accounted for. Such a delay time is expected for heartwood based on results in Van de Kuilen (2007) but scatter is high. Similarly, it is unclear whether the decay continues to progress at the same rate after the decay front has reached the boundary between sapwood and heartwood. Furthermore, pine has a different cell structure than spruce or larch and it is expected that the decay rate in pine sapwood is higher. The rate of progressing decay is probably most different for sap- and heartwood in all species, but currently there is still a considerable lack of data.

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