STRUCTURAL ASSESSMENT OF EXISTING TIMBER ROOF STRUCTURES FOR GREEN ROOFS IN ROTTERDAM

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City of Rotterdam

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STRUCTURAL ASSESSMENT OF EXISTING TIMBER ROOF STRUCTURES FOR GREEN ROOFS IN ROTTERDAM

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A Thesis Submitted in Partial Fulfillment

of the Requirements for

the Degree of Master of Science in Civil Engineering

Faculty of Civil Engineering and Geosciences

Delft University of Technology

November 2015

TITLE PAGE

Title: Structural assessment of existing timber roof structures for green roofs in Rotterdam

Description: Master thesis for the graduation work CIE5060-09.

Academy: Delft University of Technology
Faculty: Civil Engineering and Geosciences
Master Track: Structural Engineering
Specialization: Steel and Timber Construction

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State: Final

Date: November 2015

PREFACE

In the last phase of the study program 'Structural Engineering' the writing of a Master thesis is obligatory and will finalize the Master. During the search for a suitable subject I came in touch with Kees Blom, Senior Consultant Civil Engineering of the Engineering department of the Municipality of Rotterdam, who encountered different problems related to timber. One of these problems was the making of green roofs on existing timber roof structures. Rotterdam has ambitious plans for becoming greener and I enjoyed to play a role in this program. My choice for this subject is based on the upcoming demand for green roofs and the wide variety of constructive and timber aspects that are related to it.

Therefore I would like to thank Kees Blom for giving me this opportunity, providing a workspace and for sharing his knowledge over several drinks. Also thanks to his colleagues at the municipality and the city archive who did not hesitate to advise me on different matters.

I am grateful to the demolition company Struijk who provided me with freshly demolished roof beams from different time periods. Because of their contribution I was able to demonstrate and verify theoretical ideas with practical work. This would also not have been possible without the assistance of the employees of the Stevin Laboratory, for which my thanks.

While writing this thesis, it was necessary to have good monitoring and steering. For this I would like to thank Geert Ravenshorst and Wolfgang Gard whose door was always open for counseling and quick chats.
Thanks to the chairman of the committee Jan-Willem van de Kuilen for his enthusiasm, support and valuable tips on timber related problems.

At last I am grateful to my family and friends who supported me during the writing of this thesis and whose assistance played an important role in completing my study.

Delft, November 2015
Lars Rovers

SUMMARY

Green roofs, also known as vegetation roofs, are becoming very popular for residents of houses or apartments and for good reasons. Vegetation on rooftops have many benefits on global and local level. Expectations show that the climate change will lead to heavier storms and causes the sewers in Rotterdam to overflow. Hence, the municipality of Rotterdam wishes to apply this special type of roof on a large scale to buffer rainwater which can gradually be discharged. Although the concept of a green roof is nothing new, applying them on this large scale to solve an urban problem makes it an interesting topic.

Two types can be distinguished: intensive and extensive green roofs. Both types are able to retain rainwater but can be distinguished in their function. Intensive green roofs allow for recreation and gardening while extensive green roofs have an aesthetical function. The municipality of Rotterdam and its citizens both want green roofs instead of bitumen roofs, however they neither have the time nor the knowledge to determine whether their timber roof structure is suitable for this extra ballast. A first simplified calculation indicated that there was not enough strength to resists this load. If no research is done then Rotterdam will only be able to have vegetation on structures of steel or concrete while timber roofs are commonly present. In an ideal future every roof in Rotterdam is a green roof. This thesis researches the residual capacity of flat timber roofs by reducing the uncertainties associated with the strength. The main goal is therefore to be able to predict, and if necessary increase, the true strength capacity without demolishing the roof structure.

The past

The first step is to identify the size of the problem. A multi-criteria analysis was already performed by the municipality and translated into a potential map. However the criterion “year of construction” has a high level of uncertainty because there is a lack of knowledge about older timber roof structures. The map distinguished five groups with different ranges of construction years which was based on experience. A logical step is an archival research to the history and typologies of houses in Rotterdam. This investigation revealed that most houses were built before 1940 (pre-war) but a large amount of roof structures are renovated or renewed in the 80’s. Timber was the main building material for roof structures but after World War II the focus was on speed and efficiency which resulted into more concrete roofs. Another consequence of World War II was the destruction of the city archive. This resulted into the loss of information about the present timber properties and dimensions.

The structural geometry of flat roofs did not change over the years. During archival and literature research it was found that a beam supported by two masonry walls is standard practice. The main consequence of a green roof is than an increase of the bending moment of the existing timber elements. This increase may lead to collapse of the roof structure. Before extensive research towards the true strength of timber elements was performed, a more general investigation to gaining strength and possible weak spots was done. The idea was that some design norms throughout the years might have used too conservative values and thus strength could be gained by recalculating the structure with the current regulations. It was found that the values for roof structures stayed practically the same in the building codes. However most structures also satisfy the deflection requirement but this is not legally established. Beams that are designed on this requirement have extra strength. The visual grading norms, which determines the strength of a timber element based on visual characteristic, have become more flexible over the years. Also the strength classes changed, before 1933 no strength value was used but dimensions were based on experience. Afterwards two main strength classes were defined as standard building wood and construction wood which are more or less equal to C18 and C24 in modern times.

Wood is an organic material which is sensitive to time dependent processes that reduce the strength. Age is not necessarily a strength-reducing factor but is associated with strength-reducing processes. Four degradation

mechanisms can be distinguished for timber: mechanical, physical, chemical and biological. The latter is the largest problem for roofs because insulation or treating the wood was not always done (correctly). Table 2-5 gives an overview of positive and negative aspects for roof structures.

The present

Current approaches are based on identifying the state of the structure and calculating the extra load according to the active regulations for new buildings. The building code refers to the NEN8700 for coping with existing structures. This norm gives five solutions when the strength is not sufficient: reduce the reference period, values are based on actual use, adjust the use, adjust the safety margin or adjust the strength. This thesis focused on the first and last option. Reducing the reference period is discussed and not recommended unless the engineer can determine and control the load with high precision. It is not legally determined if the change of the safety level is allowed. This will lead to discussions with "construction and housing inspection" in the future because roofs are less safe. The extra capacity must thus be found in adjusting the strength.

The idea is as follows, freshly sawn structural timber is graded into a strength class. This means that a small amount of the graded timber does not have to meet a certain limit strength. Nowadays the 5% lower probability value is chosen as the limit value. This way of strength grading allows for beams to be stronger than the characteristic value. The experiments were aiming to predict the actual strength without demolishing the roof. During the research, thirteen beams were obtained from an ongoing demolishment. Ten of these members are of a renovated roof structure from 1983 while the original structure was from 1923. The other three members are from another building where the original structure of 1923 was still present. A strength prediction model was used that required the density and dynamic modulus of elasticity (MOE). The density can be measured with aid of a resistograph. Here it is important to drill in radial direction. Next a vibration meter was used to measure the wave speed which can be combined with the density to gain the dynamic MOE. Five sub-experiments were conducted to determine the difference between in-situ situations and free vibrations. As it turns out, a screw is the best way to introduce the wave and the surrounding increase the wave speed. This needs to be corrected with a certain coefficient on the frequency. A 6 to 10 percent increase of the frequency was found during testing. At last the true bending strength was checked with a four point bending test. The true strength was 20 to 140 percent higher than the characteristic value of the initial grade.

Four strategies for future assessments are proposed: calculate as new structure according the current regulations, reduce the reference period (not recommended), visual upgrading and non-destructive tests. Each step requires more work but will, most likely, lead to extra strength. Two case studies were worked out following the different strategies. As expected, non-destructive tests is the most beneficial strategy because information about the actual strength is attained. With this information the strength class could be upgraded. In this case study it becomes clear that low weight green roofs (1 kN/m²) can be applied while a heavy green roof (3,4 kN/m²) needs more attention. A solution between these two extreme is also possible. Furthermore in one case the characteristic bending strength was increased with a factor of 1,5. Time dependent factors seem to be the main problem in all strategies. The duration of load may cause excessive deflections or even creep rupture. Limits to the deflections are not legally established and can be concealed with a lowered ceiling.

The future

The choice for a method of reinforcing an existing structure depends on a number of criteria. An engineer and user should discuss the possibilities that satisfies both. The most optimal solution will depend on the existing timber structure because every situation is unique. For the case studies the best solution is to increase the cross section of specific individual beams with timber elements. This method is easy, fast and cheap.

At last an action plan was made for future assessments. Following the different steps in this protocol can make reinforcement and extra costs unnecessary and is thus the first step towards a Rotterdam with only green roofs.

TABLE OF CONTENTS

1. INTRODUCTION: THE INTEREST OF ROTTERDAM IN GREEN ROOFS

A recent article from (Rijksinsituut voor Volksgezondheid en Milieu, 2014) *shows that the area of Rotterdam has the most polluted air of the Netherlands, mostly caused by traffic and industry. Nowadays Rotterdam wants to become greener by addressing the cause and consequences of climate change. Furthermore the water systems have (almost) reached their capacity due to the increasing intensity of rainfall. Mainly in the city center flooding occurs due to a large amount of rain. There is already an underground water reserve in Museumpark to store and improve the quality of water but this measure is not sufficient in the future. To efficiently tackle the flooding event a solution must be found in making existing structures multifunctional* (Bes & Goedbloed, 2011).

One way of doing this is by making a commonly bitumen roof a green roof (sometimes called a vegetation roof or living roof). The dictionary (Dictionary.com, 2015) gives the following definition to a green roof: "a roof covered with vegetation". These can be used for growing crops or recreation and is more aesthetically pleasing. The municipality of Rotterdam sees a lot of potential in these sort of roofs because of the following effects.

Effect on city

As cities grow, more and more soil is getting covered by an impermeable layer to create room for buildings and infrastructure. The consequence is that rainwater cannot penetrate into the soil and needs to be transported by sewers. These sewer pipes were designed for a lower flow and will now overflow during a heavy storm. A green roof can retain some precipitation that falls on rooftops. This will slow down the water transport and decreases the discharge of the sewers. A study of the KU Leuven (Mentens, Raes, & Hermy, 2005) showed that green roofs help reduce the urban runoff problem but cannot solve it on its own. Furthermore with more green, the city improves the air quality, increases the biodiversity and it reduces the urban heat island effect.

Effect on individual

By taking different measures, an existing building can become more sustainable. Nowadays popular solutions are grey water circuits and solar panels while municipalities are starting to attract more attention towards green roofs. This increase of interest in green roofs is for a good reason. The extra layer causes for a better insulation during cold and warm season which reduces the energy costs. Moreover it increases the soundproofing and it acts as a protective layer for the roof covering against weather conditions.

Opportunities of Rotterdam

A study commissioned by the municipality (Bes & Goedbloed, 2011) shows that a large number of buildings (76% of total roof surface in Rotterdam compared to 15% in Amsterdam) have a flat or mild sloped roof, especially in the city center there is a lot of potential. This is caused by the bombing in 1940, where a high percentage of the inner city was destroyed or harmed. Due to the damage of fires many buildings were also demolished afterwards. Numerous citizens became homeless and thus the need for housings was high. Four days after the bombing the beginning of a reconstruction plan started. The idea was to completely renew the city center including the separation of functions and thus the houses were planned in the suburbs while offices and stores were mainly in the center (Gemeente Rotterdam, 2015). During the reconstruction the flat roof landscape arose.

In 2008 the municipality of Rotterdam started the so called Program Sustainable Roofs to stimulate green roofs. A green roof is one of the innovative solutions to temporary store the increasing rainwater. Furthermore municipality Rotterdam wants to create more urban green, new social meeting points and green energy. Citizens are now encouraged by the government to get greener by giving subsidies while businesses can make use of green deals. Since July 2008 there is a subsidy of €25,- per m^2 for private individuals. This arrangement became a big success which led to a shortage of the reserved money. Housing associations and corporations can make use of a subsidy of 50% of the total cost with a maximum of €25,- per m^2. Their goal was to realize 160.000 m^2 of green roofs in 2014, at the end of this year they had realized 200.000 m^2. Especially the municipal properties are equipped with green. At the end of 2025 the program desires 600.000 m² of green roofs and 50% of municipal property needs to be covered with vegetation (Bes & Goedbloed, 2011).

A market research concluded that most property owners are interested in green roofs after they were told about the pros and cons. The result is visible in figure 1-1.

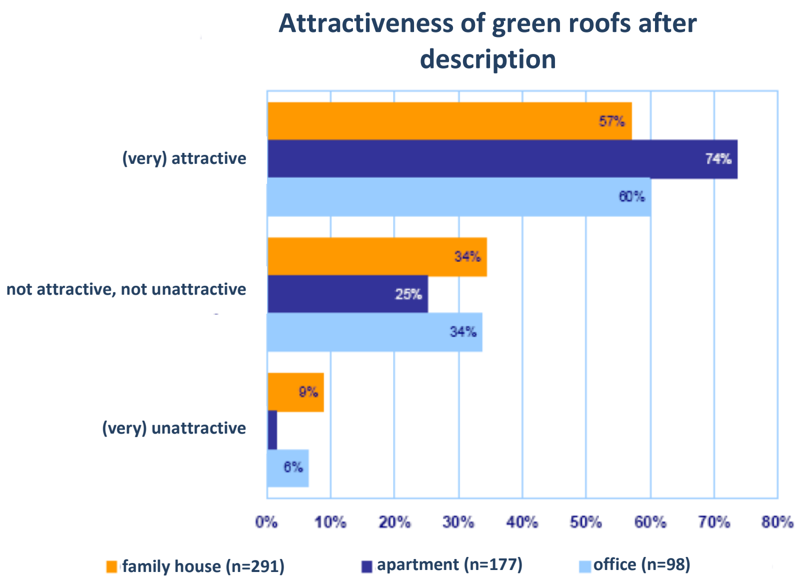


Figure 1-1: Interest in green roofs after hearing description (Bes & Goedbloed, 2011)

Citizens are enthusiastic about green roofs, only a small percentage is not attracted at all. Furthermore it was concluded that there is no difference in interest between various districts which can imply that citizens with different social and financial status are equally interested.

Because of its rising popularity regulations for commercial use are needed to control the designing aspects. The NEN-normcommision for green roofs was founded in 2012 to make sure the green roofs have sufficient performance, functionality and satisfy the testing method for vegetation on structures. However the only constructive aspect that is included is the wind load.

1.1 PROBLEM DESCRIPTION

Making a green roof does not necessarily means that the existing roof will be demolished. A standard green roof can be built upon the existing roof construction and consists of the following layers (from bottom to top, see figure 1-2): roof structure – a waterproof and root resistant cover – a protection layer – drainage layer – filter layer – substrate layer – vegetation layer.

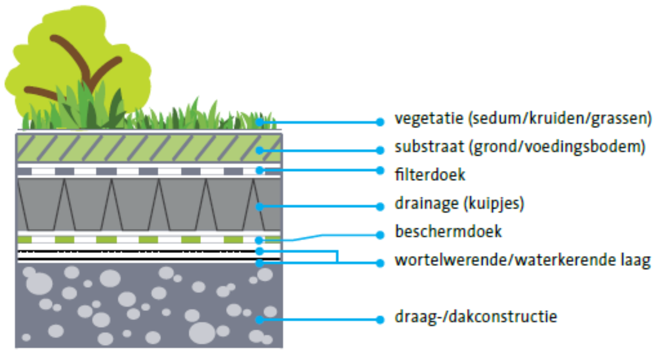


Figure 1-2: Cross section of standard green roof (Gemeente Rotterdam, 2009)

The drainage and substrate layer determine the amount of water that can be stored in a green roof. This water is then gradually drained and a part is vaporized. Current green roofs in Rotterdam can contain about 15 L/m². However the new standard is becoming 25 L/m². From water boards perspectives it is interesting when the roof can store 50 L/m² but the municipality has not yet made a decisive decision whether they want to aim for this amount as well. More buffered water will unburden the sewers even more but it also requires a stronger roof structure.

Two types can be distinguished: intensive and extensive green roofs. Intensive green roofs can be compared to an average garden because of the maintenance needed. The vegetation can consists of grass, herbs, bushes and even trees. This is sometimes combined with a roof terrace and a pond. An extensive green roof has low maintenance and consists of grass, herbs or plants. Here a slope of maximum 45° is possible. The choice between the types is often based on the maximum resistance of the roof structure. The extra load of an extensive green roof is most likely to be 20 – 200 kg/m^2 while intensive can be 300 – 1500 kg/m^2. In the latter case the roof structure usually needs to be strengthened. The municipality has no preference as long as it can buffer 25L/m² but they are interested in the possibilities for the future. Also the interest of property owners is made clear by means of a poll which asks their preference between the two types. The results are visible in figure 1-3.

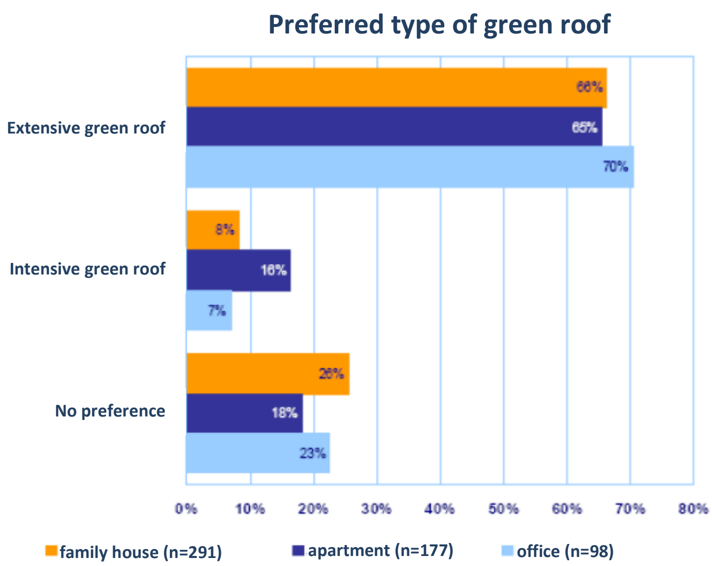


Figure 1-3: Preferred type of green roof (Bes & Goedbloed, 2011)

Extensive green roofs are the most popular. The reason might be that the maintenance is low. Also owners of an apartment are interested in intensive green roofs because they often do not have a garden.

A combination between these two types is possible or they can be combined with solar panels (Yellow roof), a water storage (Blue roof) or fulfill a social function (Red roof). These are not part of the subsidy program and are in experimental phase. A variant of the blue roof, called a Polder roof, stores the water for private use and can be discharged by a sluice before the next heavy rainfall. A multi-criteria analyses from the municipality shows which buildings have the highest potential for green roofs along with the level of uncertainty. The criteria in the analysis are ownership (medium uncertainty), slope (low), year of construction (high), water policy (low) and inside/outside dike area (low). The results are shown in so called potential maps, which are created by the municipality. The criteria "year of construction" has a high level of uncertainty and thus it might be over- or underestimated since it is not based on actual constructive roof aspects but more on the condition of the foundation and previous experience.

These different roof systems can become a critical load for an existing timber structure which was not designed for this ballast. A simplified calculation with assumptions was performed to show that the timber roof could fail under certain conditions (see appendix A). This is because it is uncertain what timber strength is present, what kind of roofs were built in that time, there are often unknown design procedures, the load history is unknown and biological attacks might have occurred. A more advanced research and calculation could reduce the uncertainties. If it still fails there might be a way to reinforce the timber. In the past different students from Hogeschool Rotterdam have made a thesis to check whether there is enough capacity left for the extra weight. However these theses only studied steel and concrete structures. The conclusion of the theses was that steel usually has no reserve while concrete structures need to be checked for every situation (Ravesloot, 2014). Since a lot of housings have timber roof structures it has the highest potential. However there is a large spread in the year of construction and over time the timber might be slowly decaying, it is not clear how much a structure of 50 years old is more deteriorated in strength than timber of 20 years old. Upcoming buildings can prevent this problem by anticipating the extra load. Existing structures might be able to replace the ballast of present tiles and gravel with vegetation which is about the same weight (Ravesloot, 2014). It is also not clear how the vegetation load is taken into account in the load combinations.

The change in the structure and the load may lead to critical situations. To prevent failure, different expected scenarios must be evaluated. This is the starting point for the preliminary assessment.

Scenario 1: Beams are not strong enough
Event 1: Collapse of roof structure

Scenario 2: Beams are not stiff enough
Event 2: Excessive deformations

Scenario 3: Roof covering is not waterproof or root resistant
Event 3: Rot or damage can occur

Problem statement

All of these ideas and desires can be summarized into a problem statement: Nowadays the municipality of Rotterdam and its citizens both want green roofs instead of bitumen roofs for their own different benefits, however they neither have the time nor the knowledge to determine whether their timber roof structure is suitable for this extra ballast. If no research is done then Rotterdam will only be able to have vegetation on structures of steel or concrete while timber roofs are commonly present. In an ideal future every roof in Rotterdam is a green roof.

1.2 SOCIAL AND SCIENTIFIC RELEVANCE

Roofs that are vegetated are not a new concept, however applying them on this large scale to solve an urban problem makes it an interesting research. This Master thesis searches for a solution that satisfies the users which also leads to a satisfied municipality. Green roofs on existing timber roof structures are interesting for a wide range of stakeholders (private individuals, municipality, building corporations, water boards, roofing industry, etc.). The municipality of Rotterdam is eager to find out if their plan to relieve the sewers is achievable without a high investment cost. Property owners are the largest group of stakeholders, they have the biggest influence on the new design since their roof function is affected.

1.2.1 SOCIAL RELEVANCE

Private level

The property owners, residents and users are very interested in a green roof when it becomes economic or social beneficial. For instance a private individual is attracted when he gets additional room for a garden and when the energy bill drops down. These two points are also favorable for housing associations since it increases the value of the building. This research can make sure that the structure is reliable and economically justified. Furthermore a reinforcing method could change the appearance of a roof structure and decrease the available space of an attic. The new design should consider the needs of the owner.

Municipal level

The municipality does not know how many timber roofs there currently are or what their current state is. It is impossible to say how many buildings have a timber roof structure. Many of the old drawings, and sometimes their calculations, can be found in the city archive. So by doing this research a list with the variety of variables of timber structures can be made. The ideas and approach of such a project can be used for other municipalities as well.

1.2.2 SCIENTIFIC RELEVANCE

The municipality of Rotterdam its first priority is to reduce the runoff problem. They want to store as many water as possible on roofs, however the remaining capacity of the timber structure restricts the amount. In the past research has been done mostly about decay of timber trusses or deterioration of historical wooden structures like monuments. There is less knowledge about timber beams in roof structures of houses. This Master thesis can fill this gap of knowledge for engineers who need to assess if a house fulfills the necessary requirements. By reducing the uncertainties a safe and economic solution is more clear to find.

The roof structures are designed according to the old Dutch construction norms, however it is not certain whether the construction fulfills the current demands of the Eurocode or vice versa. This thesis makes a bridge between the older and new norms and contributes to the evaluation of other older timber structures.

Furthermore it is not yet clear how the permanent weight of the vegetation and the variable load of the water relates to the other loads. Reasoning about the probability of loads acting together might lead to some extra capacity in strength. This result does not only apply to timber structures but to every structure that needs a green roof.

A reinforcing method to strengthen the current structure can be modeled. The key is to find a solution that is applicable for any older timber roof structure of the same construction year or type that needs extra capacity.

1.3 GOALS, RESEARCH QUESTIONS AND LIMITATIONS

In the ideal future municipality Rotterdam wants to have a decision tool for individuals to know, with just a few clicks, what the best sustainable option is for their property with the lowest investment cost. In order to realize this tool there is a need for better understanding of the strength of the current timber beams. Furthermore there might be a solution in reinforcing the beams instead of replacing them. This thesis can contribute to this tool by setting two main goals.

1.3.1 GOALS

A main and secondary goal are set that will define the core of this thesis.

Main goal

To reduce the uncertainty of the remaining capacity of existing timber roofs structures with the intention of giving advice on the maximum allowable extra ballast to guarantee a safe construction that complies with the current regulations.

Secondary goal

To design and model a general reinforcing method for timber beams to increase the buffering capacity and that is applicable for roof structures of the same construction year along with the demands of the users.

In order to achieve these goals, the following sub-goals can be set:

- To make a clear overview of roof structures and the uncertain variables (strength, dimensions, loads) along with the construction year.
- Determine the load combination factors between the current and new load.
- Determine if the current roof structure complies with the Eurocode and compare it with the old norm: "Technische grondslagen voor bouwconstructies" (TGB norms).
- To make a clear overview of deterioration mechanisms and grading methods.
- Determine a method for predicting the maximum allowable extra ballast.
- Determine the possible options of strengthening timber structures and, if needed, calculate the capacity of the reinforced timber beams.
- To have a protocol for future assessment of vegetation on timber roof structures.

1.3.2 RESEARCH QUESTIONS

The goals and problem definition can be translated into the main question.

Main research question

How much water can be buffered on the existing timber roof structures, and how can this be increased when there is more knowledge about the uncertainties of the structure?

The conclusion becomes clear when the following sub-questions are chronological answered. These questions work as a guide and give more structure to the research.

- How many different kind of timber roof structures were constructed in Rotterdam?
- What were the design procedures in the past since the norms changed through the years, starting from the first norm?
- What happened to the strength of the timber over the years?
- What kind of (non-destructive) grading methods can be used to determine the strength?
- What is the current strength of the existing timber beams?
- What combination factors can be used for the new load occurring together with the current loads?
- Do the timber beams comply with the current demands of the Eurocode standards?
- How can the strength of the beams be increased by means of a reinforcing method?
- What steps should be followed for future assessments?

1.3.3 LIMITATIONS

Limitations are needed to keep the original problem in the center. With the available time there is not much tolerance to deviate from the main tasks.

- Only roofs that have a timber structure are considered.
- The focus for green roofs is on houses/apartments because this is the largest group of the total roof surfaces.
- The city of Rotterdam is considered as case. In the conclusion recommendations about the relevance to other cities can be given.
- The research will only take the roof structure in consideration. Other building parts that might fail due to the extra ballast are only globally described in this thesis.
- There are different types of green roofs. Only types that can buffer water are considered.
- Intensive green roofs are only considered for flat roofs. Extensive green roofs can be used for flat or sloped roofs.

1.4 METHODOLOGY

Each question requires a different approach to answer. The following strategy will result in the most complete answer with the available time. The thesis can be split up in 3 major parts, each with its own different strategy.

The past: The size of the problem

1. <u>How many different kind of timber roof structures were constructed in Rotterdam?</u>
   This question can be answered by searching for old drawings in the archives of the municipality of Rotterdam. First five cases in one range (20 years) of construction year are used to determine the variety of structure types in this range. Next all different ranges (6 in total) can be compared with each other. Information about the year of construction and the slope of the building is already available.

The key is to find a relationship between the different structures of the same time period so that not every single building has to be considered. This relation can be different things like for example same architect, design stream, district, construction year, etc. If this relation cannot be found or is to broad a probability distribution of the variables can be made based on their construction year. The disadvantage of the latter is that some structures in one range will be overdesigned.

Output: A list with different types of structures that can be used for a wide range of buildings.

2. What were the design procedures in the past since the norms have changed through the years?
   This question gives an answer to the intended strength when the roof was designed. By researching loads and safety factors in old TGB norms an indication about the original strength can be obtained.

   Output: One clear overview of the different calculation procedures.

3. What happened to the strength of the timber over the years?
   Literature research about timber decay and degradation can be studied. During its lifetime the timber might have been exposed to different attacks. A leakage, many cycles of humidity, fungi and insects are reasons the strength probably has lowered. These processes can only be assessed by inspecting the timber on site. There might also be a strength difference in one batch due to local differences. Service life models can be adjusted to fit the current situation.

   Output: A scheme with different scenarios and their impact.

The present: The current strength of timber roofs

4. What kind of (non-destructive) grading methods can be used to determine the strength?
   In the past different methods have been used to asses timber in monuments. These methods can be evaluated, adjusted and used to assess timber roof structures. A literature study about grading methods on site is needed. By for instance using visual grading and measuring the deflection an indication of the strength can be obtained.

   Output: A scheme with methods and their accuracy.

5. What is the current strength of the existing timber beams?
   The current strength is first determined with a non-destructive grading method. Inspection on site is needed to determine the weak spots. There might be some (unintended) interaction between the timber beams and the covering boards on top of them. It is possible to get old timber beams from demolished buildings so that the non-destructive grading methods can be compared with destructive testing results to find out how they are correlated. Also the influence of the surrounding structure on the test results can be studied.

   Output: A method to determine the strength value.

6. What load and combination factors can be used for the new load occurring together with the current loads?
   Reasoning about the chances of loads acting together can lead to load and combination factors. A probabilistic analysis about the presence of water may lead to more favorable factors, however this is only considered when the strength check does not fulfill the requirements.

   Output: Load factors that can be used for green roofs.

7. Do the timber beams comply with the demands of the Eurocode standards?
   This question can be answered with all the collected data by performing hand calculations for the ultimate limited state and the serviceability limited state that are based on the Eurocode. Also the norm for checking an existing structure can be used which is more flexible.

   Output: Unity checks.

The future: An efficient and economical way of reinforcing

8. How can the strength of the beams be increased by means of a reinforcing method?
   In the past timber engineers used different methods to strengthen an existing construction. A literature search to these methods would already give a first impression. Based on these methods a new reinforcing approach can be designed to fit the current situation along with the demands of the user. The new strength can be determined using a finite element method. Ultimately, if possible, experiments can be performed on the reinforced timber.

   Output: A general reinforcing method.

9. What steps should be followed for future assessments?
   All of the gathered information can be summarized into an action plan. Whenever an existing structure with timber beams has to be investigated, the engineer can follow the different steps. The protocol focusses on two main subjects. Firstly the reliability of the structure must be assessed. When the structure is still capable of safely transferring the load then the engineer can continue with a more detailed assessment for gaining strength.

   Output: A protocol for future assessments.

1.5 READING GUIDE

The report is divided into three main sections: the past (ch.2), the present (ch.3) and the future (ch.4).

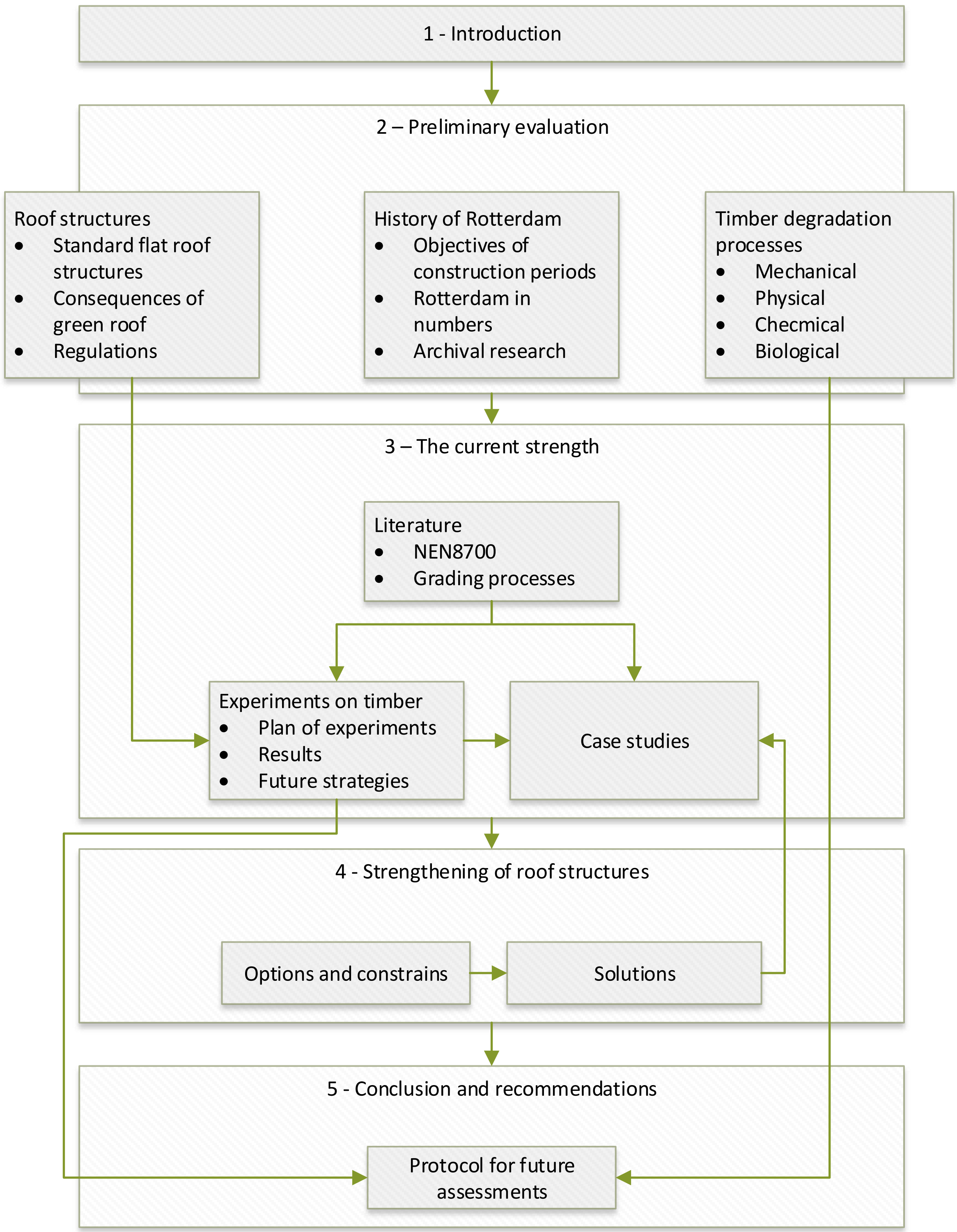
Chapter 2 starts with a brief history of important build periods in Rotterdam followed by a numerical overview. Several design aspects of built houses and green roofs are then described. To better understand the underlying reasoning of existing structures, the design procedures according to older norms are evaluated. The chapter finishes with mechanisms that cause deterioration of the strength.

In chapter 3 different ways for gaining strength are considered, starting with regulations of dealing with existing structures. Next various grading methods for in situ are described. Based on their effectiveness in practice a plan is made for experimenting with methods on obtained beams from a demolished building. The chapter ends with the test results and a strategy.

The last part, chapter 4, deals with interventions that can be taken in the future to strengthen a roof structure. Different methods are considered and based on some criteria an appropriate option is chosen.

Chapter 5 gives answers to the research questions and in the recommendations a protocol is given for future assessments.

1.6 OUTLINE


1 - Introduction
2 – Preliminary evaluation
Roof structures
Standard flat roof structures
Consequences of green roof
Regulations
History of Rotterdam
Objectives of construction periods
Rotterdam in numbers
Archival research
Timber degradation processes
Mechanical
Physical
Checmical
Biological
3 – The current strength
Literature
NEN8700
Grading processes
Experiments on timber
Plan of experiments
Results
Future strategies
Case studies
4 - Strengthening of roof structures
Options and constrains
Solutions
5 - Conclusion and recommendations
Protocol for future assessments

2. PRELIMINARY EVALUATION: TIMBER STRUCTURES IN TIME

Before doing detailed research it is important to understand the size of the problem using simple methods. Aspects like the study of documents, qualitative inspection, assessing changes during the lifetime and the conditions of the structure are essential for a first indicative overview. After this paragraph a recommendation for further research can be given. A view on the history of construction periods will explain why houses are as we know them today. Standard flat roof profiles can be analyzed to see if they match the drawings. This chapter shows where to pay attention to when assessing a timber roof structure and ends with an overview.

2.1 HISTORY OF ROTTERDAM

Throughout the years different ideas and styles have been used to construct buildings. Before examining the older drawings in the archive of Rotterdam an assessment of what is expected to be found is done by looking at standard structures. This will lead to a better understanding of older structures. Below a brief history of important construction periods is given.

Rotterdam was not always part of a metropolis as it is today. Before 1870 there were not many houses, the city was minor compared to Delft or Dordrecht. Around 1872 new wet and dry infrastructure was created which boosted Rotterdam. Many people were drawn to the city because of its fast growing economy during the industrial revolution. This strong growth of population led to the first housing shortage (Gemeente Rotterdam, 2015). Neither were there any large scale developments. Furthermore banks started to give credit mortgages which led to an uncontrolled growth of houses. These houses were built by individuals and without any steering often resulted in very poor constructions (Jellema 8, 2005). This time is also known as the jerry-building. An example in Rotterdam is the district of Oude Westen.

Due to the cholera epidemic and the poor living conditions, doctors started to support more legal guidance at the end of the 19th century. Eventually in 1901 the housing act was created to control the quality of the new, old and renovated houses. However many builders neglected the act and built alkoofhouses until 1937. These kinds of houses were standard during that time. In 1916 Rotterdam created the municipal housing department from which architect J.J.P. Oud left his mark on houses built in districts Spangen and Tussendijken. After the first world war Rotterdam became a playground for different architects which was called “the New Construction” (Gemeente Rotterdam, 2015).

During world war two the city center was bombed and afterwards destroyed by the fires. A total of 30.000 houses was lost, only a few buildings were saved (Kraayvanger, 1946). It was concluded that if Rotterdam wants to counteract the housing shortage it should build 90.000 houses over 10 years. A short time later an urban plan called the “Basisplan” was made to modernize, recover and expand the city. An important aspect was to create a new city center by not just renovating it, but also demolish the structures that could be saved. Furthermore offices and stores were planned in the center while houses moved to the suburb. These separation of functions should lead to an efficient city. Rebuilding of the ports and buildings started directly after world war two, which is now known as “the reconstruction” (Gemeente Rotterdam, 2015). Due to the many destroyed houses the need for residents was high. Although they made use of industrial building methods, which is faster to construct, there was still a lack of building supplies (Jellema 8, 2005). Different types of houses were built that are suitable for diverse age groups. Many architects are known for their influence in this time period. Typical post-war houses are the porch-tenement houses. In 1956 the government in the Netherlands stimulated municipalities to have long-term contracts with system builders, the so called “continucontracten”. Rotterdam did not participated in this which led to a higher variety of housing types than other cities (Thijsen & Meijer, 1988).

After the reconstruction period the focus was on slum clearance and cleanup because the outdated districts received no care during the reconstruction. Furthermore the attention was mainly on the city center. These plans led to many protests against the demolition of districts in 1970. Also there was a lot of criticism on the built houses of the reconstruction, the appearance was too commercial and unsociable. A new policy was made to satisfy the citizens using the slogan “Building for the neighborhood”. The aim was to maintain, renovate and improve the pre-war buildings instead of demolishing them. This time period is now known as “urban regeneration” (Gemeente Rotterdam, 2015). To boost the city center the municipality started to create small houses, more green and stopped the construction of offices. However, this led in 1984 to a dull appearance. Some roofs were replaced by roof-boxes. Until the end of the 80’s the focus was mainly on renovating buildings from the 19th century. Afterwards the focus went to the pre-war buildings until the end of the 20th century, then the focus was on post-war buildings. In this time period an oil crisis and eventually an economic crisis occurred. The lack of oil changed the way of thinking about energy. Many older buildings received insulation during the renovation (Jellema 8, 2005).

There are no large urban expansion projects planned in the 21st century, houses are refurbished and empty spaces are efficiently used. Nowadays Rotterdam wants more green in the city. Different development plans are drafted to make this possible. A good example of these developments is visible in sub municipality Delfshaven, where all future houses which are planning to that have a flat roof are obligatory to make a green roof (Algemeen Dagblad, 2008). Another example is the development plan of district Oud Westen which mentions that new or renovated buildings should anticipate the extra ballast of solar panels or green roofs. This is not mentioned in all development plans and if it is mentioned than it still remains uncertain if contractors actually take this into account.

2.2 ROTTERDAM IN NUMBERS

Nowadays Rotterdam has almost 300.000 houses throughout 13 sub municipalities. Each region has its own history which led to different property owners and building styles. When looking at the construction year of the houses in Rotterdam it becomes clear that the largest part was built before world war 2 (see appendix B). The second largest group was built between 1945-1970. Due to the housing shortage many extra houses were built. The last large group was between 1980-1989. Sub municipalities Kralingen-Crooswijk and Prins Alexander built many houses in this time for the urban expansion.

It was already mentioned that 76% of all buildings in Rotterdam have a flat roof. This percentage is lower for houses, here the total flat roof surface is still two times higher than the sloped roof area.

2.3 LITERATURE REVIEW OF HISTORY AND TYPOLOGIES ROOF STRUCTURES

Previous research on older timber constructions is mainly focused on monuments or churches. The use of timber beams to support a roof already existed long before the roman era. Many things changed over the years, newer build techniques were developed and more different wood species are used. Rotterdam started to grow around 1870, in this thesis only changes after this moment are considered. Roofs can be divided into two main groups: flat or sloped. In the past centuries a sloped roof was commonly built because water was easier to drain. The rise of zinc as roofing material in the 19th century made new structures like a flat roof possible. Around this time mainly softwood is used for the construction. Even though there were often problems like rot, the builders were still attracted to it. More modern building techniques, which used iron or steel and later on also concrete, lowered the market position of timber (Janse, 1989). According to Veerman[1] it is expected that 90% of the pre-war houses have a timber roof structure. The post-war houses are harder to estimate because reducing the housing shortage had the priority. A faster construction method was possible with modern build techniques which made use of prefabricated elements. Thijssen and Meijer (1988) encountered in their research a slope roof with lightweight concrete slabs.

Appendix C shows standard building methods of timber roof structures. Common practice is a masonry wall with a notch for the support of the timber beams. This means that the timber is in direct contact with the masonry wall. In the past this wall was also the outer wall of the building and thus subjected to different weather conditions. This could lead to moisture related problems. Around 1960 the making of a cavity wall became mandatory. Other moisture problems in roof strucutres came from bad insulation. A cold roof system also caused high humidity.

2.3.1 QUALITATIVE VISUAL INSPECTION

Preliminary visual inspection is difficult to perform when it comes to flat roofs. The room under the roof is often used as living space unlike a sloped roof were this space is more commonly used as storage area. This means that whenever the room is used as a living space the bottoms of the beams are concealed behind (plaster)boards and the top is covered with roofing. It is not possible to see the structure without demolishing or removing the finishing.

One location is visited that has a sloped roof and where the beams are visible due to already demolished ceilings. The roof consisted of a new and older part (see figure 2-1). The new part came from a renovation project around 1983 while the original structure dates from 1923. During the inspection attention is paid to the condition and the critical parts. The sizes of the beams and the moisture content are measured. Unfortunately a fire happened in the older part of the structure. The heat and extinguishing may have influenced the moisture content. In the new beams some small cracks were visible but nothing major. The outer wall does not look old so it is possible that this is placed during the renovation along with a cavity.

Figure 2-1: Older and newer parts roof structure

The sizes of the older beams are 60x150 mm while the new beams are 58x156 mm. The moisture content is measured with a magnetic moisture meter at 7 places that were expected to be critical.

[1] Phone contact on 24-02-2015, John (V.W.) Veerman studied history of art and building history along with renovation. Nowadays he works as an individual and is founder of Veerman Bouwhistorie.

- Old beam left side at wall: 17,5%
- Old beam right side at wall: 30,0%
- Old beam right side middle: 14,0%
- Old beam right side at timber frame: 9,5%
- Middle of timber frame: 13,6%
- New beams right side at timber frame: 13,2%
- New beams right side at middle: 16,0%

Noted is that the highest percentages are found at the wall where the beams make direct contact with the inner wall (see figure 2-2). At the timber framework a shoe is used. The decking consists out of planks and is attached to the beams by means of nails.

Figure 2-2: Beam in notch masonry and in shoe

2.4 THE VARIETY OF ROOF STRUCTURES IN ROTTERDAM

It would be very unpractical to look at all houses and check how they deviate from the standard roof structure. Also two or three case studies would not be sufficient to understand the timber roof structures through time. A method between these two extremes is used to get the best result in the giving time.

The information about the history of Rotterdam and the history of roof structures can be compared with old drawings of houses to see how the literature matches the reality. Due to the high variety of buildings in Rotterdam it is necessary to find key parameters that can divide the many buildings into strength groups so that only a few roofs of that group have to be considered. Important aspects like ownership, year of construction and housing type are needed to divide houses into groups. Especially the construction year is important since the building methods changed before and after the war.

Looking at the history of Rotterdam, six main groups exist:

- Group 1: All houses before the housing act which are the jerry-buildings (<1901)
- Group 2: All houses from the housing act until the new construction period (1901-1919)
- Group 3: The new construction period until the bombing of world war 2 (1920-1939)
- Group 4: The reconstruction period (1940-1969)
- Group 5: The urban regeneration period (1970-2000)
- Group 6: The new buildings (>2000)

All documents of the preliminary build phase are stored at the city archive in Rotterdam. The original building plans and all changes after the completion that need a building permit can be found in the archive. The documents usually consist of a permit request, original drawing and calculations of the construction. The key to finding the right dossier is the permit code which can be found on the website of the city archive. A street name and number refers to a specific permit code. Unfortunately the archives of Rotterdam were bombed during world war two which caused the loss of all drawings between 1904-1940. However some renovation projects around 1980 made drawings of the old situation before they started to renovate. Renovation projects usually consisted of more building complexes which fall under the same permit. Furthermore dossiers of built houses before 1940 were requested from municipality Delft and Schiedam to fill the gap of that period. One problem that occurred is that some files are incomplete and thus no conclusion can be drawn.

Looking at Rotterdam on a sub municipality level it becomes clear that Delfshaven has the richest history and can be used as the average region. Delfshaven is located close to the city center and 11,3% of the houses of Rotterdam are located there. Also the density in this region is high, 95% of the buildings are houses. Delfshaven can then be divided into 8 districts with their own characteristics. Table B-1 in appendix B.3 gives a short overview of the districts and indicates the period when many new houses were built. Note that there are not many new buildings between 1960 and 1980. Here in the urban regeneration period renovation had the priority. Behind each building period in brackets the slope of the current roof structure is given. A mixed roof structure mostly occurs on pre-war buildings, this may indicate that the renovated houses first had a sloped roof but are now flat. The search was mainly focused on Delfshaven. To get a total overview of Rotterdam the search is completed with randomly chosen houses throughout all sub municipalities.

The technical variables that need to be examined are the lengths of timber beams, the dimensions, the distances between the beams, the boundary conditions, the permanent and variable loads, the used timber species, the used maximum stress and the modulus of elasticity. These variables are the uncertainties in a calculation. Furthermore the roof covering can be of use. Remarkable aspects will be noted. To support these ideas a number of old drawings are evaluated. Appendix B.4 shows an overview of the requested drawings.

2.4.1 ANALYSIS OF OLD DRAWINGS

Only houses that currently have a flat roof were evaluated. 50% of the these houses currently have a timber roof structure. Other popular structures are steel SAB plates or a prefabricated concrete slab. It is observed that many of the houses have 4 build layers.

The first aspect that stands out is the urban regeneration period. In the 80's the plan was to renovate all pre-war houses. The old drawings confirm that this has been done (see figure 2-3). Only one drawing of a pre-war building is found that was not renovated. The reason for this renovation is often the merging of more houses into one because the compact living situation was not acceptable anymore. This includes a change in the layout and a new roof structure.

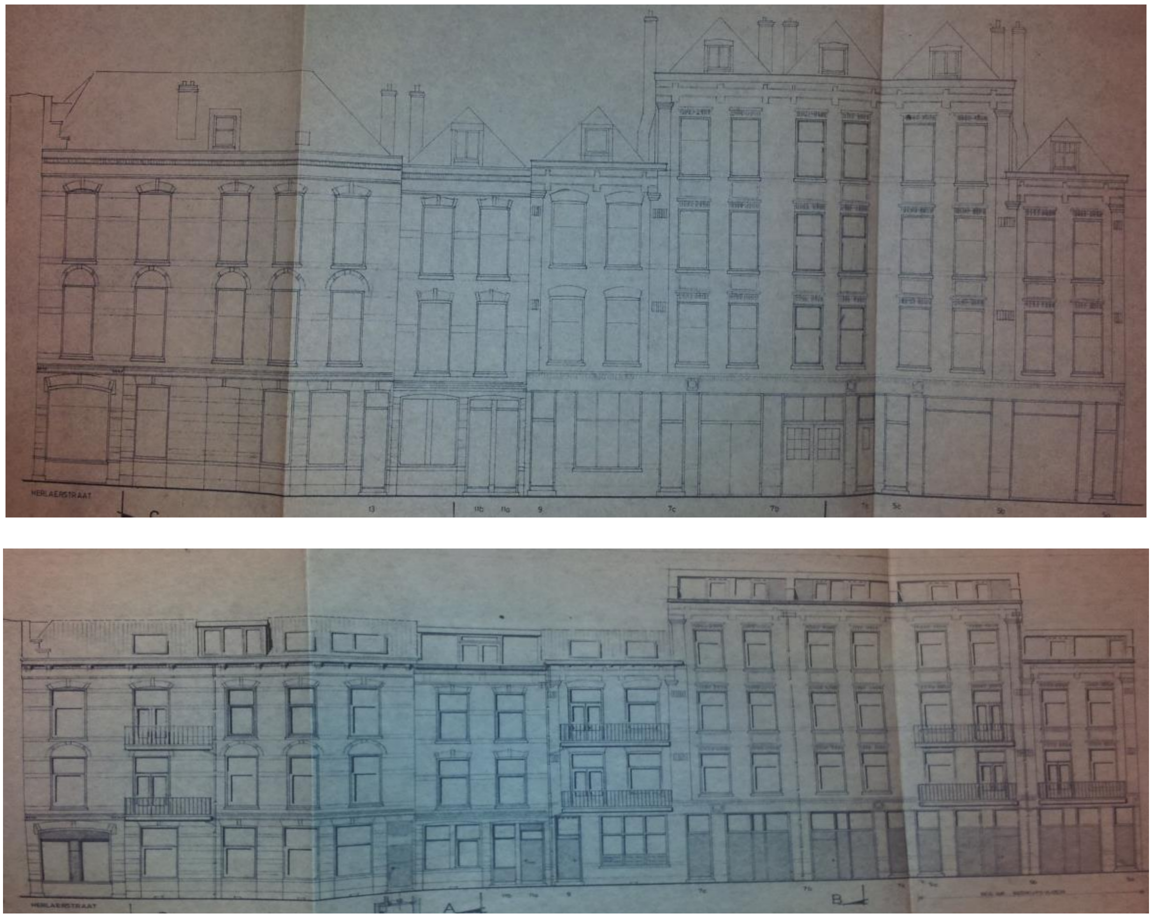


Figure 2-3: Herlaerstraat old (1890) and new (1982) façade

In one case it was noted that a beam had rot. It can be assumed that this defect occurred at more houses and was one of the reasons for a new roof structure. Inspection on site is therefore always needed when giving advice on the maximum allowable extra ballast.

A renovation of the roof is done in different ways. In almost every case the original roof had a slope while after the renewal it became flat. However since only flat roofs are considered it is hard to say how many of the original sloped roofs became flat, but it is safe to assume that this change happened in most projects. Searching for a flat roof of a house that was built before 1880 was harder than other periods. Although they are renovated they did not lose their slope.

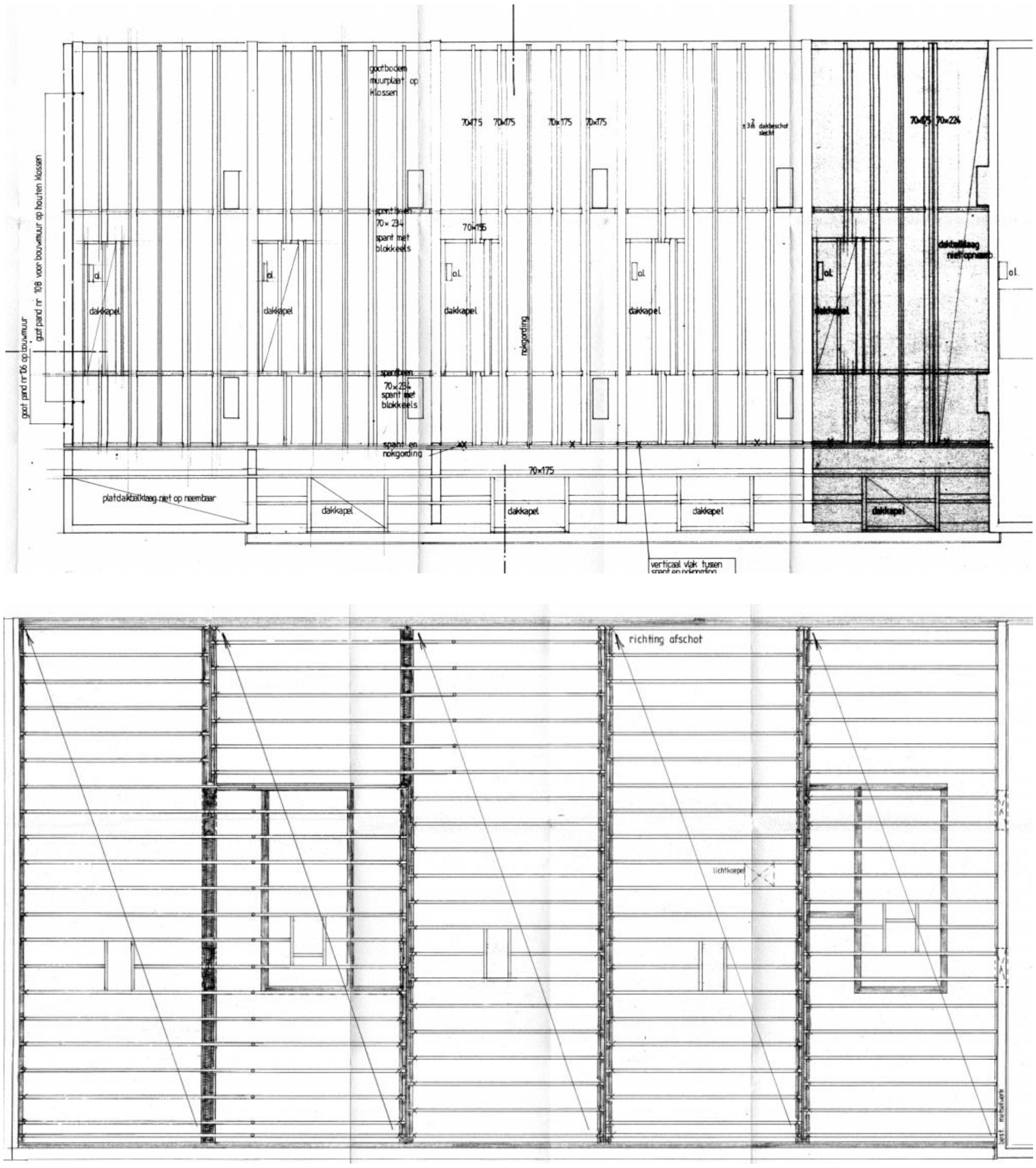


Figure 2-4: Roof beams of Taandersstraat old sloped (1925) and new flat (1985) structure

The history of Rotterdam showed that after 1945 the need for houses was high. This led to new building methods where timber roofs were no longer standard. Fast construction had the priority and this was not possible with traditional building methods. Thijssen and Meijer (1988) concluded in their research about houses between 1945 and 1965 that more than one third of the buildings have a non-traditional building method. This matches the findings on the old drawings where one third of the houses have a concrete roof. Many of these post-war houses have shortcomings like bad isolated roofs. One of the drawings showed a cold roof system structure which could lead to rot. Although Rotterdam did not participate in the “continucontracten”, system building got the upper hand after 1956. The rise of industrial building methods lowered the dominating timber roof structures. Steel SAB plates and prefab concrete slabs were mostly found in the new post-war buildings. It seems that mainly structures which originally had a timber roof currently still have one. Another point of interest is that according to the literature post-war buildings were renovated in the 90’s. This was not visible on the building permit which concludes that no renewal of the buildings was done and thus the structures are still in original state. The renovation projects were probably focused on refurbishment. One rule that seems to be valid is whenever a house has timber floors, the roof will also be of timber (see figure 2-5). However vice versa is not always true.

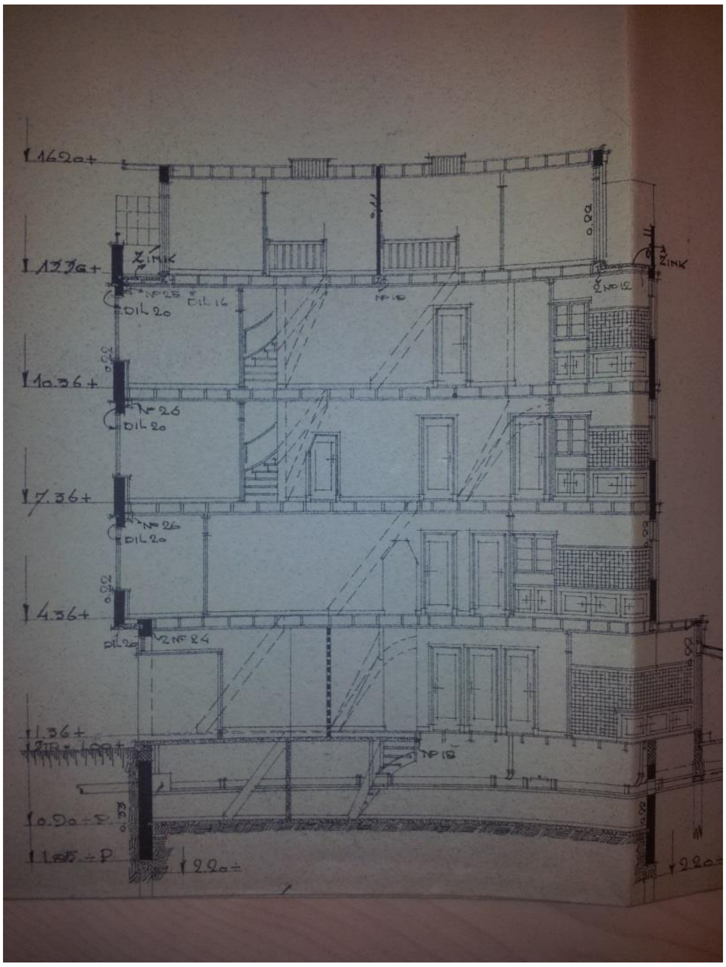


Figure 2-5: Cross section house on Schieweg

Eventually none of the newer structures had a timber roof even though there is no longer a shortage of houses. This does not mean that timber roof structures are no longer built. The houses after the year 2000 have mixed roof angles. Looking at recent building projects it seems more sloped roofs are built with concrete slabs, the priority of a modern house changed the simplicity and efficiency.

The end of the beam might have issues with a high moisture content. It is not always clear whether a cavity is present because on most drawings the building is part of a block and only the house in the middle is visible. The beams span in the shortest direction which is in the longitudinal direction of the block and thus the adjacent houses protect the ends of the beams from weather conditions. Two cases are found where the beams span to the outer wall. The first case is from 1923 where only one wall is shown. In 1983 an inner wall is placed but the roof structure stayed untouched. The second case shows a house of 1956, here the beams are supported by an inner limestone wall (see figure 2-7).

When looking at the structures themselves the attention is drawn towards the width and height of the beams. It seems that around 1980 when the standard sizes were included in norms a more uniform distribution in sizes was used. This also indicates that after the renovation of a roof, new timber beams were used. Furthermore the distance between beams became more uniform. A space of 600-610 mm is widely used. The beams are most of the times simply supported on the walls and anchored with a steel strip like the hook or strip anchor (figure 2-6). In some cases the beam goes over the support and is coupled to the next beams with a nipped scarf joint. This may indicate that a defected part is replaced with new timber. The length of the beams varies, a span of 4-5 meters is most common.

A small gradient is applied to prevent water accumulation, the arrows in figure 2-6 show the direction of this slope. Information about the decking is rarely found. This could indicate that these have standard values.

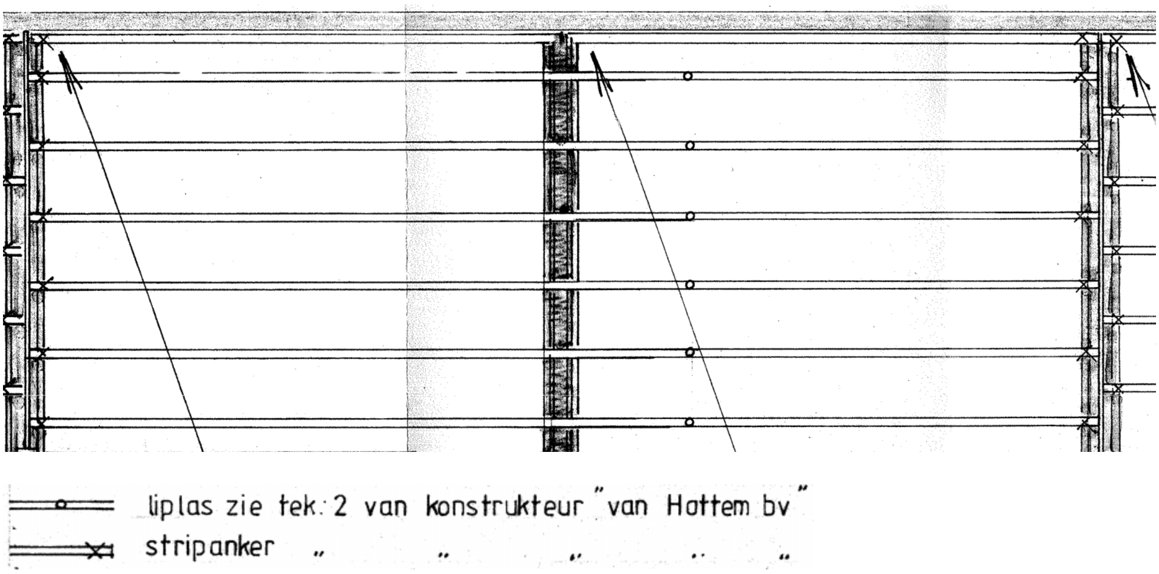


Figure 2-6: Beam layer of Taanderstraat showing hook and strip anchors

How the end of the beam is connected with the wall is often unclear. One case is found that shows this detail (see figure 2-7). Here the beam lies in a notch of the inner wall. It can be assumed that this is often the case for roof structures.

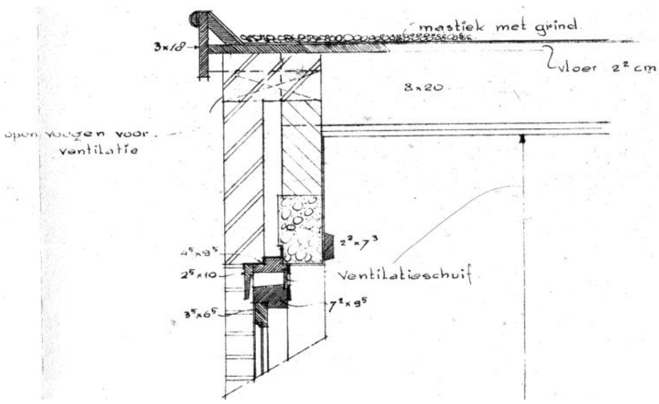


Figure 2-7: Roof detail of van Drimmelenstraat

When considering the strength of the timber the group standard building wood is commonly present. In one case beams of the group construction wood are found. In more recent projects the strength class is mentioned. Class K17 was encountered which refers to quality class C for spruce and pine. Several drawings also showed the roof coverings which varies between a warm or cold roof system and sometimes with gravel on top. Mastic (asphalt) is often used as waterproof covering.

In some cases the calculations were also present. Unity checks were performed that used a maximum allowable stress of 7 N/mm² and in one case 10 N/mm². The modulus of elasticity is always 10000 N/mm². These values are not to be confused with the characteristic values as we know them today. In the past the safety factors were included in the allowable stress value. More information can be found in appendix E.4. Some of the requested dossiers show the loadings on the roof, see appendix B.4. When looking at the weights it becomes clear that the renovated structures are lighter. The main reason is the roof covering, a gravel layer leads to more ballast.

2.5 GREEN ROOF

A first description of a green roof is given in the introduction. Intensive or extensive green roofs are both an option, the municipality leaves this choice to the user. When intensive use is chosen, than an access to the roof is needed. This can affect the roof structure. Furthermore it is common practice to increase the weight of zones that are submitted to high suctions of wind. An example is a gravel layer around the edges and corners.

2.5.1 REGULATION

No permit is needed when the structure is untouched or inaccessible. When reinforcing of the roof structure is needed or when the roof becomes accessible, the owner needs to apply for a planning permission. Included should be calculations that show compliance with the Building Act. The document should consist of the following information:

- Loads and load combinations for strength and stability of the complete building
- ULS of the structure and parts of the structure
- Drawings and calculations of the existing structure
- The used materials
- A written explanation of the design

No special demands are given for a green roof. The permit department of the municipality will check and provide the permit. Thereby only the part of checking is for the municipality, the responsibility of the design stays with the structural engineer. The engineer is held responsible if the structure fails.

During construction and intensive use of a green roof the risk of falling should always be accounted for. For construction the working condition act is active. These safety measurements are sometimes temporary and only used again during maintenance. The responsibility during use or maintenance is with the owner of the building, this is determined in the housing act. A solution to prevent falling is by making a railing. Figure 2-8 shows measures that can be taken.

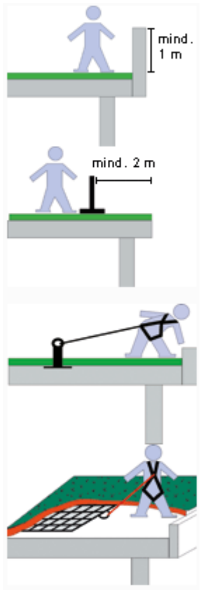


Figure 2-8: Measures for falling (ZinCo, 2015)

2.5.2 CONSEQUENCES OF A GREEN ROOF

Making a green roof will have consequences for the climate in the room below the roof and thus also affects the timber structure. As noted in the introduction, a green roof has an insulating effect in the summer and the winter. This reduces the alternation of temperature in the structure. According to Groendak b.v. no exact insulation value can be given and is highly dependent on the thickness of the layers. The relative humidity stays the same for warm roofs or is positively influenced for cold roofs. A negative consequence of vegetation is when the water or root resistance covering fails. The former will result into leakages that are only discovered when the ceiling becomes wet. A timber structure above the ceiling may not be exposed too long to water because this increases the chance for biological attacks. The leakage spot is often hard to find with vegetation on top. Early-detection devices have been developed for faster localization of the leakage spot. Failure of the root resistance covering might lead to damage of the decking. This is highly unwanted, especially when the decking is working together with the beams. The negative consequences give rise to the question: how is the timber roof structure being monitored when visibility is impossible or limited? Solutions can be searched in endoscopy, a hatch in the ceiling or permanent present measure equipment.

Another consequence is the extra load due to vegetation and water. In the past accessible roofs (besides maintenance) were calculated as a floor. This leads to a stronger structure. Next, the extra load on top also needs to be transmitted by the insulation in case of a warm and reverse roof. Insulation has a certain compression strength in order to resist permanent deformations. Different insulation materials can be present: organic (cork), synthetic (Expanded Polystyrene, Extruded Polystyrene, Polyurethane, Phenol formaldehyde) or inorganic materials (rockwool, cellular glass, perlite) (Jellema 4a, 2005). The lowest compression strength for insulation plates without permanent deformations is 20 kN/m² for cork (EnviroNomix, 2009), other plate materials are higher. In case of wool the insulation will become more compressed and is therefore less effective.

2.5.3 GREEN ROOF SUPPLIERS

In practice a green supplier has experience with timber roofs. Four green roof suppliers (Groendak b.v., Optigroen, Groenedaken.net and Zinco)[2] were consulted to gain more knowledge about the possibilities. The municipality wants to buffer 25 L/m². A rule of thumb is that 1 cm of substrate layer holds 1 L/m². The vegetation to contain this amount of water is usually sedum because these plants are greasier. The saturated weight is then around 85 kg/m². However the suppliers recommend a retention roof because they can retain the water plus the water of a second storm after 24 hours can also be buffered. This is possible because a thicker substrate layer can grow higher vegetation which evaporates more and thus allows new water in a shorter period. The retention roof weights around 120 kg/m² in wet conditions. A second rule of thumb is that each day 5 liters of water is evaporated. The demand for 25 L/m² is possible for roofs till 5° and becomes harder to achieve when the roof has a higher slope. An angle of 40°-60° can theoretically buffer 20 L/m² but this proves to be difficult to hold since all the water is gathering in the lowest point.

Two suppliers were also confronted with the question if a green roof is suitable for older timber structures. Groendak b.v. does inspection on site and performs calculations on the existing structure with the modern procedures, for older structures consultation with a structural engineer is needed. Optigroen follows more or less the same procedure but uses a more practical approach. A person around 85 kg walks on top of the roof structure and listens if the timber creaks. When no sound is heard a low weight sedum roof is possible. Popular techniques to increase the strength of the structure are decreasing the distance between beams (more beams in a row) and use a stronger decking.

2.5.4 THE LOAD

The load depends on the use (intensive/extensive), the thickness and the amount of water present. The latter depends on the delay coefficient, thickness of the layers, rain intensity and flow rate outwards. Other secondary factors that influence the amount of present water are the evaporation speed, extraction by plants, the location and the gradient of the roof.

A green roof supplier has a variety of systems for different purposes. For this reason the saturated load also varies. Table 2-1 is used as a starting point. Extensive roofs make use of low growing plants that require little maintenance. The weight of intensive roofs has a high uncertainty because it depends on its use. Loads vary from a herb garden to a fully grown garden with terrace. High growing vegetation like trees are not expected to be used on a house. A maximum value is set on 3,4 kN/m² based on the roof garden system of Zinco. The last column shows which other variable loads need to be considered although the combination factor is not known yet.

Green roof	Dry condition	Saturated condition	Combination load
Extensive	0,75 kN/m²	1,00 kN/m²	Maintenance Snow
Intensive	0,75 kN/m² - 2,30 kN/m²	1,00 kN/m² - 3,40 kN/m²	Adjusted floor load Snow

Table 2-1: Loads from green roof

[2] Respectively websites: www.groendak.info; www.optigroen.nl; www.groenedaken.net; www.zinco.nl

Green roof suppliers always use the weight of saturated conditions. Note that the weight of a dry condition is calculated as the saturated condition minus the density of buffered water. According to soil mechanics this is not correct. Soil consists of air voids and pores which have a negligible weight. The real dry value would be:

$$\gamma_d = (1 - n) * \rho_k * g \quad \textit{(Eq. 1)}$$

Where n = porosity [-]
ρ_k = density of soil [kg/m³]
g = gravity [m/s²]

The actual weight would thus be lower but due to the varying values of different green roof systems the values in table 2-1 are used as a starting point.

Extensive green roofs

The weight of the dry condition is always present and therefore classified as a permanent load. The presence of water is time dependent and thus a variable load.

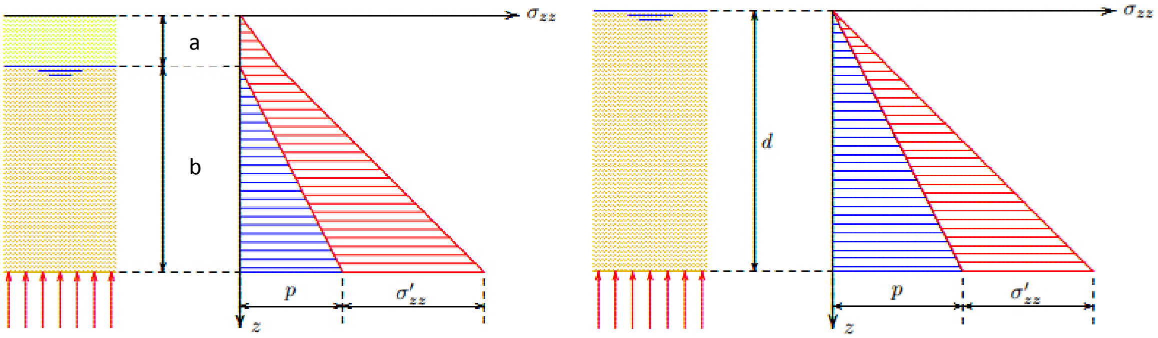


Figure 2-9: Partially saturated (left) and fully saturated (right) soil (Verruijt, 2010)

The total weight consists of pressure from the water and the soil.

Intensive green roofs

An intensive green roof allows also a load on top.

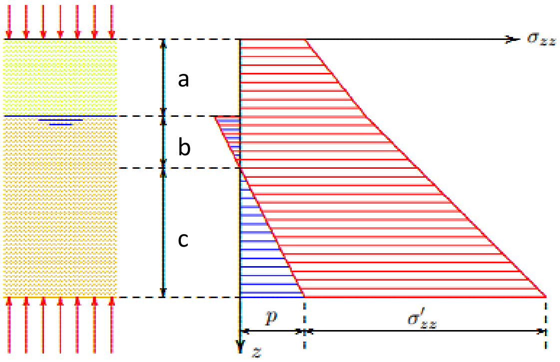


Figure 2-10: Partially saturated soil with capillary action and load on top (Verruijt, 2010)

The total weight consists of the water, soil and pressure on top.

A completely dry ground will rarely be present because there is always some moisture in the pores. There are two ways for considering the combination between dry ground and the water:

- Deterministic approach (This option is chosen in this thesis)
  The saturated soil is seen as a permanent load: γ_g (G + Q_{water}). This is plausible because the purpose is to buffer and slow down the water drainage. Besides, the load has a maximum value (extra water is discharged by the emergency overflow) and thus it makes sense to use a smaller partial factor because the uncertainty of exceeded loading is small. Even though the load can be predicted with good accuracy it is recommended to still use a load factor due to possible gardening in the future.
- Probabilistic approach
  The water is seen as a variable load: γ_g G + $\gamma_{q,adj}$ $Q_{water,adj}$. A probabilistic analyses of the expected water being present can lead to a lower partial factor than γ_q. Also only a part of the water load needs to be taken into account since the uncertainty of present water is considered in the partial factor.

Another aspect is the combination with the loads from maintenance, snow and intensive use. Because the green roof can contain water for 24 hours, it is likely that during this time a variable load is present. Also here a deterministic or probabilistic approach can be considered.

Combination with maintenance load: Q = 1 kN/m², Ψ_0 = 0
When the soil is saturated and a leakage is spotted than a person must be able to access the roof for maintenance.

Combination with snow: Q = 0,56 kN/m², Ψ_0 = 0
During cold seasons it can for instance rain during the day and snow during the night, all within 24 hours.

Combination with intensive use: Q = (0,60 – 0,90) x 1,75 kN/m², Ψ_0 = 0
The loading for floors in a house can be compared to the intensive use for a roof. This load takes the presence of persons into account that are dancing or stomping. However the load also contains the weight from furniture which may not be present depending on the function of the roof. A herb garden will weigh less than a terrace. It would be wise to control this load by setting some boundaries of what is allowed.

The probabilistic approach will only be considered when more strength is needed. Furthermore the duration of load determines the modification factor k_{mod} on the strength. Because the green roof is seen as a dead load, the load duration class for green roofs is permanent.

2.5.5 TRANSFERRING THE LOAD

On top of the timber beams there is a decking which protects the room from weather conditions and transfers the loads to the timber beams. Two positive phenomenon occur.

The first effect is cooperation between the timber beams and the decking through a mechanical connector and thus increasing the overall strength and stiffness. This will be shown later on. The second effect is that the decking works as a plate which divides the load over several beams. Because of this spreading a part of the load is carried by the adjacent beams and thus relieving the most stressed member. This is made visible in figure 2-11.

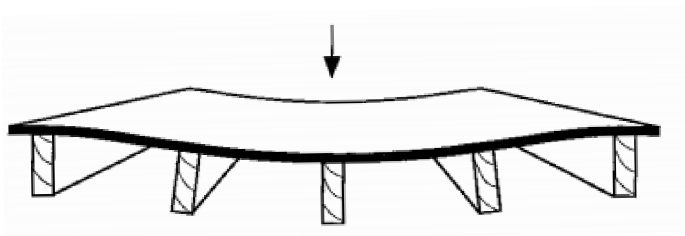


Figure 2-11: Influence of a concentrated load on a roof (Blass, Belastingverdeling, 1995)

Since the stiffness between the timber beams varies and load is attracted to stiffer parts, and thus stronger parts, the load distribution will not be even. For a distributed load the total deformation does becomes more or less the same. When a beam does get to high stresses, which causes cracks or plastic deformations and thus reduces the stiffness, a redistribution takes place so that the damaged member can still contribute to the bearing system (Blass, Belastingverdeling, 1995). The Eurocode 5 gives a factor k_{sys} which allows for an increment in strength because of the described effect. This is more or less an increase of 10% in strength.

In almost all cases a beam on two supports was found. Another possibility is three supports which causes a negative moment in the beam, see figure 2-12.

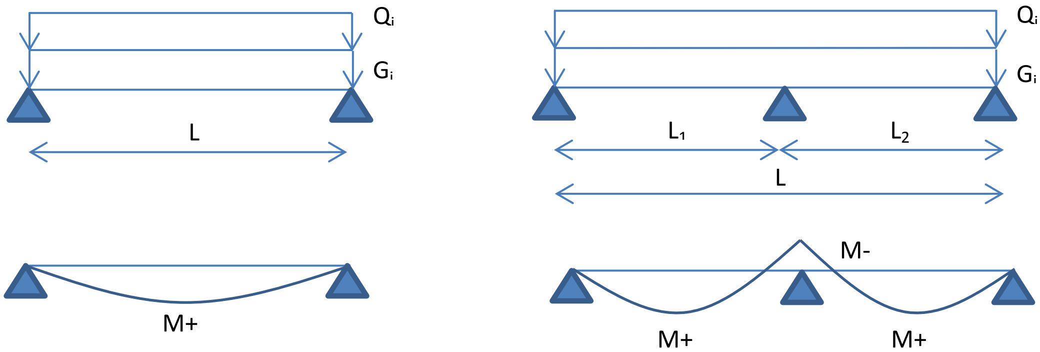


Figure 2-12: Most common mechanical systems for flat roof structures with their moment distribution

It is most likely that the beam will break due to the bending moment in the middle. A closer look at this region can explain the failure mode and a more appropriate reinforcing method can be found.

When clear wood is loaded in bending, the wood tensile properties are about three times higher than the compressive strength. A high bending moment will thus first lead to crushing of the compressive fibers. A small force causes a linear interaction between the compression and tension stress (elastic behavior). When increasing the load even more the compressive zone will deform plastically (elastic-plastic behavior). At last the neutral axis will move towards the tension zone. For equilibrium the elongation in the tension zone must increase until stresses are too high and fibers eventually break.

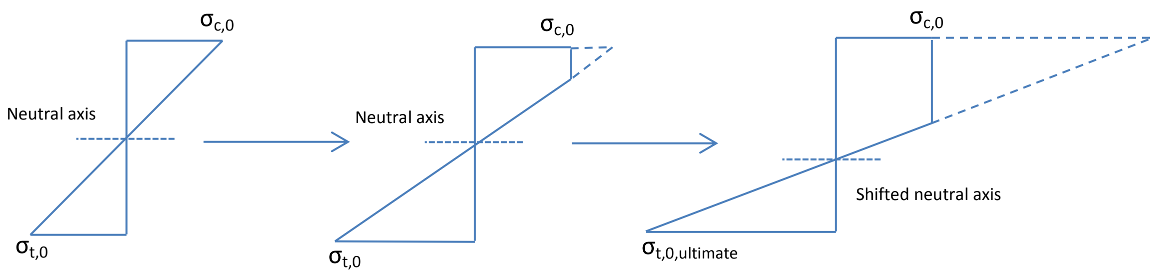
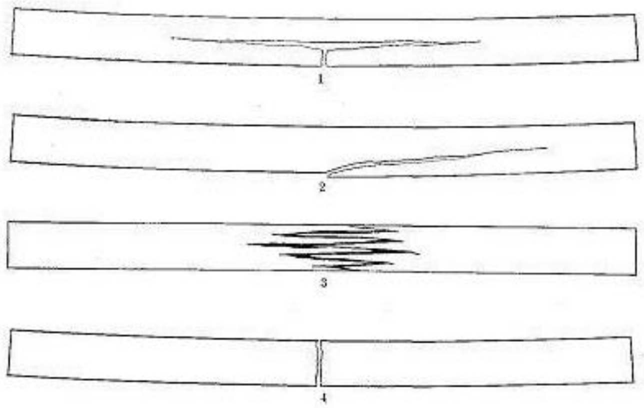


Figure 2-13: Elastic and elastic-plastic behavior of clear wood

Defects, like a knot, can reduced the ultimate tension strength in structural timber. Depending on the size of the defect, the tension strength can become lower than the compression strength.

According to (Samuel, 1914) four flexural failure mechanisms can be distinguished:



Simple tension: Tensile stress parallel to grain.

Cross-gained tension: Tensile stress with an angle.

Splintering tension: A number of small fractures.

Brittle tension: A sudden fracture entirely through.

Figure 2-14: Flexual failure mechanisms (Samuel, 1914)

The defects determine the failure mechanism, especially the knots around the middle will weaken the region. A good solution to increase the resistance is by making optimal use of the ductile behaviour and the good tensile properties.

Shear and bending stresses act in the members. However the existing decking might contribute to the load bearing. The interaction between decking and beam depends on the means of connection. Mechanical and glued connections are possible but the former is expected in older structure. In order for full cooperation the horizontal shear stresses must be transmitted. Aspects that play a role for the cooperation are: shift modulus of a mechanical connecter, amount of connecters and distance of connecters. Often nails are applied which allows for some displacement between the force and cross section, hence it is a semi-rigid joint. The beams transfer the load to the bearing walls which causes a force on the foundation.

A range can be given of the minimal and maximum strength of the roof by considering a single beam and a beam with cooperation of the decking. To make this clear the case of Kerkhofstraat described in appendix G.1 is used for the calculation. The cooperation is not taken into account during the design of the original structure but due to stiffness of the nails their might be a favorable interaction in practice.

Some assumptions are needed:

- The decking is made from plywood F20/10 E60/40 with thickness of 19 mm.
- The nails are not predrilled and are strong enough to transfer the shear force.
- The full strength of the timber elements is present.
- The vessel direction of the decking is perpendicular to the beams.
- Simply supported beam with one span.

<u>Overview:</u>

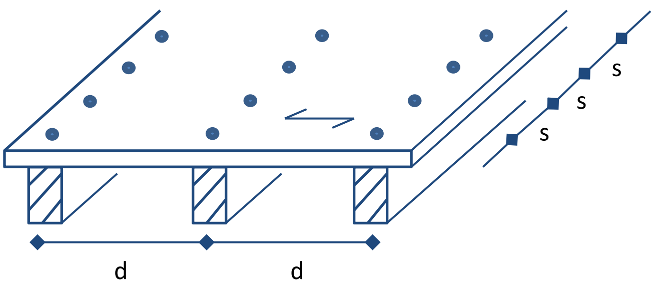


Figure 2-15: Overview of roof decking on beams

No interaction

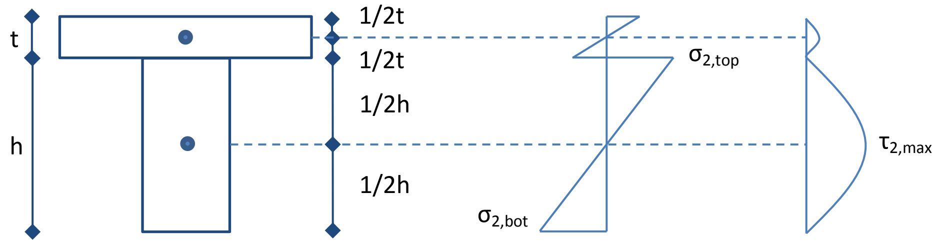


Figure 2-16: Cross section with stress diagram when decking and beam do not work together

$\sigma_{2,top}$ = -8,91 N/mm²
$\sigma_{2,bot}$ = 8,91 N/mm²
$\tau_{2,max}$ = 0,43 N/mm²

Full cooperation between decking and beam

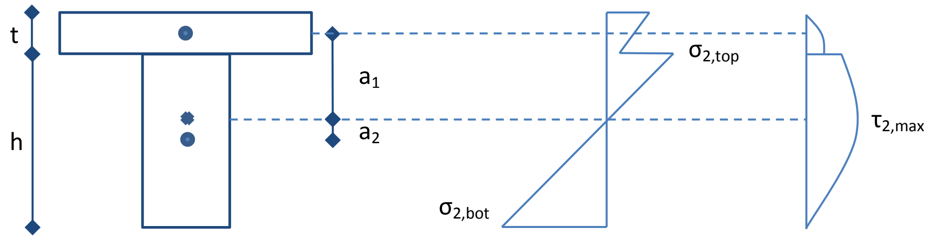


Figure 2-17: Cross section with stress diagram when decking and beam work together

$\sigma_{2,top}$ = -6,86 N/mm²
$\sigma_{2,bot}$ = 7,81 N/mm²
$\tau_{2,max}$ = 0,40 N/mm²

The real stresses will be between the two extremes but an exact value is hard to predict without any additional information. Even more so inspection on site does not reveal the connection of the decking to the beam. Nevertheless this shows that there is some unintended additional strength.

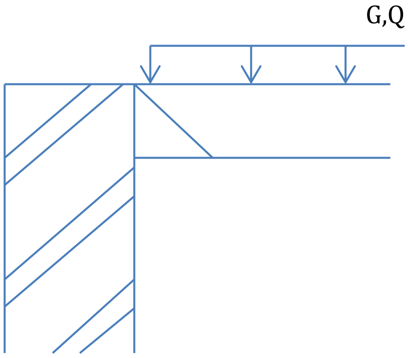
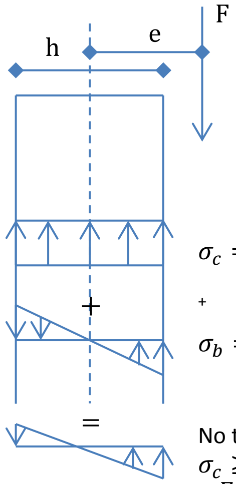
Beam to wall

The members are simply supported by the bearing walls which restricts rotation around the longitudinal axes. Various ways of detailing are possible which lead to different force distributions in the masonry. Four failure mechanisms can occur:

- Partial failure: breaking of the stone/mortar at the corner due to a concentrated load. When this occurs the resulting force will move towards the center of the stone and has a positive effect on the force distribution.
- Compression failure of the wall due to dispersed stresses. This failure mechanism is not expected because a stone has a high compression strength.
- Tension failure in the top stone.
- Buckling of the wall. This can be critical because there is no compression force from a higher floor.

Below the most common situations and their behavior are described.

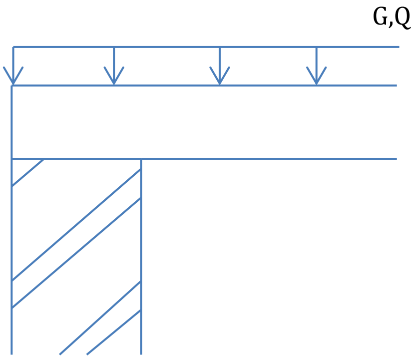
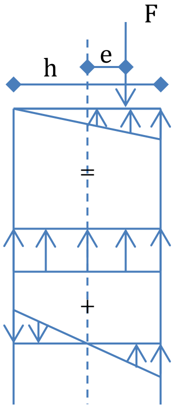
Situation A:

<table>
<tr><th>Detail</th><th>Force distribution</th></tr>
<tr><td>
</td><td>

$\sigma_c = \frac{F}{A}$
+
$\sigma_b = \frac{F * e}{W}$

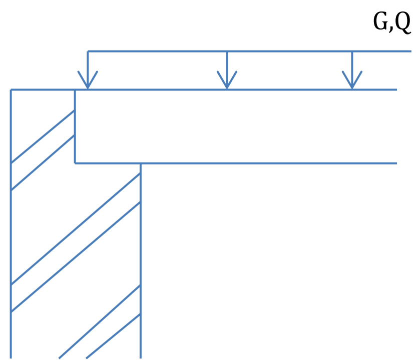
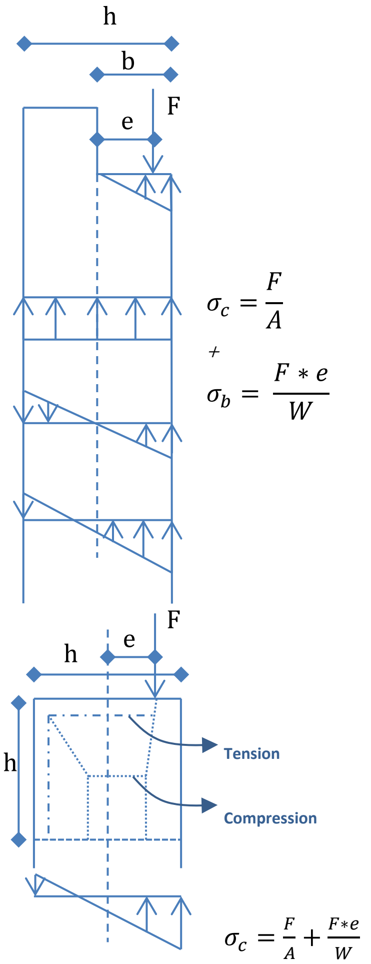
No tension if:
$\sigma_c \geq \sigma_b$
$\frac{F}{b * h} \geq \frac{F * e}{\frac{1}{6} * b * h^2}$
$\frac{1}{6} * h \geq e$</td></tr>
<tr><td colspan="2">Description: e = h/2 + 25 mm. The resulting force is outside the core of the wall. This causes a tension force in the wall. The mortar-stone connection has a very low tension strength, a gap is than formed which can eventually lead to failure in bending or buckling. The tension decreases over the height due to the self weight of the stones.</td></tr>
</table>

Situation B:

<table>
<tr><th>Detail</th><th>Force distribution</th></tr>
<tr><td>
</td><td>

$\sigma_c = \frac{F}{A}$
+
$\sigma_b = \frac{F * e}{W}$</td></tr>
<tr><td colspan="2">Description: e = h/2 – h/3 = h/6. The resulting force is on the edge of the core. It can be expected that there is partial failure of the mortar and stone due to high stresses in the corner. This should not lead to any problems. Failure in compression is the first mechanism.</td></tr>
</table>

Situation C:

<table>
<tr><th>Detail</th><th>Force distribution</th></tr>
<tr><td>
</td><td>
</td></tr>
<tr><td colspan="2">Description: e = h/2 –b/3. Depending on the supporting length some tension in the wall is expected. High concentrated stresses at the corner can lead to partial failure. Also a tension force is needed to make equilibrium however this is small compared to the compression. The most likely failure mechanism is buckling of the wall.</td></tr>
</table>

Situation D:

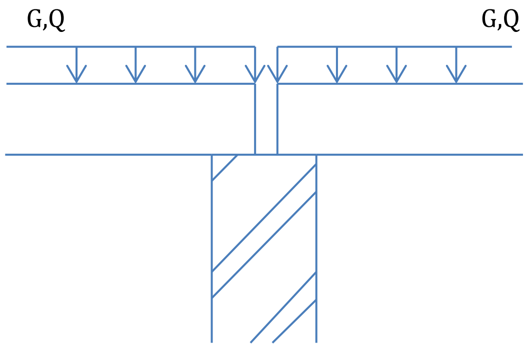
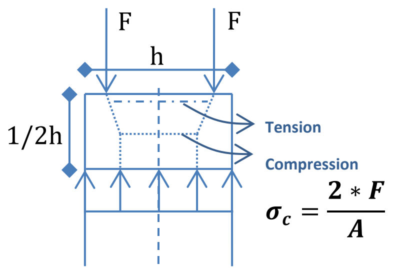
<table>
<tr><th>Detail</th><th>Force distribution</th></tr>
<tr><td>
</td><td>
</td></tr>
<tr><td colspan="2">Description: The two eccentricities counteract each other which results in an (almost) uniform stress distribution depending on the force and eccentricity ratio. At the corners partial failure may occur and between these forces a tensile bar is needed for equilibrium. High compression forces will lead to failure.</td></tr>
</table>

Figure 2-18: Ways of connecting and the resulting stresses

For extensive green roofs the added weight is 1 kN/m². In worst case scenario this means the total load is increased with 50%, assuming a permanent load of 1 kN/m² and the maintenance load of 1 kN/m². This is not expected to cause any problems.

For intensive green roofs the added weight is 1 – 3,4 kN/m² plus the intensive use of maximum 1,75 kN/m². This gives an increase of 115%-235%. Here the failure mechanisms have to be checked, especially buckling of the wall is critical.

Foundation

Two types of foundations exist in Rotterdam: piles or strip foundation. The choice depends on the soil layers. Foundations are designed on a weight calculation of the total structure. When a green roof is created, the walls will spread the load over the foundation. Therefore failure is expected due to higher compression forces in the soil or piles. However Rotterdam has some problems with foundation settlements. One of the reasons is the absence of negative skin fraction in the design.

Assuming the foundation is in a good state, a global calculation about the increase of weight can give an indication if interventions will be needed. The example is a three story building consisting out of wooden floors, limestone walls of 240 mm and a pile foundation.

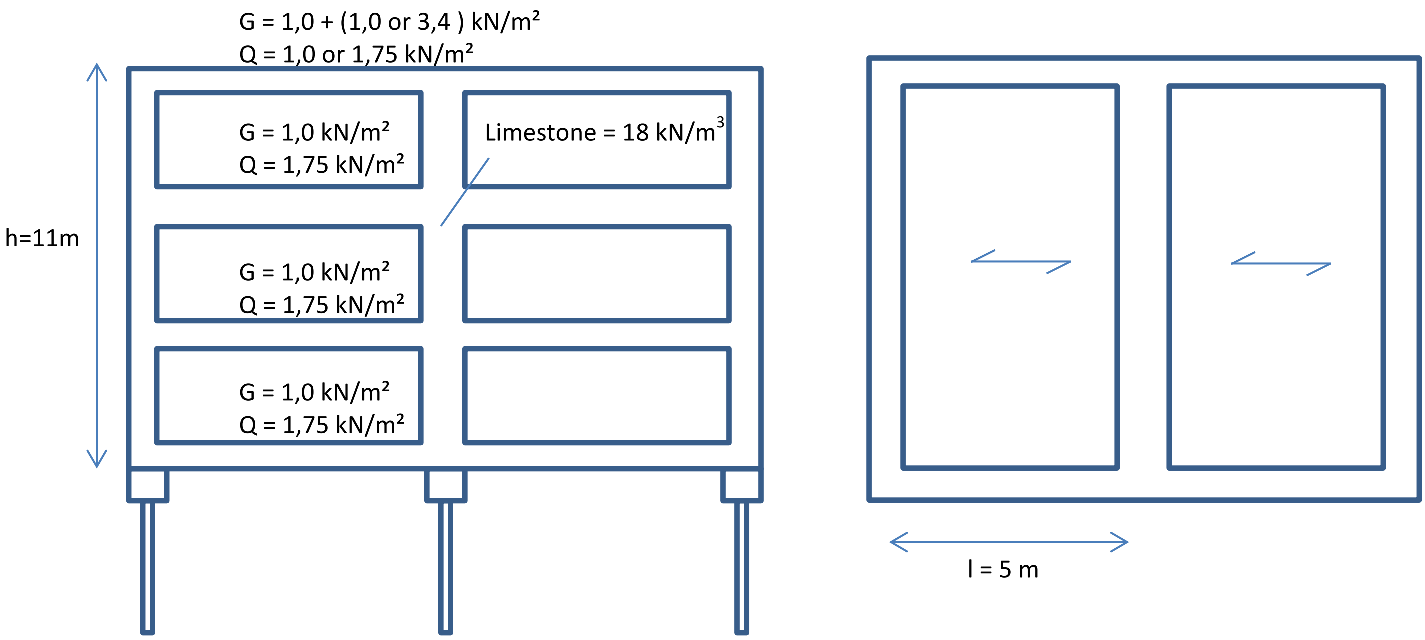


Figure 2-19: Vertical and horizontal cross section of example building

Total weight without green roof:
3 x 5 x (1,2 x 1,0 + 1,5 x 1,75) + 5 x (1,2 x 1,0 + 1,5 x 1,0) + (1,2 x 18 x 11 x 0,24) = 128 kN/m

Total weight with extensive green roof:
3 x 5 x (1,2 x 1,0 + 1,5 x 1,75) + 5 x (1,2 x 2,0 + 1,5 x 1,0) + (1,2 x 18 x 11 x 0,24) = 134 kN/m (5% increase)

Total weight with intensive green roof:
3 x 5 x (1,2 x 1,0 + 1,5 x 1,75) + 5 x (1,2 x 4,4 + 1,5 x 1,75) + (1,2 x 18 x 11 x 0,24) = 154 kN/m (20% increase)

This example is really conservative because some loads are neglected and the floors are often from concrete which would reduce the total increase of a green roof. Nevertheless it can be seen that an extensive green roof gives a small increase that is expected to be in the safe margin while the heaviest green roof (3,4 kN/m²) needs extra attention.

2.6 DESIGN CODES THROUGHOUT THE YEARS

Knowing which design codes are used for the calculation of the construction gives an indication of the minimum dimensions of the beams. This can be compared with the actual applied beam which then gives a first impression of how much strength capacity is left. Deterioration is included in the norms by means of a factor. A closer look at the norms is also needed because the lack of knowledge in the past and for safety reasons the values in the norms are always conservative even though this led to waste of material. Figure 2-20 shows the active norms in a chronological way. Unfortunately older norms are not digitized and could therefore only be visibly consulted at the NEN institute. This applies to all norms before 194

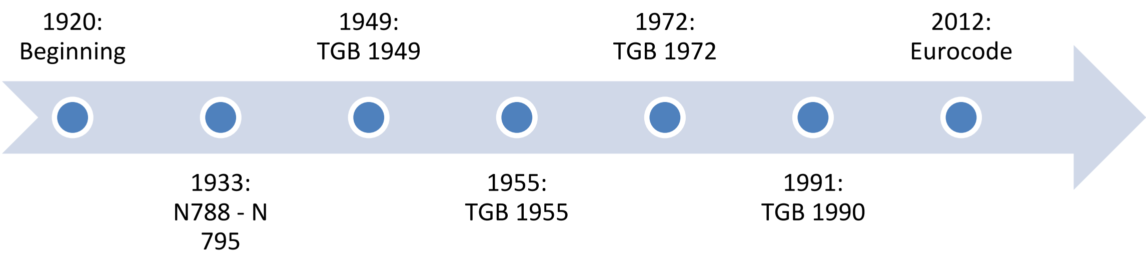


Figure 2-20: Periods of active construction norms

In between the periods of active norms, there were draft norms which gave more realistic values than the active norms. When sufficient updated values were specified and more insight in material was gained a new norm became standard. It is also possible to deviate from the standard as long as it can be proven that the structure is safe. In appendix E a list is given with the different construction aspects. Note that only parts of the norm that are needed to calculate a roof structure are included.

<u>Comparison of the norms</u>
When looking at the loads for a roof structure it becomes clear that a maintenance person on a roof is always taken into account by means of a uniform distributed load of 1 kN/m². Since 1990 there is no reduction possible when the surface is more than 10 m². This maintenance load can also be a concentrated load that since 1972 increased to 2 kN. The concentrated load becomes governing for structures with a short span and a small distance between the beams. The calculation of snow and wind load changed throughout the years but never exceeded the value for the maintenance load, which is expected to be governing for roof beams in houses. Also there is never a combination of the variable loads needed. Load from water can, when complied with certain criteria, be prevented in TGB 1972 and later. The norms before this time do not give this criteria or a value for the load.

Since the TGB 1972, timber in construction uses a factor which takes the load duration into account. Before this norm the maximum allowable stress already contained a load and material factor. The norms after 1990 separated these factors. A k_{mod} is given to consider the load duration and climate class and is applied on the resistance instead of the load. This value is different for the TGB 1990 and the Eurocode. Besides, the resistance is also reduced with a material factor which is based on the timber property. For both factors the Eurocode gives a higher value.

In all the years, the maximum allowable deflection of the beams is around 0,004 L. Until the TGB 1972 only instantaneous deflections were considered. Later on also the additional deflection due to creep is taken into account. This results into a 29% reduction on the E-modulus for the TGB 1990 while the Eurocode gives 44% reduction. According to the TGB 1972 the creep deformation is the same as elastic deformations plus an additional part from the variable load.

Example

The following example demonstrates the different cross section modulus that are needed according to the norms. A simple roof structure is checked in the ULS for its bending strength and in the SLS for the deformations. When the required beam sizes vary a lot, it would give a first indication of where strength can be gained. The case of Kerkhofstraat is used for this example. The used consequence class (CC) is 2 for buildings with 4 layers or more.

The input data is as followed:

- Length of the beams: 4150 mm
- Width of the roof: 12100 mm
- Distance between beams: 605 mm
- Permanent load: 0,60 kN/m² (including assumption self-weight beams)
- Variable load: different per norm
- Bending strength: Standard building wood or C18

ULS	TGB 1949	TGB 1955	TGB 1972	TGB 1990	Eurocode
Governing variable load	See TGB 1955	1,0 kN/m²	1,44 kN (with reduction) in middle	2 kN in middle	2 kN in middle
Load combination		0,605 x (0,60 + 1,0) = 0,97 kN/m	0,605 * 0,60 = 0,36 kN/m 0,85 * 1,44 = 1,22 kN	0,605 * 1,2 * 0,60 = 0,43 kN/m 1,5 * 2,0 = 3 kN	0,605 * 1,2 * 0,60 = 0,43 kN/m 1,5 * 2,0 = 3 kN
Moment		2,08 kNm	2,05 kNm	4,05 kNm	4,05 kNm
Maximum stress		7 N/mm²	7 N/mm²	0,85 * 18/1,2 = 12,75 N/mm²	0,90 * 18/1,3 = 12,46 N/mm²
Section modulus		297703 mm³	292460 mm³	317647 mm³	325040 N/mm³
Beam size		**63x175 mm**	**63x175 mm**	**63x175 mm**	**≈63x175 mm**

Table 2-2: ULS calculation of required beam size with different norms

SLS	TGB 1949	TGB 1955	TGB 1972	TGB 1990	Eurocode
E-modulus	See TGB 1955	10000 N/mm²	10000 N/mm²	9000 N/mm²	9000 N/mm²
E-modulus including creep				$(9000 * k_{mod})/(\gamma_m * \Psi_{krp})$ = 6375 N/mm²	$9000/(1+k_{def})$ = 5000 N/mm²
Load		0,968 kN/m	P: 0,363 kN/m V: 1,44 kN	P: 0,363 kN/m V: 0,605 kN/m	P: 0,363 kN/m V: 2 kN
$W_{instantaneous}$		13,28 mm	3,3 + 5,1 = 8,4 mm	2,2 + 3,6 = 5,8 mm	3,1 + 6,6 = 9,7 mm
W_{creep}			3,3 + 1,7 = 5 mm	3,1 + 3,1 = 6,2 mm	5,6 mm
W_{total}		13,3 mm	13,4 mm	12 mm	15,3 mm
Allowed			16,6 mm	16,6 mm	16,6 mm
Beam size		**No restrictions**	**63x200 mm**	**75x225 mm**	**75x200 mm**

Table 2-3: SLS calculation of required beam size

Note that all norms give the same cross section in the ULS, however there is a small difference in the cross section modulus. It seems that modern codes require a higher resistance. The modification, load and material factors were always present but applied in different ways. The largest difference between the norms is the load that is governing.

In the SLS the checks show a variety in beam sizes. The maximum allowable deflection did not change throughout the years but a better understanding of the creep phenomenon gives different calculation procedures. It might be the case that the TGB 1990 was too conservative and thus beams were overdesigned.

<u>Reserves in strength when comparing the norms</u>
Reserves can be gained by evaluating the following design aspects: loads, factors, load combinations, strength values and deformations. It might be possible that for some norms values were used which are too high compared to the current standard.

Only minor changes are found in the loads throughout the years. The self-weight is based on measured values of materials. Wind, water and snow were never governing. This leaves the variable load due to maintenance which stayed practically the same. (Reserve available: none)

More factors in the design procedure were distinguished as the standards were progressing. The allowable stresses for European softwoods are based on permanent long duration loads, it is therefore allowed to reduce the variable load but this is not always done in practice. The structures before TGB 1990 might thus have some extra strength capacity, however, creep was not taken into account before the TGB 1972 which compensates the over-dimensioning. Furthermore there are no big differences in load and material factors. The material factor depends on the coefficient of correlation of the strength class distribution. (Reserve available: uncertain)

The load combinations always stated that the same loads should be combined. (Reserve available: none)

The acceptable failure probability for allowable stresses is different. Two types of values for bending stresses exist: the maximum allowable stress which is based on a failure probability of 0,1% and the characteristic stress with a failure probability of 5%. In the early edition of TGB 1990 standard building wood, with an allowable stress of 7 N/mm², is referred to as strength class K17 with a represented bending strength of 17 N/mm², in later editions this changed to C18 with a characteristic bending strength of 18 N/mm². For construction wood, the norm uses 10 N/mm² as allowable stress. (Reserve available: yes, the values that are used in design codes might be too conservative)

There is a high variety of beam sizes required to fulfill the deflection requirements. This is due to the different approaches of creep. However complying with the demand is not mandatory. (Reserve available: uncertain, depends on the used norm and if the deflection requirement is taken into account.)

In conclusion, no design code is too conservative.

2.7 STRENGTH GRADING THROUGHOUT THE YEARS

Wood is a natural product, the quality depends on several factors like species and growth conditions. This causes for every tree to have unique properties which makes grouping by strength harder. In construction strength classes are introduced for designing safe and economic structures. The classes in the past were based on visual characteristics. In 1960 machine grading was introduced to increase the accuracy of strength grading. Figure 2-21 shows the development of visual grading.

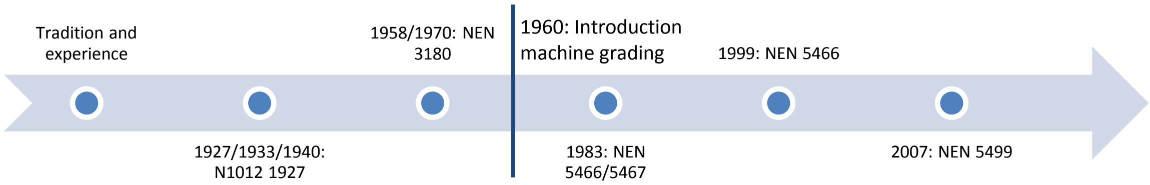


Figure 2-21: Visual grading norms throughout the years

The N1012 prescribed quality demands for timber used in houses but did not involve material properties, see appendix E.3 for the requirements. A grading process based on visual aspects was firstly considered in the NEN 3180 in 1958 and is only applicable for European softwood. The quality demands allowed for two strength classes for structural purposes: standard building wood and construction wood. The properties of these strength classes were already used in material tables in the design code of 1933. In 1983 a new norm for pine and spruce came on the market as respectively NEN 5467 and NEN 5466. Four strength classes are distinguished from A to D where B equals construction wood and C standard building wood. The classes stayed the same until the NEN 5499 from 2007 which uses also four classes but now T3 (C30), T2 (C24), T1 (C18) and T0 (C14). Standard building wood is classified as C18 and construction wood C24.

A distinction must be made between strength grading and appearance grading. For instance class A in the NEN 5466/5467 was a special class which was only used for timber with very high appearance demands. The appearance grading is also used for non-structural timber.

Different defects of wood are assessed during visual strength grading: slope of grain, ring growth width, resin bags, presence of heart wood, curvature, knots, mechanical damages, rot, twisting, discoloration due to fungi, cracks, wane and insect holes. A comparison between the visual grading norms shows if a period was too conservative. However this can only be done in a qualitative way since some demands were merged together while others are more specific distinguished.

Comparison of the visual grading standards

Three editions of the NEN 3180 came out, each edition became more elaborated but values did not change. Early versions of the NEN 5466 noted some aspects as limited allowable. The version of 1999 was more specific and addressed a value to the limited parts. Comparing this to the NEN 5499 shows varying results. Aspects like slope of grain, insect holes and geometric defects became stricter while discoloring and resin related defects became less strict.

It can be concluded that the differences in strength classes over the years is the main reason why visual grading norms cannot be compared directly. A better way to compare the norms is by actually performing the grading methods on timber beams which is done in chapter 3.5.3.

2.8 DETERIORATION OF STRENGTH

Wood is an organic material which is sensitive to time dependent processes that reduce the strength. Age is not necessarily a strength-reducing factor but is associated with strength-reducing processes. The level of degradation determines if interventions need to be taken. The degradation process normally starts immediately after completion of the structure. Four degradation mechanisms can be distinguished for timber (van de Kuilen & Montaruli, 2008):

- Mechanical degradation
  The main mechanical degradation process is called duration of load effect which means that the strength of a structure is slowly reduced when long term stresses are present. Norms from 1972 and later considered this duration of load phenomenon separately and gave a reduction factor on the load or resistance. Damage is only expected when the loads are short and high, a permanent load is too low to cause actual damage (van de Kuilen J. , 1999). Cracks need to be examined closely and their cause needs to be determined. Ruptures at various angles to the grain might indicate mechanical damages.
- Physical degradation
  This degradation process not only affects the appearance of the structure but may also lead to a lower structural safety. Four processes determine the physical degradation: High temperatures (fire), wind, UV radiation or drying. For roof structures only drying is expected to cause degradation. Different parameters related to cracks (climate conditions, moisture content, etc.) determine the decreased stiffness. Attention should also be paid to the shear strength which is reduced due to cracks. The bending strength is less affected due to the internal lever staying intact.
- Chemical degradation
  Timber has a high resistance against a wide range of chemicals. Alkaline solutions are destructive for wood fibers. It is not likely that roof structures will be subjected to chemical degradation. However when the beams can get wet, due to failure of the roof covering, a phenomenon called nail sickness can occur. Corrosion of a metallic fastening (metallic salt) can lead to chemical decay by a fixation with the cell walls, but this has only a local effect (Domone & Illston, 2010).
- Biological degradation
  Degradation due to biological attacks is without a doubt the most important mechanism. Three groups can be distinguished: insects, fungal and bacteria. The key parameter for these wood attacks is the moisture content. This was already addressed in the first timber norm around 1926. Roof structures with bad insulation and no ventilation have a high risk. As noted before, the connection of the beam with the stone wall is also a critical point. The main concern is the reduction in weight which results in a reduced strength of a timber beam. The norms take the biological durability into account by means of service classes. Other European standards make use of use classes (formerly hazard classes). EN-350 2 provides a list with the natural durability depending on the species. The common species are shown in table 2-4. Note that the table implies heartwood, sap-wood is always not durable (class 5).

Species	Natural durability fungi	Natural durability insects
North European spruce	Slightly durable (class 4)	Susceptible
Fir	Slightly durable (class 4)	Susceptible
Pine	Moderately to slightly durable (class 3-4)	Susceptible

Table 2-4: Natural durability wood species

Appendix F gives more information about these processes. Information on the history of the structure, the classification and the degradation processes can be used as input for flow charts that analyses the remaining lifetime.

2.9 CONCLUSION PRELIMINARY EVALUATION

An initial check (see appendix A) already showed that the roofs may or may not fail depending on different parameters. Based on this preliminary research there is no reason to assume that the current roof structures no longer have a residual service life left. Older structures are renovated therefore the roof structures might not always be that old. To give advice on the maximum allowable extra ballast a more detailed investigation is needed to reduce the uncertainties. The following can be concluded about the past:

Rotterdam:

- The criteria year of construction seemed to work well for categorizing roof structures. The grouping can be done in a new system which combines the history with the periods of active norms:
    - Group 1: Pre-war roof structures that are not renovated
        - 1A: Before first norms (<1930)
        - 1B: After first norms (1930-1944)
    - Group 2: The reconstruction period (1945-1969)
    - Group 3: The urban regeneration period (1970-1999)
    - Group 4: New buildings (>2000)
- The largest group of roof surface are pre-war houses. Almost all pre-war buildings have been renovated in the 80's. Only one case was found where the renovation did not include the roof structure, this means that the original roof structure from 1923 is still in place. Modern structures have more accurate strength values and can anticipate the extra load.
- Timber roof structures are mainly found in pre-war houses. There are less post-war houses with a timber roof than expected.
- The new buildings (group 4) have a high quality, defects in the roof structure are not expected to be present unless a leakage occurred.

Literature:

- Traditional building methods were mostly used pre-war. After the war modern techniques with prefab elements are used. Roof beams are usually simply supported by a wall.
- Occasionally there are gravel or tiles present to keep the roof covering in place. This weight can be reduced from the dead load of a green roof.
- A cold roof causes a high humidity under the decking, extra attention should be paid to these roofs.
- It seems that modern standard beam sizes came on the market around 1980. Therefore whenever a roof is renovated, the beam size on a drawing can indicate if new or old beams are present without visual inspection.
- Spruce, fir and pine are the most common wood species for construction in the Netherlands.
- Defects that endanger the structure can only be described here and therefore site inspection is always needed to assess if the structure is still reliable.
- Older structures are checked with the current regulations for new structures according.
- Attention should be paid to monitoring the timber structure when a green roof is present. Also failure of the waterproof covering and root resistance layer should be spotted and fixed shortly after failure.

Green roofs:

- A saturated green roof is seen as a permanent load.
- Possibly more strength present than designed for due to cooperation between beam and decking.
- Attention should be paid to aspects like falling over the edge and monitoring of the structure.
- The bearing walls can collapse due to increasing tension forces in the wall when a green roof is built.
- The foundation needs extra attention when making an intensive green roof.

The norms:

- The norms showed small differences in the needed cross section modulus to fulfill the strength requirements.
- This was not the case for the required stiffness, the effect of creep was not taken into account before the TGB 1972. These houses may therefore not fulfill the modern requirements (if this is demanded), attention should be paid to the deformations when the extra ballast from a green roof is present.
- No norm stands out for being too conservative.
- Reserves should be searched in the real strength values. Currently there are more strength classes than in the past. The classification in the past was purely based on visual grading after which they are assigned to one of the two strength classes. Previous grading norms were stricter and therefore the older beams which were graded as standard building wood might nowadays be assigned to a higher class than C18.
- In the considered cases, the maintenance load was always governing. Depending on the dimensions of the roof, the snow load might be governing. Also pay attention to water and snow accumulation.
- Fulfilling the deformation requirement is not mandatory, therefore beams that are designed for this requirement might have extra strength capacity.

Deterioration of strength:

- During inspection on site attention should be given to critical locations (supports, chimneys, gutters). These locations are vulnerable for moisture related problems. Furthermore the amount of cracks and their depths should be measured along with their cause.
- The main problem with roofs is the biological degradation. This is often caused by bad insulation of the roof covering or when no cavity is present. A cavity is obligatory since 1960.
- The ends of the members are usually in a notch of the wall. Therefore the end is hidden from visual inspection while this location proves to be vulnerable for high moisture contents and thus also for biological attacks.
- Almost all houses in Rotterdam are row houses. The beams in the roof structure span in the shortest direction which is the longitudinal direction of the row. Therefore, moisture related problems are only expected to be present where beams span to an outer wall. This was the case in Rusthofstraat where a rotten beam had to be replaced.
- Service life modeling is needed to determine the residual lifetime of a decayed beam.

A good solution is to determine the current strength and use this value for the ULS check because it is not always clear what procedures were used in the past. To gain more strength a more precise calculation procedure or a thorough investigation in the design procedure is necessary. Also inspection on site is always needed to assess the damage that is caused by a degradation mechanism. Destructive and non-destructive testing methods must be researched to find the best solution for assessing the strength on site. An initial idea for a strengthening solution is by making sure the beams work together with a new structural material.

Gain in strength	Possible weakening
Gravel can be removed	Bad insulation, high humidity
(Possible) cooperation between beam and decking	Degradation mechanism
Members are sometimes designed for deformations	Beam ends in contact with outer walls
Higher strength beams can be present due to 5% value	High tension forces in wall
Capacity left in the unity check	Overloading due to snow/water accumulation
Control the load (like walkable paths)	Bad foundation
Allowed reduction factors are not always used (TGB 1972)	

Table 2-5: Overview of positive and negative factors that influence the strength

3. THE CURRENT STRENGTH OF A TIMBER ROOF STRUCTURE

This chapter researches the residual capacity of timber beams. In order to extend the lifetime of the timber elements it is necessary to proof the remaining capacity is sufficient. Different criteria, like time, target safety levels, economics and political preferences determine the decision (JCSS, 2001). *To achieve a safe and economical structure a thorough evaluation of the existing elements is required. Literature already reveals methods for assessing older structures. Guidelines, standards and studies can be found to ensure structural integrity over a specified residual service life.*

During the research, 13 beams were obtained from an ongoing demolishment. 10 of these members are of a renovated roof structure from 1983 while the original structure was from 1923. The other 3 members are from another building where the original structure of 1923 was still present. Their service life is described in appendix G.1. The members were placed in a climate controlled room until experiments were needed. The climate conditions here are 20°C with a relative humidity of 65% which is in compliance with the EN 408 for testing pieces. Different (non)destructive tests can be performed on these members to find a suitable method for in situ measurements.

The first paragraph explains the calculation procedure of an engineer that needs to check an existing structure with new loads. Here it is assumed that the real strength is known. The second paragraph clarifies methods that can be used in situ when the current strength is unknown. Some of these methods are used on the obtained beams in paragraph 3.5. The results from this research can then be used in paragraph 3.6 as example for future assessments.

3.1 NEN 8700

The “NEN 8700 – Assessment of existing structures in case of reconstruction and disapproval” gives applicable rules for the evaluation of an existing structure. The Building Act of 2012 requires to use the NEN 8700 for changes in existing structures. Table 3-1 gives an overview of the chain of regulations.

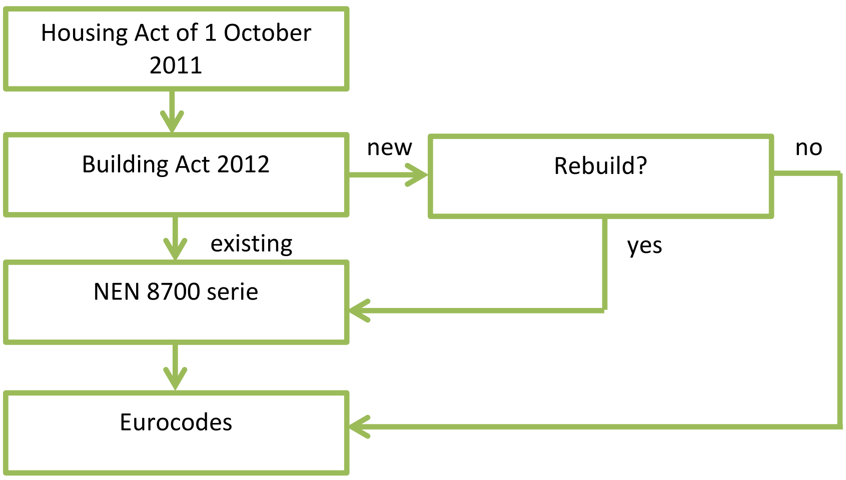


Table 3-1: Chain of regulations (de Vries, 2012)

The first thing that should be noted is that making a green roof is according to the norm a rebuilding. This means a physic interference of the structure along with a change in load. The NEN norm for coping with existing timber structures was still in development during the writing of this thesis. First the service life of the structure must be set in order to determine the required safety. For a rebuild with CC2 this means that its residual life time should not end before the original designed reference period with a minimum of 15 years. Thus all buildings before 1965 must comply with the minimal residual life time of 15 years. The other building components, that are indirect effected, are subjected to rules for the state of the structure. This means that a certain performance level, to resist the new loads, is needed.

The NEN 8700 provides the following steps to be taken for the assessment of existing structures:

1. Visual/global inspection, the result can indicate that no further research is necessary.
2. When damage is detected, an explanation must be given. The calculations, properties, loads and mechanisms must be checked.
3. Determining the current state and reliability.
4. Special inspection and advanced calculation procedures must be executed when insufficient safety is obtained.
5. The decision is based on the costs.

When the existing structure does not fulfill the Eurocode requirements other options can be used:

a. Reduce the reference period.
b. Based on actual use.
c. Adjust the use.
d. Adjust the safety margin.
e. Adjust the strength.

Furthermore it is mentioned that the SLS can be based on the actual behavior and not on the indirect requirements of the Eurocode. The rejection of a construction is purely based on the ULS. This makes more economic structures possible. Advanced calculation models are often used for existing structures to prevent high costs. These models are a better approximation of the reality than conservative models. The uncertainties in the advanced models can be evaluated using conservative models or (non-) destructive tests.

The calculation procedure makes a distinction between two rejections levels:

- Rebuilding level: Minimum level of structural safety when checking the design of a rebuild.
- Reject level: Minimum level of structural safety with enforcement by the competent authority.

The reject level is less conservative and is only used at the end of a reference period. Appendix G.2 provides an overview of the design aspects according to NEN 8700. Note that the load on existing structures comes from the NEN 8701. This norm refers to the values of the EN-1991 and gives possible reductions on the load.

It becomes clear that some strength is gained in the load factors and the variable load of maintenance. The maintenance load can be reduced by setting requirements (e.g. no heavy persons/material allowed or only make use of specific walkable paths) or determining a more realistic lower load. Note that the minimum distributed load is 0,56 kN/m² due to snow and cannot be reduced. In appendix E.2 calculations with the Eurocode were performed with a governing load of 2 kN that came from the construction phase. This can now be replaced with a smaller load of 1,5 kN. It is assumed that the reduction of the variable load is 10%. Table 3-2 shows a noticeable result, Schiedamsesingel now needs a lower section modulus (see also table E-2).

ULS	Schiedamsesingel	Kerkhofstraat	Van Drimmelenstraat
Governing variable load	1,35 kN in middle	1,35 kN in middle	1,35 kN in middle
Load combination	0,580 * 1,3 * 0,80 = 0,60 kN/m 1,3 * 1,35 = 1,76 kN	0,605 * 1,3 * 0,60 = 0,47 kN/m 1,3 * 1,35 = 1,76 kN	0,680 * 1,3 * 1,40 = 1,24 kN/m 1,3 * 1,35 = 1,76 kN
Moment	1,82 kNm	3,07 kNm	4,46 kNm
Maximum stress	12,46 N/mm²	12,46 N/mm²	12,46 N/mm²
Minimal section modulus needed **Used section modulus**	**146067 mm³ (44x150 mm)** **187500 mm³ (50x150 mm)**	**246388 mm³ (63x160 mm)** **500000 mm³ (75x200 mm)**	**358237 mm³ (75x175 mm)** **533333 mm³ (80x200 mm)**

Table 3-2: Three cases compared with NEN 8700

The same calculation can be performed with the presence of a green roof. Here an extensive green roof is used because this has the lowest weight and complies with the needed buffer.

ULS	Schiedamsesingel	Kerkhofstraat	Van Drimmelenstraat
Governing variable load	1,35 kN in middle	1,35 kN in middle	1,35 kN in middle
Load combination	0,580 * 1,3 * (0,80 + 1,0) = 1,36 kN/m 1,3 * 1,35 = 1,76 kN	0,605 * 1,3 * (0,60 + 1,0) = 1,26 kN/m 1,3 * 1,35 = 1,76 kN	0,680 * 1,3 * (1,40 + 1,0) = 2,12 kN/m 1,3 * 1,35 = 1,76 kN
Moment	2,56 kNm	4,99 kNm	6,68 kNm
Maximum stress	12,46 N/mm²	12,46 N/mm²	12,46 N/mm²
Minimal section modulus needed **Used section modulus**	**205457 mm³ (63x150 mm)** **187500 mm³ (50x150 mm)**	**400482 mm³ (63x200 mm)** **500000 mm³ (75x200 mm)**	**536116 mm³ (75x225 mm)** **533333 mm³ (80x200 mm)**

Table 3-3: Three cases compared with NEN 8700 plus the weight of an extensive green roof

Note that Kerkhofstraat and Van Drimmelenstraat will meet with the strength requirements. No structural adjustments are needed but inspection must show that the strength is still sufficient.

3.1.1 DISCUSSION NEN8700

One can be skeptical about the NEN 8700 because it allows a higher variable load. Besides the required level of safety is not a straightforward value but a reasoned target. (Vrouwenvelder, Scholten, & Steenbergen, 2011) note the ideas behind the NEN 8700 which is briefly discussed below.

Philosophy for new structures

The current active norm is based on the reliability index β which is in direct relation with the chance of failure P. For new build with consequence class 2 (CC2) the β value is 3,8 with a reference period of 50 years. The corresponding chance of failure is around 10^{-4}. CC is chosen based on economic and human safety considerations. Thus CC2 and its corresponding safety level are applied for new houses with 4 layers or more.

Philosophy for existing structures

The result of using the β-value is that during a reference period of 50 years, each individual year has a lower failure chance than a reference period of 1 year. This is based on the investment costs in relation with durability. Thus the minimal reference period of 15 years for rebuild does not reduce the reliability index. The reduction is found in the extreme value of the governing load which is smaller over a shorter period. Also the NEN 8700 finds it reasonable that a rebuild safety level does not need to comply with newly build because of high costs for the remaining life-time. Therefore the lower limit β becomes:

$\beta_{rebuild} = \beta_{new} - 0,5$

At last it is debatable if the rebuild level applies only to the adjusted structure or the complete structure. The latter makes the calculation procedure easier, however, the Housing Act requires the safety level only to be applied to the adjusted parts.

Discussion

It is undefined what consequence class should be used when making a green roof. It is certain that the roof structure belongs to the rebuild level but other parts of the building are uncertain. In accordance to the above philosophies an adjustment of only the roof structure, and thus only one build layer, is classified as CC1 while the complete structure could be CC2. It is plausible that if CC1 wants to be applied progressive collapsing should not occur. NEN 8700 gives additional requirements for upgrading, the weight must be less than 2 kN or 0,3 kN/m² and the extreme variable load in the combination may not be from people, furniture or finishing. The advantage of a lower class is a lower reliability index and thus a lower partial factor. CC1 is divided into A (no chance in loss of human lives) and B (small chance in loss of human lives). It is assumed that the area under the roof is used as a living space and not for storage hence CC1B is needed.

Consequence class	Minimum reference period	Newly build β_{new}	Rebuild $\beta_{rebuild}$
CC1B	15 years	3,3	2,8
CC2	15 years	3,8	3,3

Table 3-4: Reliability indexes (β) for different CC classes (Vrouwenvelder, Scholten, & Steenbergen, 2011)

To check if CC1B is allowed, the worst case scenario is considered. This is when the full weight of the roof is present on the floor beneath. Floors are designed to withstand higher loads than roofs:

- TGB 1955: 1,5 - 2 kN/m²
- TGB 1972: 1,5 kN/m²
- TGB 1990: 1,75 kN/m²
- Eurocode: 1,75 kN/m²

These loads represent furniture and persons walking, dancing or stomping. It is not expected that both the roof and the floor have the extreme value. Table 2-1 showed that a value between 1 kN/m² and 3,4 kN/m² can be expected on the roof when the soil is fully saturated. Add to this a permanent load of circa 1,0 kN/m² and it can be seen that the floor would not hold the roof weight. Besides the variable load is not included and a part of the variable load for floors is present due to (permanent) furniture. Therefore CC1B is not allowed.

Secondly, the partial factors in NEN 8700 are reduced in all CCs for the fundamental combinations of strength. Apparently higher loads are allowed when the structure is older. The reason is that the β-value is different and thus the reference period is smaller than originally designed for. In other words, when the reference period is smaller, the chance of an extreme load occurring is decreased. This raises the question if 15 years after making a green roof the existing structure is checked, will the safety be checked on rejection level which allows even more loads? Or is demolition necessary? (Vrouwenvelder, Scholten, & Steenbergen, 2011) are unclear about this point. However, keep in mind that only the loading part is adjusted and not the strength part. The partial factors for rebuild level are determined by taking the average and rounding up of the rejection level and new level. This seems to be a conservative approach which can be debatable.

At last the true permanent load can be measured and the new load can be controlled. This means a lower uncertainty and thus a lower load factor. Using the NEN8700 the load factor for the permanent load would go from 1,2 to 1,15. This makes sense but the small decrease will not lead to large profits. The load factor for the variable load goes from 1,5 to 1,3. This reasoning is difficult to support because it is unclear what happens after the new reference period ends. Also the load from weather conditions are hard to predict.

In this thesis reducing the reference period is not recommended but seen as an additional option that the building act gives. It is not legally determined if the change of the safety level is allowed. This will lead to discussions with “construction and housing inspection” in the future because roofs are less safe. An alternative option is to consider the existing roof as a new roof because the NEN8700 is the minimum required safety level.

3.2 IN SITU METHODS FOR GRADING TIMBER

When older timber structures have to be assessed because of the change in function or action, the first activity is a preliminary inspection. The objective of inspection is to gain reliable data for a structural engineer to assess the structure. This data needs to include the quality of the timber (physical and mechanical properties), the level of decay or damage, the risk of decay or damage in the future and the remaining effective cross section. Attention is paid to important aspects like cracks, fungi, holes from insects, the supports and the decking. Simple tools like a hammer, screwdriver or drill in combination with visual defects are used in this phase. The purpose is to have a first indication if the timber has a residual lifetime and can be reused. Visual strength grading, evaluation of critical sections and an estimation of internal or invisible decay is done to check the residual functioning portion (Ceccotti & Togni, 1996). If after inspection no solid conclusion can be given than the uncertainties need to be evaluated in a detailed inspection.

A detailed inspection makes use of advanced tools to measure the material properties or the level of decay. This data is then used to assess the residual lifetime. The most efficient method to determine the properties is destructive testing but this requires time and specimens which is not always possible. Therefore non-destructive tests (NDT) or semi-destructive test (SDT) are necessary to obtain the results. The correlation with the destructive testing indicates if the NDT results are acceptable. More advanced tools that make use of stress waves or radiations are used when more precise results are required. The accuracy of the result is always device and human dependent.

Appendix G.3 gives an extensive review of methods that can be used in situ. Important parameters to be measured for timber assessments are:

- Visual characteristics (dimensions, knots, slope of the grain, decay, ring shakes, etc.)
- Mass
- Moisture content (risk of decay)
- Dynamic/static modulus of elasticity
- Bending strength

Table G-2 shows the correlation of common methods with the material property. Note that the Resistograph, Pilodyn and stress waves are popular methods. Considering the different results reported from varies researchers it becomes clear that some ND methods give a direct good result. The coefficient of determination (R^2) can give an indication about the reliability of the prediction. Very high values for timber are not expected due to its inhomogeneity. (Faggiano, Grippa, Marzo, & Mazzolani, 2009) gives a table for the ranges:

Range	Correlation
$0 < R^2 < 0.1$	Low
$0.1 < R^2 < 0.3$	Moderate
$0.3 < R^2 < 0.5$	Medium
$0.5 < R^2 < 0.7$	Good
$0.7 < R^2 < 1$	High

Table 3-5: Ranges of R^2 (Faggiano, Grippa, Marzo, & Mazzolani, 2009)

A low R^2 value does not necessary mean that the method is bad. The effectiveness strongly depends on the conditions of the measurement. The measurement in the longitudinal and transversal directions can give different results. Clear wood shows better correlations than structural timber but these results are not reliable enough for calculations.

It can be presumed that the three reference properties can best be determined by the Resistograph (for bending strength and density) and stress waves (for modulus of elasticity).

Furthermore several researchers showed promising results when different methods were combined.

3.3 GOAL OF EXPERIMENTS

Although there are several ways of gaining some extra strength (a closer look in the load factors, an advance finite element model or probabilistic modeling), the used method in this thesis is considered to be the most effective. The idea is as follows, freshly sawn structural timber is graded into a strength class as discussed in appendix E.5. This means that a small amount of the graded timber does not have to meet a certain limit strength. A timber batch should meet the requirements on average value, here a representative number of beams are tested. The material factor must than remove the uncertainty. Figure 3-1 shows the distribution of a strength class. Nowadays the 5% lower probability value is chosen as the limit value.

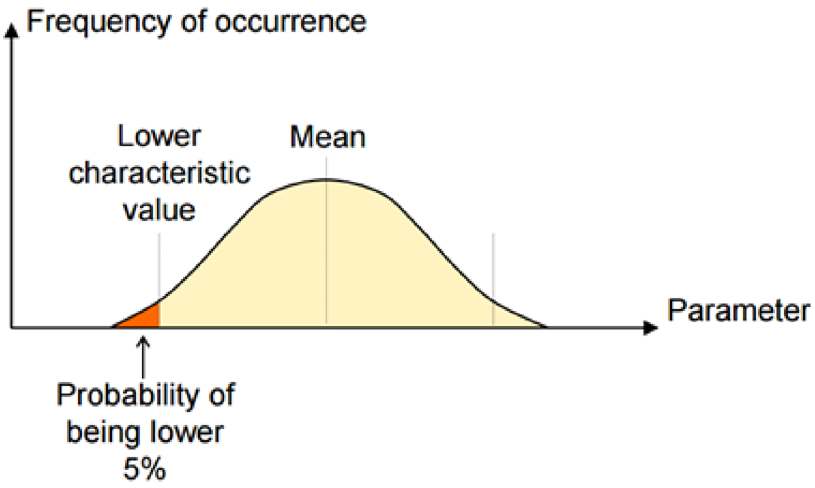


Figure 3-1: 5% value of a distribution

This way of strength grading allows for beams to be stronger than the characteristic value. The experiments are aiming to predict the actual strength. An extra challenge is to be able to predict the bending strength without demolishing the roof. There might also be more strength available because of stricter visual grading norms in the past.

3.4 THE EXPERIMENTS

Timber properties can vary significantly between different members of a batch. Also within one member the properties are not uniform. The bending strengths correspond to the 5-percentile failure of its probability distribution or before 1991 to a strength failure probability of 1/1000. An easy and commonly used way to assign a class to a member in situ is by making use of visual grading standards. The experiments planned give more information about the member and can be used for better prediction of the real strength and stiffness.

Not all the methods from paragraph 3.2 can be used for this thesis. Factors like budget, time, non-availability of the equipment and lack of experience restricts the options. The following methods can be used and are chosen because of their ease of use, available equipment and efficiency:

- Visual grading
- Dynamic stiffness measurement
- Resistance measurement
- Four point bending test

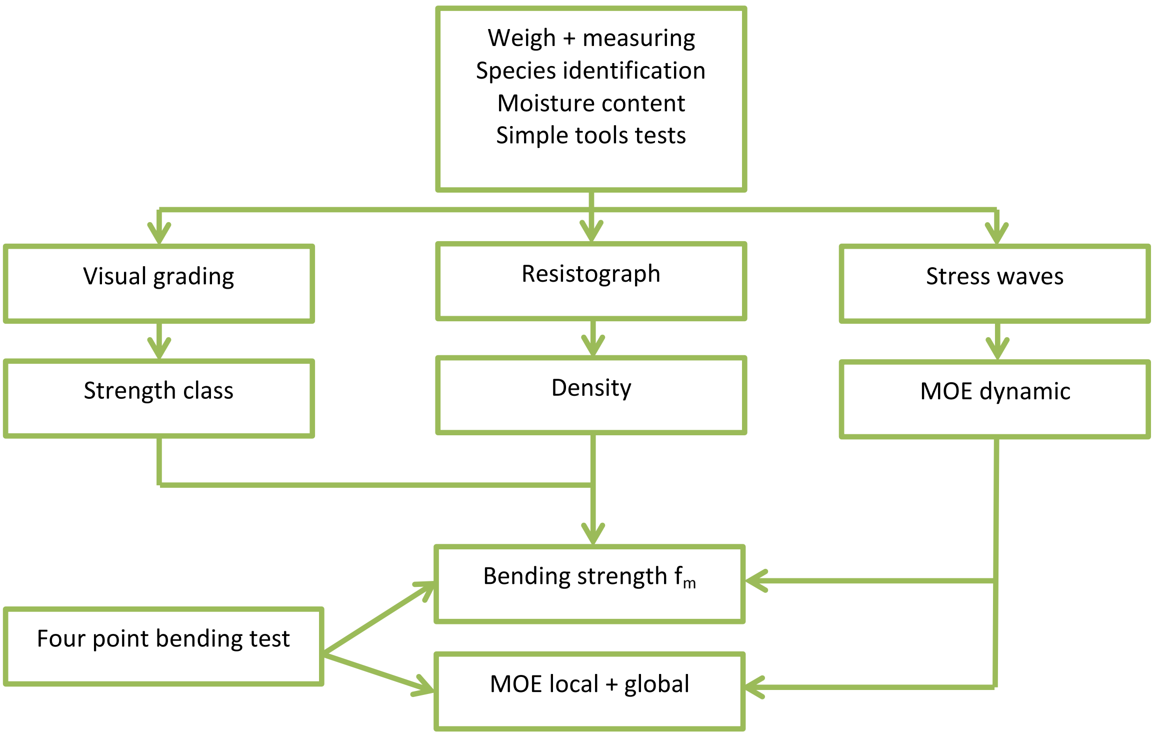


Figure 3-2: Strategy of experiments

Appendix G.4 gives more information about the plan. The batch consists of the following beams:

Beam ID	Taken from	Period of use	Species trade name	Dimensions (average bxhxl)	n
S	Kerkhofstraat	1983-2015	Spruce	76x194x4068	10
L	Kerkhoflaan	1923-2015	Spruce	91x242x4633	3

Table 3-6: Dataset of samples

3.5 EXPERIMENTAL RESULTS

This section describes the results of the performed experiments. Here a summary is given of the outcomes and noticeable results are described. Various authors concluded that the size of the members have influence on the strength. However this effect is mostly noticeable on smaller size timber. For structural timber (Ravenshorst G. , 2015) concluded that no depth effect needs to be taken into account for the bending strength. In EC5 a size modification factor is allowed for beams smaller than 150 mm, it is not expected that smaller depths than 150 mm are used for roof structures.

3.5.1 MOISTURE CONTENT

The measuring devices showed moisture content values between 12-14% as was expected. The two measurements on ¼ and ¾ of the length showed lower values than the measurement in the middle because water can move easier to an end. Also the FMW meter, which makes use of a magnetic field, showed lower results than the penetrating meter. After destructive testing a sample was taken close to the fracture for measuring the wet and dry weight. The true moisture content is on average 12,5%. This means that the FMD meter gives a better approximation and the FMW underestimates the moisture content.

S1 Heartwood side				**S1 Sapwood side**			
	FMW [%]	FMD_{2cm} [%]	$FMD_{3,8cm}$ [%]		FMW [%]	FMD_{2cm} [%]	$FMD_{3,8cm}$ [%]
Left end	10,6	11,1	12,5	**Left end**	8,3	12,8	12,8
Middle	11,0	11,6	13,1	**Middle**	9,9	13,0	13,3
Right end	10,3	11,7	13,0	**Right end**	11,8	12,2	13,0
Average	10,6	11,5	12,9	**Average**	10,0	12,7	13,0
S2 Heartwood side				**S2 Sapwood side**			
	FMW [%]	FMD_{2cm} [%]	$FMD_{3,8cm}$ [%]		FMW [%]	FMD_{2cm} [%]	$FMD_{3,8cm}$ [%]
Left end	10,3	12,8	12,6	**Left end**	10,4	12,6	11,7
Middle	11,0	13,7	12,9	**Middle**	10,3	13,1	12,5
Right end	11,3	11,7	12,6	**Right end**	10,3	12,8	12,2
Average	10,9	12,7	12,7	**Average**	10,3	12,8	12,1
S5 Heartwood side				**S5 Sapwood side**			
	FMW [%]	FMD_{2cm} [%]	$FMD_{3,8cm}$ [%]		FMW [%]	FMD_{2cm} [%]	$FMD_{3,8cm}$ [%]
Left end	11,3	13,3	13,1	**Left end**	11,0	13,1	13,0
Middle	11,1	13,7	13,1	**Middle**	10,5	13,4	13,2
Right end	11,5	13,7	13,1	**Right end**	10,7	13,7	13,0
Average	11,3	13,6	13,1	**Average**	10,7	13,4	13,1
L1 Heartwood side				**L1 Sapwood side**			
	FMW [%]	FMD_{2cm} [%]	$FMD_{4,2cm}$ [%]		FMW [%]	FMD_{2cm} [%]	$FMD_{4,2cm}$ [%]
Left end	11,1	13,5	13,6	**Left end**	10,7	13,6	14,3
Middle	11,5	13,1	14,1	**Middle**	10,7	13,7	13,7
Right end	11,2	13,5	13,8	**Right end**	10,5	13,5	14,1
Average	11,3	13,4	13,8	**Average**	10,6	13,6	14,0

Table 3-7: Measured moisture content using the FMW and FMD

The true moisture content:

Member ID	Moisture content [%]	Member ID	Moisture content [%]
S1	12,6	**S8**	12,1
S2	12,5	**S9**	13,6
S3	12,7	**S10**	11,9
S4	12,6	**L1**	11,9
S5	12,6	**L2**	11,8
S6	12,6	**L3**	12,0

Table 3-8: True moisture content using oven dried method

The moisture content in situ is expected to be higher. This will influence the timber (dynamic) properties. (Unterwieser & Schickhofer, 2011) concluded that the dynamic properties are linear dependent on the moisture content below fiber saturation point (FSP). The dynamic MOE will decreases when the moisture content increases. Above FSP, the dynamic MOE stays nearly constant because of an increase of moisture the density increases but the sound velocity decreases.

3.5.2 DENSITY

Measuring and weighing were the first steps for each beam. With this information the average density can be calculated:

Member ID	Length [mm]	Thickness [mm]	Width [mm]	Weight [kg]	Average density [kg/m³]
S1	4073	194	76	26,26	436
S2	3970	192	75	28,86	502
S3	4090	193	76	29,08	482
S4	4081	195	74	26,56	454
S5	4085	192	76	28,28	475
S6	4090	192	76	28,96	486
S7	4074	194	76	26,42	438
S8	4066	195	77	27,06	442
S9	4075	195	77	29,44	482
S10	4075	196	76	27,82	456
L1	4611	244	98	46,12	419
L2	4653	240	80	40,48	452
L3	4635	240	95	48,04	455

Table 3-9: Dimensions and density of the timber beams

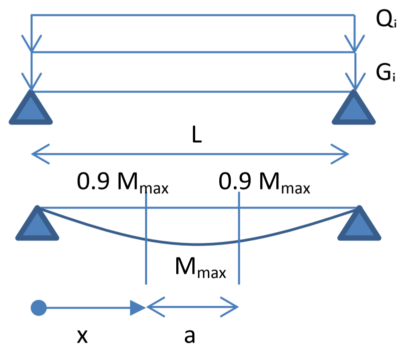
Batch ID	Mean [kg/m³] ($\bar{x}$)	SD (s)	n
S	465	23	10
L	442	20	3

Table 3-10: Distribution of the density

The thickness and width is taken as the average of three measurement points. Note that for strength class C18 the average density is around 380 kg/m³ and 420 kg/m³ for C24.

3.5.3 VISUAL GRADING

All defects were recorded on a paper and graded according the current standard for softwoods (NEN 5499). Special attention is given to the size and location of the defect. The visual grading norms do not take the way of loading into consideration which can result in a lower strength class. A green roof will cause a larger bending moment in the middle of the span. Therefore defects that are located in this region matter the most and some members get (locally) a higher strength class than originally graded. The region is defined as the area of 10% of the maximum moment:



$$q = Q + G \,; M = \frac{1}{2} * q * L * x - \frac{1}{2} * q * x^2$$

$$0.9 * M_{max} = \frac{1}{2} * q * L * x - \frac{1}{2} * q * x^2$$

$$0.9 * \frac{1}{8} * q * l^2 = \frac{1}{2} * q * L * x - \frac{1}{2} * q * x^2$$

$$x = \frac{1}{2} * L - \frac{\sqrt{10}}{20} * L$$

$$a = L - 2 * x \approx \frac{1}{3} * L$$

Figure 3-3: Region of highest bending strength

Some extra flexibility is needed to prevent rejecting of some members due to mechanical damage that occurred during demolition of the building. This concerns mostly broken parts near the ends or torn vessels. At last it is important to know the origin of other visible defects. Cracks are judged based on findings of (Fech, 1987), as described in appendix F.2.

The grading norms that needs to be used is determined by the geographic origin. Environmental aspects influence the grow, and thus the strength, of a tree. The strength of a specie should therefore be determined with the associated visual grading norm of the geographic area. However for in situ members the origin is often unknown. The NEN 5499 made use of the rules from the Scandinavian norm INSTA 142 which should cover the most important softwood countries. Destructive testing will conclude if the norm works for older unknown origins as well. For the 10 beams from 1983 the grading norm of 1970 (NEN 3180) is also used to know the intended strength class. It seems that the demands for the main defect, the knots, is eased in the newer norms. Members that are graded as standard building wood (C18) in the past are nowadays graded as C24. Other important defects like slope of grain and ring size are also eased. Notable is the demand "heart", the old norm does not allow enclosed heart while the current standard gives no demand. Table 3-11 shows an overview of the results. The complete evaluation can be found in appendix G.5.1.

Member ID	NEN 5499	NEN 3180	Conclusion (when knowing the way of loading and defects)
S1	C18	Reject	**Low class is determined by large knot near beam end, expected failure is due to a knot in the middle close to the bottom edge and a knot cluster→ C24.**
S2	C18	C18	**Large crack near the end. Expected failure is due to knot cluster of 77 mm around the middle → C24.**
S3	C24	C24	**Individual knot of 29 mm and 25 mm in the middle on the bottom can cause failure in bending → C24.**
S4	C18	Reject	**Some fungi in one knot is present but is allowed. One large knot was on top which is not visible insitu. Expected is failure in bending due to individual knots of 30 mm (bottom) and 35 mm (side) in the middle → C24**
S5	C30	Reject	**Reject because of wane. Expected to fail due to bending at knot cluster in middle → C30**

S6	C18	Reject	**Large knot cluster is most likely to cause failure in bending but is not in the middle→ C24**
S7	C18	Reject	**Heart is present. Cracks near support reduces shear capacity. Also curly grain is present here. Expected failure is due to individual knot of 35 mm on side in middle → C24**
S8	C18	Reject	**Heart is present. Crack on top and knot cluster will weaken the middle zone. Failure in bending around the middle → C24**
S9	C14	Reject	**Cracks over full length and heart is present. Expected failure is in middle due to individual knot of 43 mm on side → C24**
S10	C14	Reject	**Cracks over full length and heart is present. Expected failure is in middle due to individual knot of 40 mm on side → C24**
L1	C14	-	**Cracks over full length but with small depth. Expected failure is bending due to knot on bottom of 32 mm around the middle → C30**
L2	C14	-	**Cracks over full length but with small depth. Expected failure is bending due to knot on bottom of 25 mm around the middle → C30**
L3	C14	-	**Cracks over full length but with small depth. Expected failure is bending however no knots were found → C30**

Table 3-11: Results of visual grading

Two other visual grading norms (NEN 5466 and NEN 3180:1958) are used for grading two members. Noticeable is that wane gives different strength classes over the four visual grading norms. The important parameters like knots, slope of grain and growth ring width stay more or less the same between the NEN 3180 from 1958 and 1970. These aspect become more flexible with the later norms and thus the same beam graded in the past can get a higher strength grade today.

A remark must be made on updating the strength class. This was possible because there were no significant defects around the middle where the highest bending moment will occur. However this does not change the fact that the overall beam was graded in a lower strength class. When using the material properties for timber, the beam is considered as a homogeneous bar. Locally visually upgrading of the beam requires it to be considered as an inhomogeneous bar.

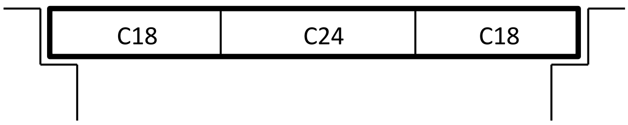


Figure 3-4: Local upgrading leads to inhomogeneous properties

At last a second student graded one member to indicate the human factor. The measured defects are often the same but their size and expected consequence are human dependent. It can be concluded that knots and growth rings (when pith is not visible) have the highest sensibility. Especially knot clusters where more knots are summed can lead to a different class. The slope of grain is less sensitive to human errors.

3.5.4 DYNAMIC STIFFNESS MEASUREMENT

The dynamic modulus of elasticity is determined by a relation of the wave speed and the density. All beams were first measured as a free vibration. The vibration meter was placed on the beam end were a hammer induced a wave. This gave a clear signal. Table 3-12 gives an overview of the results.

Member ID	Length [mm]	Density [kg/m^3]	Frequency [Hz]	$E_{dynamic}$ [N/mm^2]
S1	4073	436	566	9272
S2	3970	502	634	12715
S3	4090	482	649	13579
S4	4081	454	610	11249
S5	4085	475	615	12003
S6	4090	486	630	12909
S7	4074	438	634	11687
S8	4066	442	644	12125
S9	4075	482	581	10807
S10	4075	456	610	11260
L1	4611	419	576	11827
L2	4653	452	590	13641
L3	4635	455	561	12294

Table 3-12: The dynamic modulus of elasticity

The next tests were conducted in the laboratory to simulate an in situ situation and to study the influence of the surroundings. Appendix G.5.2 shows the different test setups. Three things can be concluded: hitting a screw on the side was the best method for introducing the wave, placing the meter on the bottom side gave good results and the surroundings (decking and wall) increase the wave speed. However the signal quality needs an experience user to evaluate if it is reliable.

Finally measurements were conducted on two garages. Here the beams can be accessed easily and are supported by two masonry walls and covered with wooden plates/planks, mastic and in one case gravel. The main difference with a house is the absence of insulation. However the results need to be interpreted manually because the quality in the frequency domain is bad. The disturbances make the signal more unreliable. One can make use of a prediction model as figure 3-5 to find the right frequency. Also see appendix G.5.2 for more information about the prediction and the in-situ measurements.

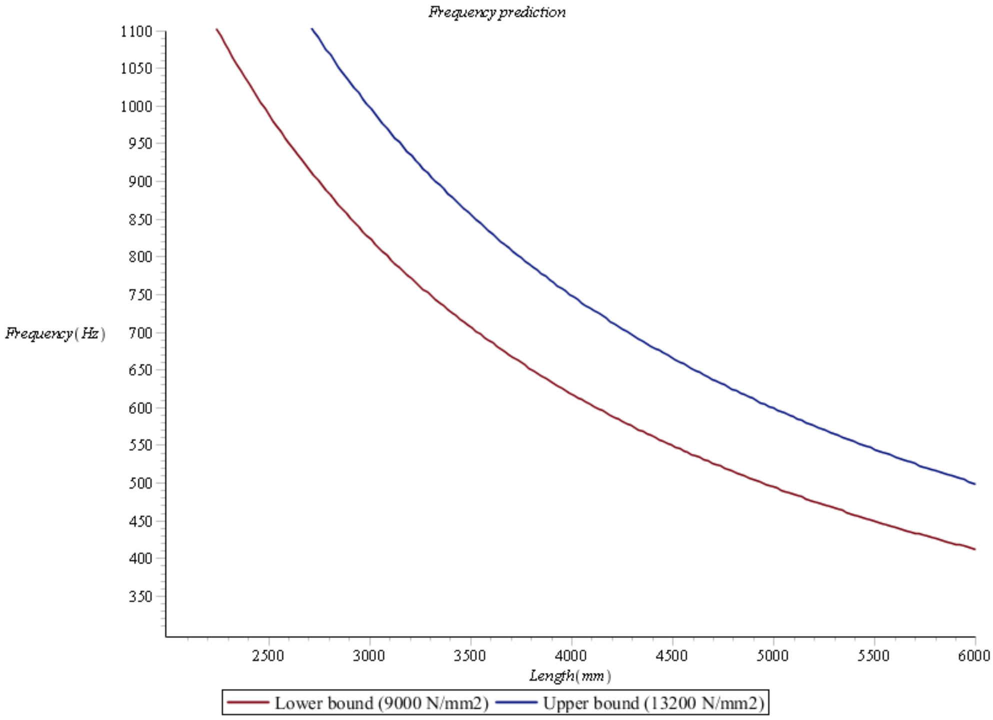


Figure 3-5: Graphical representation for predicting the frequency with C1 = 0,94 and C2 = 1,06

The frequency needs to be adapted because tests showed that the surroundings increase the frequency. Based on the result the following formula is proposed:

$$E_{dyn} = \frac{\sum_{i=1}^{5} 4 * l^2 * (\frac{f_i}{C2})^2 * (\sum_{j=1}^{3} \rho_j)}{15} \qquad \text{(Eq. 2)}$$

Where
ρ = the density [kg/m³]
l = the length [m]
f = the measured frequency [Hz]
C2 = a correction factor to take into account the surroundings. During testing a factor of 1,06 was found but it is expected that a higher correction factor is needed.

Whether the dynamic stiffness can be measured in-situ is still debatable. More research is needed to determine the reliability of the signal, the influence of the surroundings and the best location of the sensor. It is possible to use other NDT or SDT to check if the measured $MOE_{dynamic}$ is reliable.

3.5.5 RESISTANCE MEASUREMENT

A resistograph was used to drill 15 holes in a beam perpendicular to the growth rings. During drilling, the energy needed for rotating and feeding the needle were recorded along with the depth. The late- and earlywood become clearly visible (figure 3-6). Latewood is denser and requires more energy to penetrate.

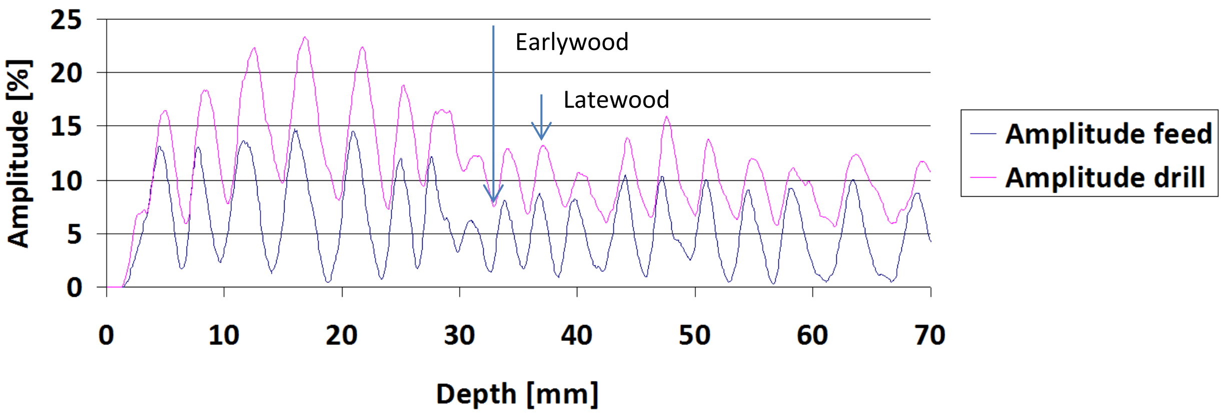


Figure 3-6: Graph made by using the resistograph perpendicular on the growth rings

After destructive testing cylindrical cores nearby the drill holes were extracted and its density was determined. Two settings of the resistograph were used with different drill speed and feed speed. The measured resistance is than correlated to the density. In figure 3-7 it can be seen that the drill resistance can best be used for predicting the density. The high R^2 value indicates a good correlation. Secondly, a faster drill or feed speed increases the correlation with the density.

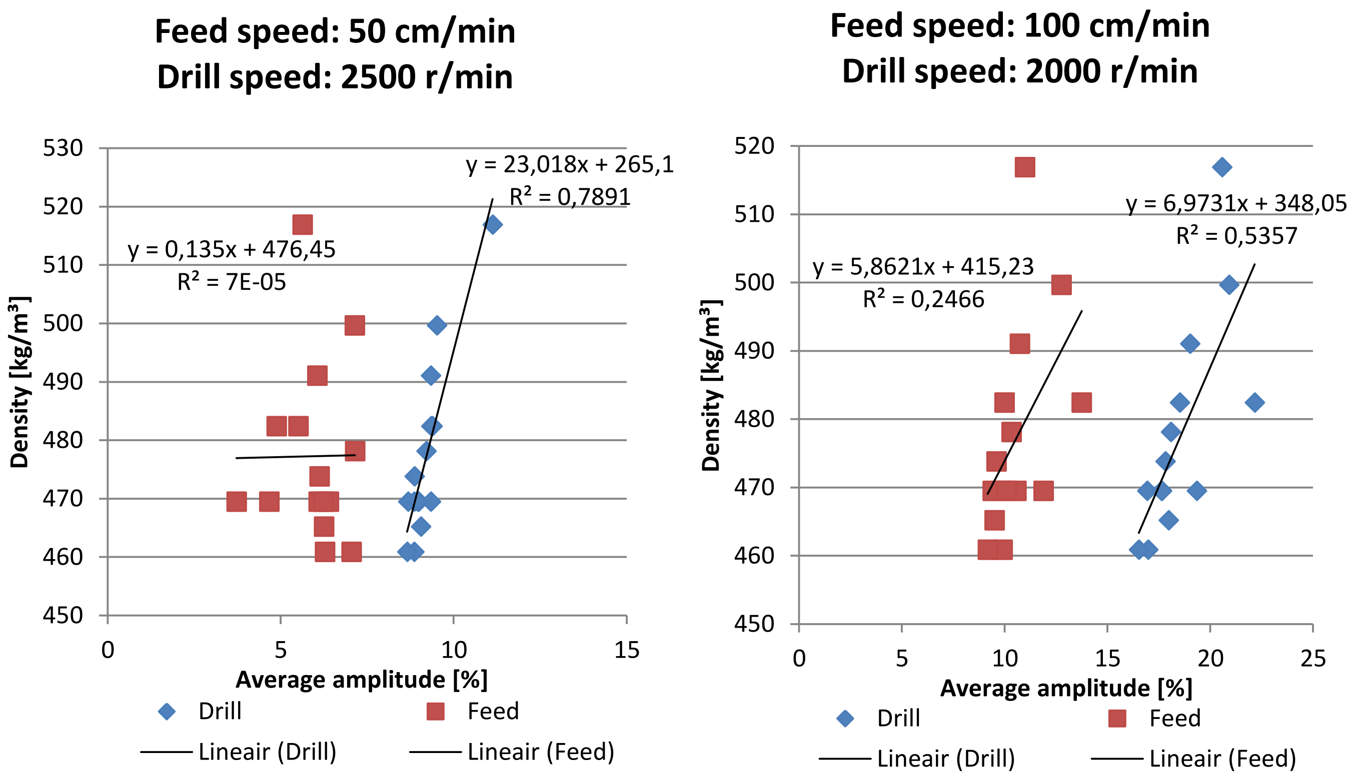


Figure 3-7: Correlation of the energy needed with the measured density

For the second test a random drill angle to the grain is chosen to simulate the in situ situation where the place of the pith might not be as clear. This results in a less regular pattern of energy use.

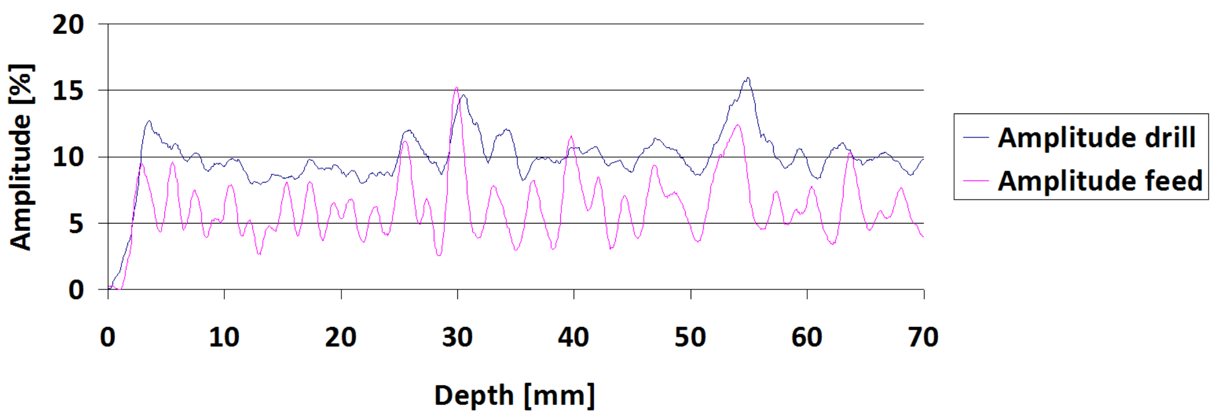


Figure 3-8: Graph made by using the resistograph with a random angle on the growth rings

The lower percentages are over a longer depth which indicates that the needle is driven under an angle through the early wood. This way of drilling influences the average energy needed and thus the correlation with the density. Figure 3-9 shows a low R^2 value which indicates a bad prediction behavior.

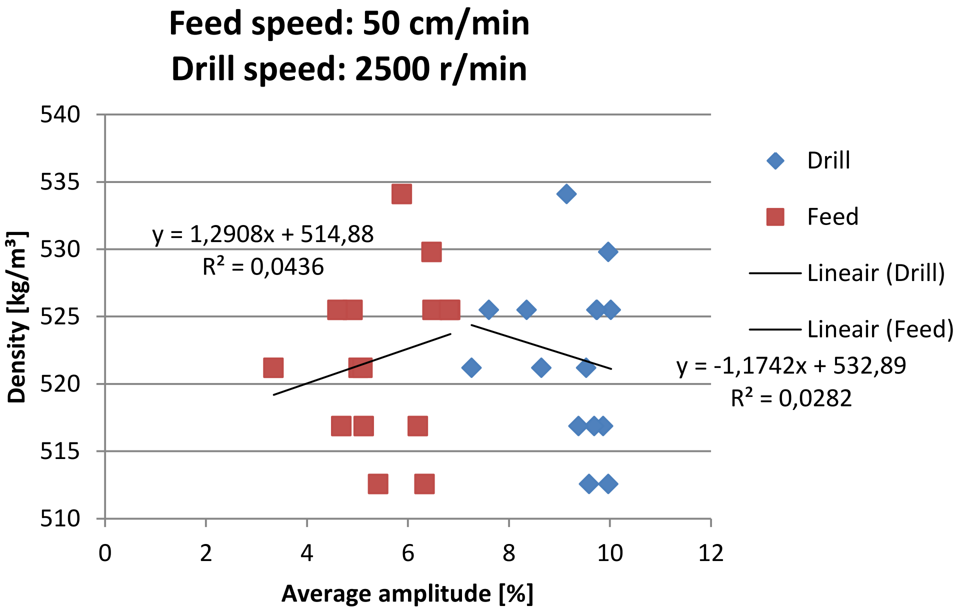


Figure 3-9: Correlation of the energy needed with the measured density

Figure 3-10: Cross section of timber beam with sapwood in the corners

It is thus important to drill perpendicular to the growth rings. An experience user might be able to tell the location of the pith by taking a closer look at the visible grains on the side. A more destructive way is to drill in a random direction and repeat it with an adjust angle until a satisfied pattern is found.

An observation can be made here. Wooden beams of structural size are usually cut close to the pith. As figure 3-10 shows the pith lies around the middle of the height but this isn't always the case. However towards the other side of the beam sapwood is present which means the rings are closer to each other so that there is more latewood and thus a higher density.

3.5.6 FOUR POINT BENDING TEST

A four point bending test was performed, however the required test setup could not be followed. A distance of 6h (≈1150 mm) between the point loads was required but the present settings only allowed for a distance of 900 mm. Therefore also the configuration for measuring the local MOE was adjusted to l_1=750 mm. This alternative setup does not lead to wrong results because the formulas for the MOR and MOE were derived for any distance.

Before destructive testing the expected bending strength in N/mm² was calculated using (Ravenshorst G. , 2015) for softwoods:

$$f_{mod} = -0.0071 * \rho_{12} + 0.00304 * E_{dyn,12} + 4.94 \quad \text{(Eq. 3)}$$

Where ρ_{12} = the density 12% moisture content [kg/m³]
$E_{dyn,12}$ = the dynamic modulus of elasticity at 12% moisture content [N/mm²]

All beams were tested with the same top side as during its period of use. This was important because the knots were often present in the compressive zone. The expected failure mechanisms, as shown in figure 2-14, were clearly visible. Crack initiation was always from the knot on the bottom or on the side close to the bottom. Appendix G.5.3 describes the failure mechanisms.

Table 3-13 shows the results along with the prediction of the bending strength. Member S7 was not used for this test and member L3 did not fail before the limit of the bench was reached.

Member ID	W, ultimate [mm]	F,ultimate [kN]	E_{local} [N/mm²]	E_{global} [N/mm²]	f_m [N/mm²]	f_{mod} [N/mm²]	Error [%]
S1	52,48	23,57	7704	6702	30	30	0,00
S2	80,94	37,07	13536	10207	48	40	-16,67
S3	50,27	34,09	11680	10556	43	43	0,00
S4	48,63	22,61	10960	8857	29	36	24,14
S5	48,93	28,78	11659	9495	37	38	2,70
S6	41,80	27,78	13990	10192	36	41	13,89
S7						37	
S8	70,65	35,50	11464	9721	44	39	-11,36
S9	67,66	30,55	10803	7999	38	34	-10,53
S10	51,17	32,50	11055	9341	40	36	-10,00
L1	47,94	58,24	12159	7897	36	38	5,56
L2		52,99	12508	8847	52	43	-17,31
L3	>41	>52	10511	8718	>38	39	

Table 3-13: Results of the four point bending test

The first thing that stands out is the modulus of rupture. There is no reason to assume the strength of older timber beams is reduced over the years. Calculations from the city archive showed that the beams from Kerkhofstraat were graded as standard building wood which equals C18 and thus a bending strength of 18 N/mm². The actual bending strength turned out the be much higher, as was expected. Figure 3-11 shows the difference with the 18 N/mm². The 90-year old beams from Kerkhoflaan were still very strong, partly due to the minor presence of defects.

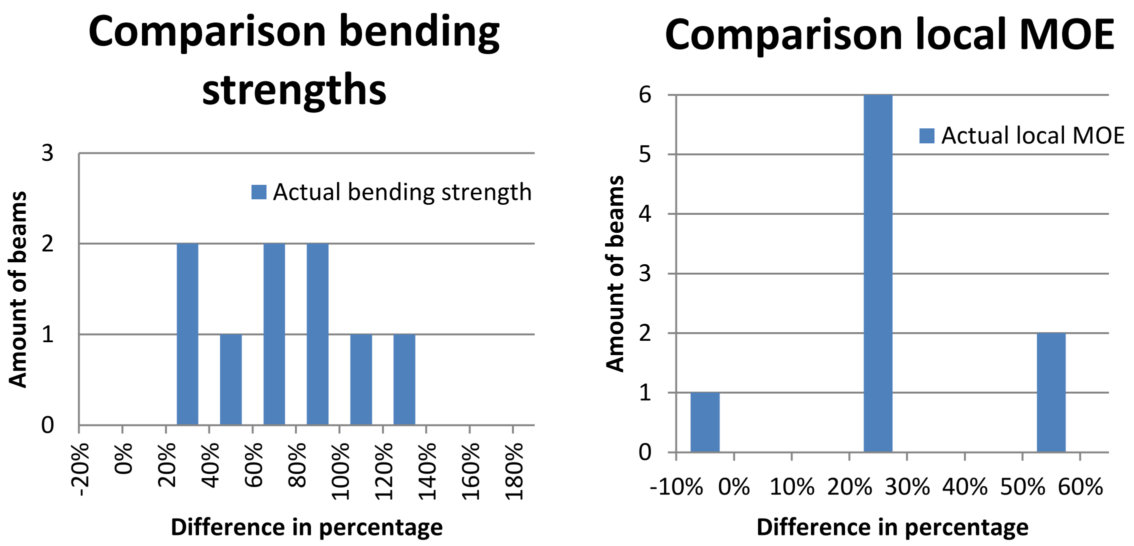


Figure 3-11: Difference between the characteristic value of C18 and the true value in percentage of the S-batch

Secondly, the used prediction formula showed good results. This formula was based on a regression analysis of a dataset of softwood beams. To verify if this formula can be used on older timber beams as well, the test results are plotted in the dataset (see figure 3-12). All points fall within the cloud and are close to the linear regression line. Therefore the prediction formula can be used as an estimator for older structures as well.

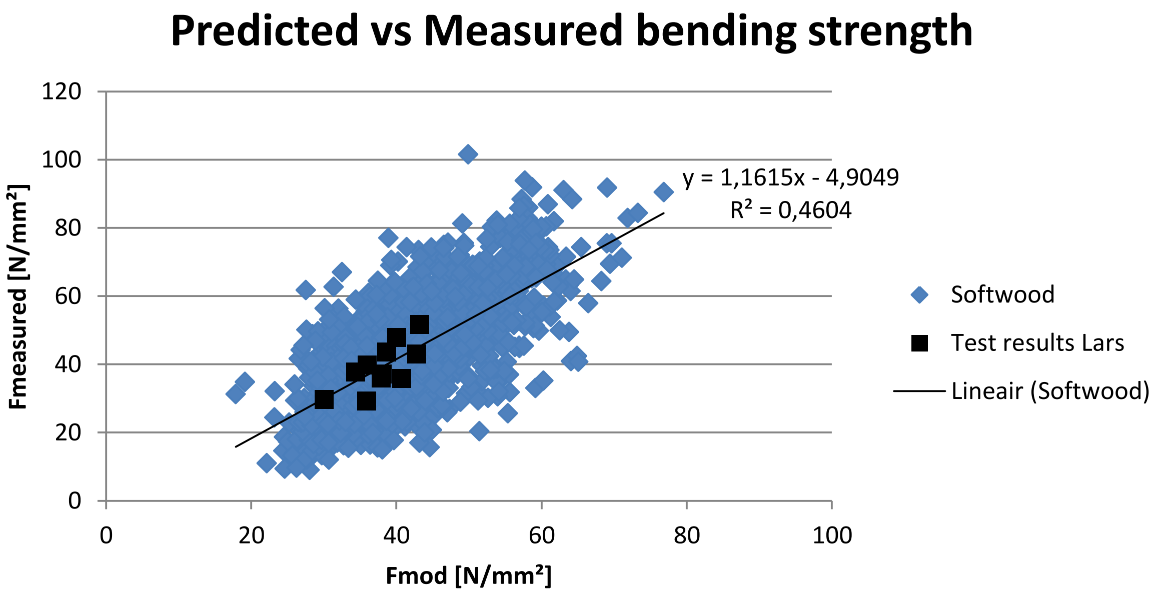


Figure 3-12: True bending strength compared with the predicted bending strength

The modulus of elasticity was measured. For the strength classes, this is determined as the average value. In this case the average MOE_{local} is 11428 N/mm² and 11726 N/mm² for the 30 year and 90 year old beams respectively. Strength class C18 uses an average MOE of 9000 N/mm² while (Govers, 1966) concluded that older softwood beams are around 10000 N/mm². Note that the local modulus of elasticity, which is pure bending, gives higher results than the global modulus of elasticity which includes shear deformation. It was not expected to find a large increase in the elasticity modulus because the characteristic value is based on the average and not the 5% value.

Batch ID	Global MOE [N/mm²]		Local MOE [N/mm²]		Bending strength [N/mm²]		n
	Mean ($\bar{x}$)	SD (s)	Mean ($\bar{x}$)	SD (s)	Mean ($\bar{x}$)	SD (s)	
S	9230	3286	11428	4168	38,67	6,26	9
L	8487	515	11726	1066			3

Table 3-14: Distribution parameters for the MOE and bending strength

At last the relationship between the different material properties is given in appendix G.5.3 by means of scatterplots. A linear regression line is drawn based on the least squares regression. A summary of the regressions is given in table 3-15. Note that the last column contains the recommended values taken from (Ravenshorst G. , 2015) which is based on more test results.

Relationship	Test results		Recommended results	
	Regression line	Coefficient of determination R^2	Regression line	Coefficient of determination R^2
MOE_{dyn}-MOE_{local}	0,96x	0,63	0,95x	0,69
MOE_{dyn}-MOE_{global}	0,75x	0,59	0,81x	0,65
MOE_{global}-MOE_{local}	1,27x	0,52	1,11x	0,78
MOE_{dyn}-MOR	0,004x - 8,37	0,54	0,0036x - 2,96	0,48
Density-MOE_{dyn}	31,84x - 3052,5	0,36		
Density-MOR	0,15x - 30,23	0,27		

Table 3-15: Test results of the scatterplots compared with (Ravenshorst G. , 2015)

3.6 STRATEGIES FOR FUTURE ASSESSMENTS

The NEN 8700 gives several options for gaining strength (chapter 3.1) when the Eurocode calculation with the parameters of a new roof do not fulfill the requirements. Based on options a and e, three strategies can be applied for assessing future timber roofs. Each step requires more work but will (most likely) lead to extra strength.

Strategy 0: Calculate as new structure

With only the available information from the city archive, perform unity checks with the same parameters as a new structure according to the Eurocode. This might already be sufficient.

Strategy 1: Reduce the reference period

According to the NEN8700 the reference period may be reduced and thus lowers the load factors. This is already discussed in chapter 3.1 and is greyed out here because it is not recommended unless the engineer can determine and control the load with high precision.

Strategy 2: Visual grading upgrade

Visual grading needs no expensive tools but requires an experienced timber grader. The strength class can be upgraded because the way of loading is known and thus the stress distribution. For the case of Kerkhofstraat this leads to inhomogeneous beams with strength class of C24 in the middle and C18 in the outer parts.

Strategy 3: Non-destructive testing

Non-destructive testing involves expensive tools (e.g. resistograph and vibration meter). Three approaches can be used which make use of probabilistic models.

a. Update the strength class
   The predicted strengths can be plotted in a database of known test results. Based on the lower 5-percentile value a strength class can be assigned to the beams.

b. Bayesian updating
   The Bayesian approach makes use of the already available information, the so called *a priori* information. This contains information about the original quality (which may be found in the city archive) or the visual strength grade. Non-destructive tests applied on the present beams leads to *a posteriori* information.

c. Classical inference
   This approach assumes no *a prior* information is available and thus only information from the measurements exists. A stochastic variable $X = (x_1, x_2, \ldots, x_n)$ is measured from non-destructive testing. This variable has a unknown deterministic parameter ϴ. The measured X belongs to a distribution of probability $f_x \cap P(X|\Theta,\Omega)$ where the space Ω of all possible outcomes of X is unknown. An estimator for ϴ is searched for using the obtained data and the smallest error.

3.6.1 DISCUSSION STRATEGIES

This chapter makes clear how the safety of the structure changes and discusses the design values that should be used during calculations.

A structure is considered safe when the solicitation (S) has a small chance of being larger than the resistance (R). Therefore $S_d \leq R_d$ which is simplified displayed in figure 3-13.

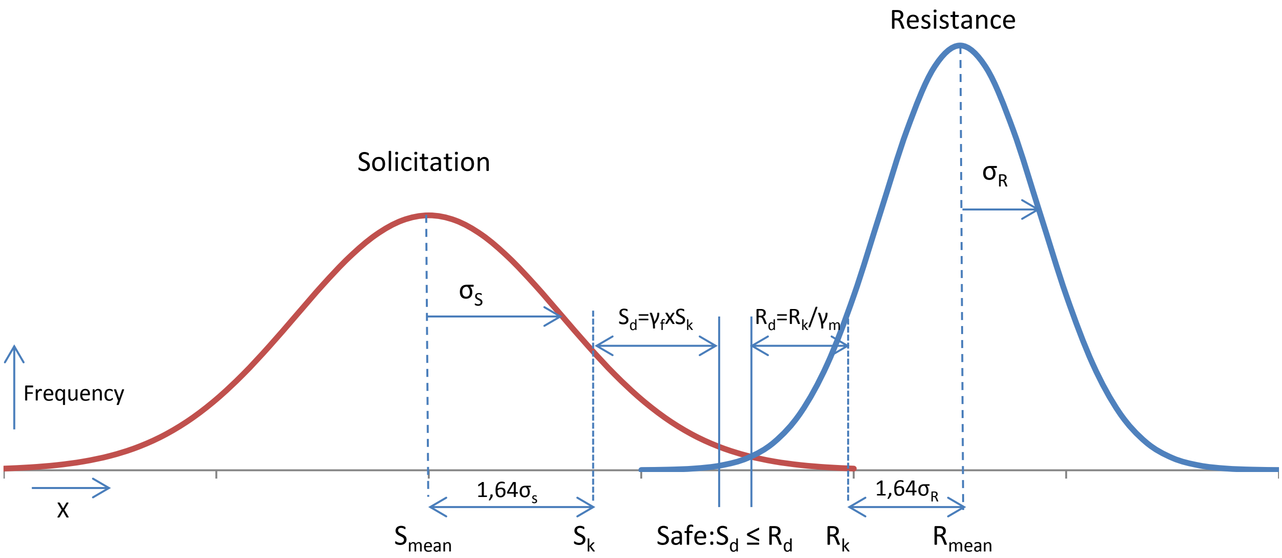


Figure 3-13: Simplified display of the solicitation distribution vs the resistance distribution

Adjusting only the Solicitation side

In strategy 0 and 1 only the load is updated. The extra load will move S_{mean}, and thus the total graph, towards the right. This is no problem as long as $S_d \leq R_d$. Strategy 1 updates the reference period and the associated load factors γ_s. Thus S_{mean} will move towards the right and S_d towards the left. Whether this is allowed was already discussed in chapter 3.1. It basically comes down to how well the accuracy of the loads in the future can be estimated or controlled.

Adjusting the load and resistance side

Strategy 2 and 3 also adjust the resistance side. R_{mean} and S_{mean} will move towards the right. Because the resistance can be estimated with a certain precision, σ_r will also decrease and R_d moves further to the right. This is made visible in figure 3-14.

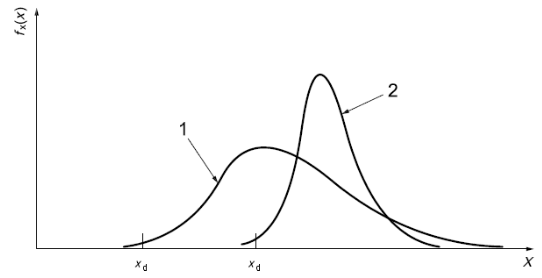
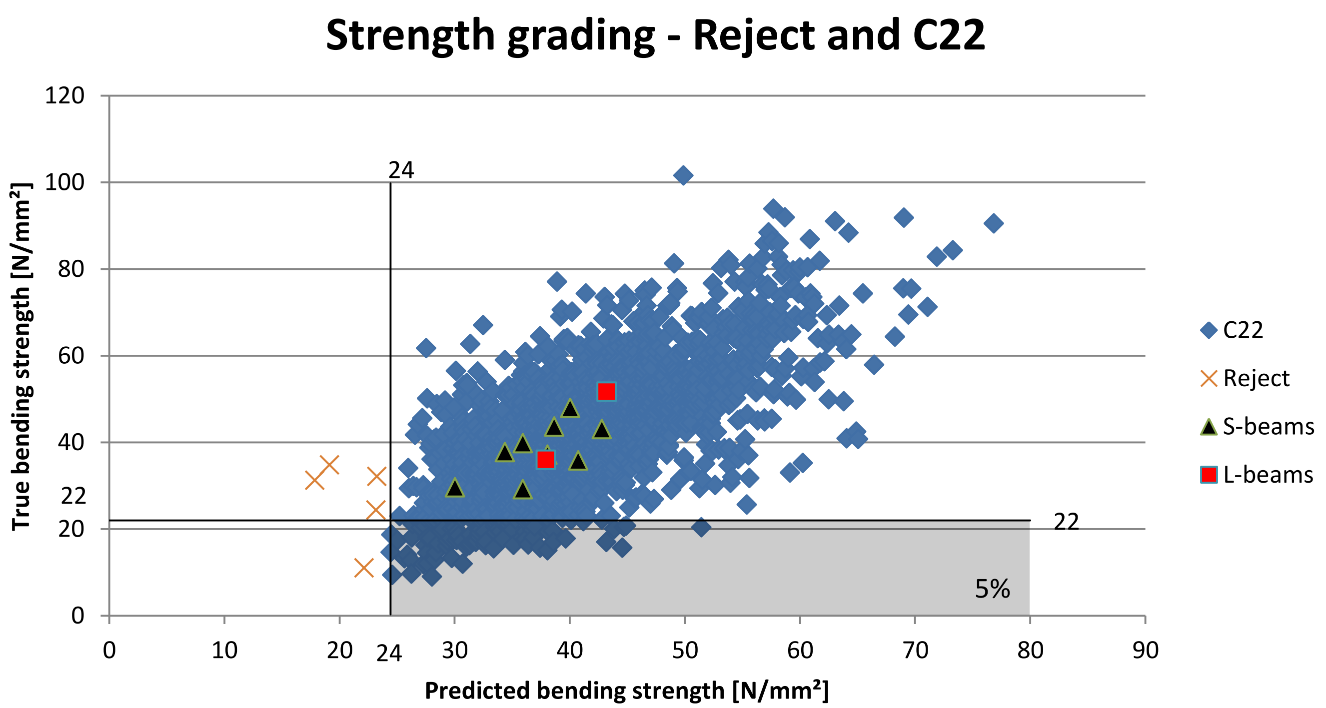


Figure 3-14: (1) the initial and (2) the updated probability density function (NEN-ISO 13822:2010)

A distinction must be made between two options for determining the design value for ultimate limit state checks. Either the test results update the strength class (strategy 3a) or the individual reference properties (strategy 3b/c).

Strategy 3a

The database of softwood test results can be divided in different strength classes. An iterative search leads to an acceptable results. Figure 3-15 shows an example on the tested roof beams. All beams could be graded as C22 but also a combination between C18 and C27 is possible. In this case the latter has the preference because only two beams are not C27. These can be strengthened if necessary or the lower strength class is accepted because the decking in a roof spreads the loading to stiffer parts as explained in chapter 2.5.5.



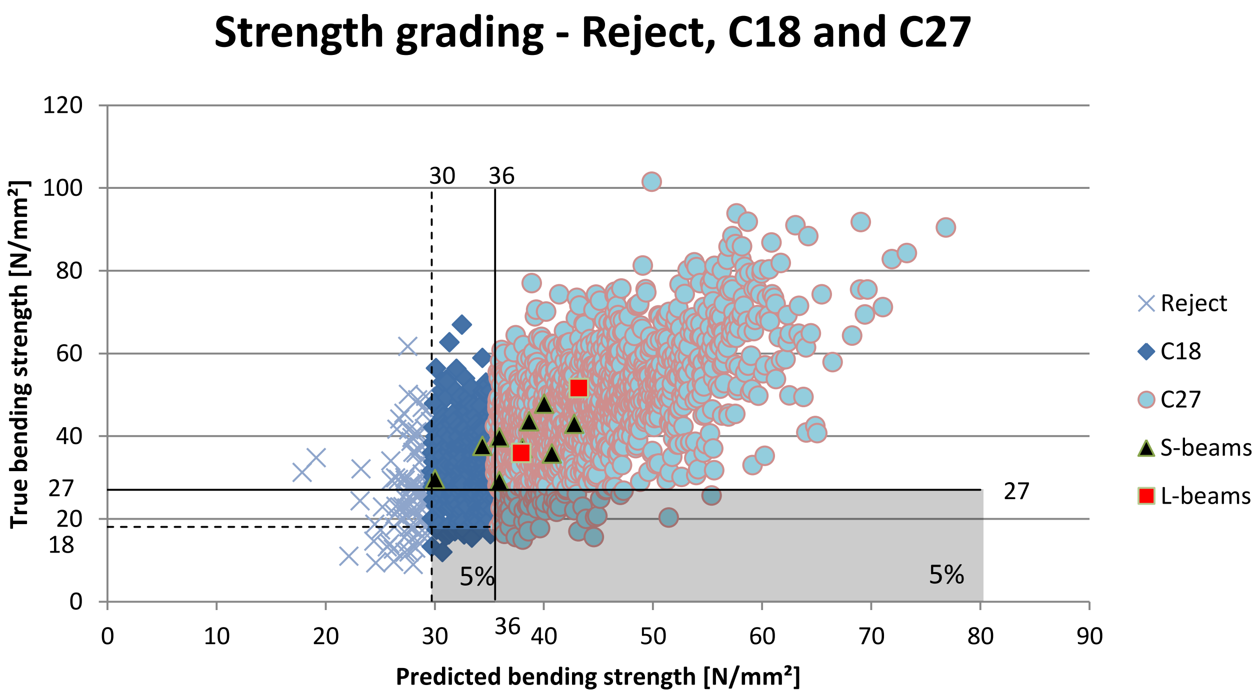


Figure 3-15: Strength grading based on the lower 5% value. The two plots show that different combinations are possible.

<u>Strategy 3b/c</u>

Updating the individual reference properties is reliable when the required property can be predicted with a good accuracy as for instance the modulus of elasticity. The prediction for the bending strength was based on the best fitted linear regression line. This means that the prediction has a certain distribution and variance. Thus in some cases there is an overestimation of the true strength. At the same time, a prediction of one in situ roof beam contains an error because the two used parameters, density and dynamic MOE, also have a certain distribution. Here the best prediction would be the average of different measurements in one beam:

$$\mu_{f_{mod}} = -0.0071 * \mu_{\rho_{12}} + 0.00304 * \mu_{E_{dyn,12}} + 4.94 \qquad \text{(Eq. 4)}$$

Where $\mu_{\rho 12}$ = the mean density at 12% moisture content [kg/m³]
μ_{Edyn12} = the mean dynamic modulus of elasticity at 12% moisture content [N/mm²]

A probabilistic analysis is needed to determine the material factor to take into account the model uncertainties.

When many test results are available than the classical inference has the preference. Here the best estimator is searched for. In this thesis classical inference is used only for the modulus of elasticity. The best estimator will then be the mean of the test results.

One may argue that testing every beam individually is time consuming. Hence another aspect is whether tests on a few beams can tell something about all the beams that were used in that roof or even building block which was built in the same project. It is plausible to assume that the used batch came from the same growth area and may therefore have a smaller variance in the properties between the beams. Here the Bayesian updating has the preference because there is only limited amount of new test results available. According to (JCSS, 2001) the stochastic variable X can be found by:

$$X = m'' + t_{v''} * s'' * (1 + \frac{1}{n''})^{0.5} \qquad \text{(Eq. 5)}$$

Where '' = posterior
m = the mean value

t_v = the central t-distribution
s = the standard deviation
n = the number of observations.

Because the mean value of the modulus of elasticity is searched, the second part of the formula can be neglected. The mean value is then defined as:

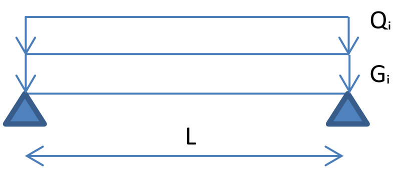
$$m'' = \frac{n' * m' + n * m}{n''} \qquad \textit{(Eq. 6)}$$

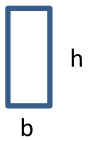
Where ' = a priori
'' = posterior
m = the mean value
n = the number of observations

3.6.2 STRATEGIES APPLIED ON TWO CASES

Appendix G.7 shows a worked out example on the case of Kerkhofstraat and Kerkhoflaan. The input, results and assumptions (marked with *) are shown below.

Constructive scheme:





Kerkofstraat:	Kerkhoflaan:
L = 4100 mm	L = 4600 mm
b = 75 mm	b = 90 mm
h = 195 mm	h = 240 mm
Distance between beams = 600 mm	Distance between beams = 500 mm (*)
G_1 = 0,60 kN/m²	G_1 = 1,40 kN/m² (*)
Strength class: C18	Strength class: C24 (*)

Material: Sawn timber
Consequence class: CC2
Building category: H – Roofs
Climate class: 2
Duration of load class: permanent (permanent load) and short (variable load)
Maximum allowed deflection: L/250

The different loads and combinations are as follow:

A)	Existing permanent load	Varies
B)	Green roof extensive	1 kN/m²
C)	Green roof intensive 1	1 kN/m²
D)	Green roof intensive 2	3,4 kN/m²
E)	Uniform distributed load due to maintenance	1 kN/m²
F)	Green roof use	1,75 kN/m²

LC	A	B	C	D	E	F
1	1,35	1,35				
2	1,35			1,35		
3	1,20	1,20			1,50	
4	1,20		1,20			0,60 x 1,50
5	1,20			1,20		0,80 x 1,50
6	1,00	1,00				
7	1,00			1,00		
8	1,00	1,00			1,00	
9	1,00		1,00			0,60 x 1,50
10	1,00			1,00		0,80 x 1,50

Table 3-16: Load combinations of both cases

Strategy 0 results, Eurocode new:

	Kerkhofstraat		Kerkhoflaan	
ULS check	Applied stress [N/mm²]	Allowed [N/mm²]	Applied stress [N/mm²]	Allowed [N/mm²]
LC1	5,7	8,3	5,0	11,1
LC2	14,3	8,3	9,9	11,1
LC3	9,1	12,5	6,7	16,6
LC4	9,3	12,5	6,8	16,6
LC5	18,3	12,5	12,0	16,6
SLS check	Deflection [mm]	Allowed [mm]	Deflection [mm]	Allowed [mm]
LC8 (w_{inst})	13,8	16,4	8,7	18,4
LC9 (w_{inst})	14,1	16,4	8,8	18,4
LC10 (w_{inst})	28,6	16,4	15,8	18,4
LC8 (w_{fin})	20,5	16,4	13,6	18,4
LC9 (w_{fin})	20,8	16,4	13,7	18,4
LC10 (w_{fin})	45,7	16,4	25,7	18,4

Table 3-17: Results of applying strategy 0

Strategy 1 results, NEN8700 (informative):

	Kerkhofstraat		Kerkhoflaan	
ULS check	Applied stress [N/mm²]	Allowed [N/mm²]	Applied stress [N/mm²]	Allowed [N/mm²]
LC1	5,1	8,3	4,4	11,1
LC2	12,7	8,3	8,8	11,1
LC3	8,3	12,5	6,2	16,6
LC4	8,5	12,5	6,2	16,6
LC5	17,0	12,5	11,2	16,6
SLS check	Deflection [mm]	Allowed [mm]	Deflection [mm]	Allowed [mm]
LC8 (w_{inst})	13,8	16,4	8,7	18,4
LC9 (w_{inst})	14,1	16,4	8,8	18,4
LC10 (w_{inst})	28,6	16,4	15,8	18,4
LC8 (w_{fin})	20,5	16,4	13,6	18,4
LC9 (w_{fin})	20,8	16,4	13,7	18,4
LC10 (w_{fin})	45,7	16,4	25,7	18,4

Table 3-18: Results of applying strategy 1

Strategy 2 results, visual upgrade around the middle:

	Kerkhofstraat (C18->C24)		Kerkhoflaan (C24->C30)	
ULS check	Applied stress [N/mm²]	Allowed [N/mm²]	Applied stress [N/mm²]	Allowed [N/mm²]
LC1	5,7	11,1	5,0	13,8
LC2	14,3	11,1	9,9	13,8
LC3	9,1	16,6	6,7	20,8
LC4	9,3	16,6	6,8	20,8
LC5	18,3	16,6	12,0	20,8
SLS check	Deflection [mm]	Allowed [mm]	Deflection [mm]	Allowed [mm]
LC8 (w_{inst})	11,3	16,4	8,0	18,4
LC9 (w_{inst})	11,5	16,4	8,0	18,4
LC10 (w_{inst})	23,4	16,4	14,6	18,4
LC8 (w_{fin})	16,9	16,4	12,4	18,4
LC9 (w_{fin})	17,1	16,4	12,5	18,4
LC10 (w_{fin})	29,0	16,4	19,0	18,4

Table 3-19: Results of applying strategy 2

Note that the deflection is decreased compared to a homogeneous beam. Calculating the beam as inhomogeneous led to a gain in stiffness of almost 20%. The difference between homogeneous and inhomogeneous in other situations, where inhomogeneous part is L/3, can be defined as:

$$\varepsilon = \frac{5}{384} * \frac{q * l^4}{(EI_1)} - \frac{5}{384} * \frac{q * l^4}{(EI_2)} + \frac{1}{216} * \frac{q * l^4 * (EI_1 - EI_2)}{EI_1 * EI_2} = -\frac{29}{3456} * \frac{q * l^4 * (EI_1 - EI_2)}{EI_1 * EI_2} \qquad \text{(Eq. 7)}$$

Where q = the load [N/mm]
l = total span [mm]
EI_1 = Stiffness properties from 0 to L/3 [N/mm²]
EI_2 = Stiffness properties from L/3 to 2L/3 [N/mm²]

For the derivation see appendix G.6.

Strategy 3a results, updating strength class:

As was shown in figure 3-15, the strength class depends on the division. Below a table is given with different grading groups and the number of beams in that group.

Divisions	Kerkhofstraat	Kerkhoflaan
C22	9	3
Reject/C24	1 / 8	0 / 3
Reject/C16/C24	0 / 1 / 8	0 / 0 / 3
Reject/C18/C27	0 / 2 / 7	0 / 0 / 3
Reject/C18/C30	0 / 6 / 3	0 / 1 / 2
Reject/C22/C30	2 / 5 / 2	0 / 1 / 2

Table 3-20: Number of beams that can be classified in different groups

For Kerkhofstraat the group Reject/C18/C27 is used with the idea that the two C18 beams can be reinforced until the same strength as C27 is reached. Reject/C18/C30 is chosen for Kerkhoflaan. This seems plausible since the average MOE of Kerhoflaan (11428 N/mm²) and Kerkhoflaan (11726 N/mm²) seem to match the average of the strength class C27 (11500 N/mm²) and C30 (12000 N/mm²)

	Kerkhofstraat (C18->C27)		Kerkhoflaan (C24->C30)	
ULS check	Applied stress [N/mm²]	Allowed [N/mm²]	Applied stress [N/mm²]	Allowed [N/mm²]
LC1	5,7	12,5	5,0	13,8
LC2	14,3	12,5	9,9	13,8
LC3	9,1	18,7	6,7	20,8
LC4	9,3	18,7	6,8	20,8
LC5	18,3	18,7	12,0	20,8
SLS check	Deflection [mm]	Allowed [mm]	Deflection [mm]	Allowed [mm]
LC8 (w_{inst})	10,8	16,4	8,0	18,4
LC9 (w_{inst})	11,0	16,4	8,0	18,4
LC10 (w_{inst})	22,4	16,4	14,6	18,4
LC8 (w_{fin})	16,2	16,4	12,4	18,4
LC9 (w_{fin})	16,4	16,4	12,5	18,4
LC10 (w_{fin})	35,6	16,4	19,0	18,4

Table 3-21: Results of applying strategy 3a

Strategy 3b/c results, Classical and Bayesian inference:

In figure 3-16 a comparison is made between the different methods. The error with the true value is given as a normal distribution. A priori information is provided by the initial strength class whereas the coefficient of variation given in (JCSS, 2006) is used. In the Bayesian inference it was considered that the test values weigh 3 times more than the priori information. Figure 3-16 shows that both methods reduce the error. In case of the S-batch the classic inference is better because more test results are available. The Bayesian inference showed a smaller error in the L-batch.

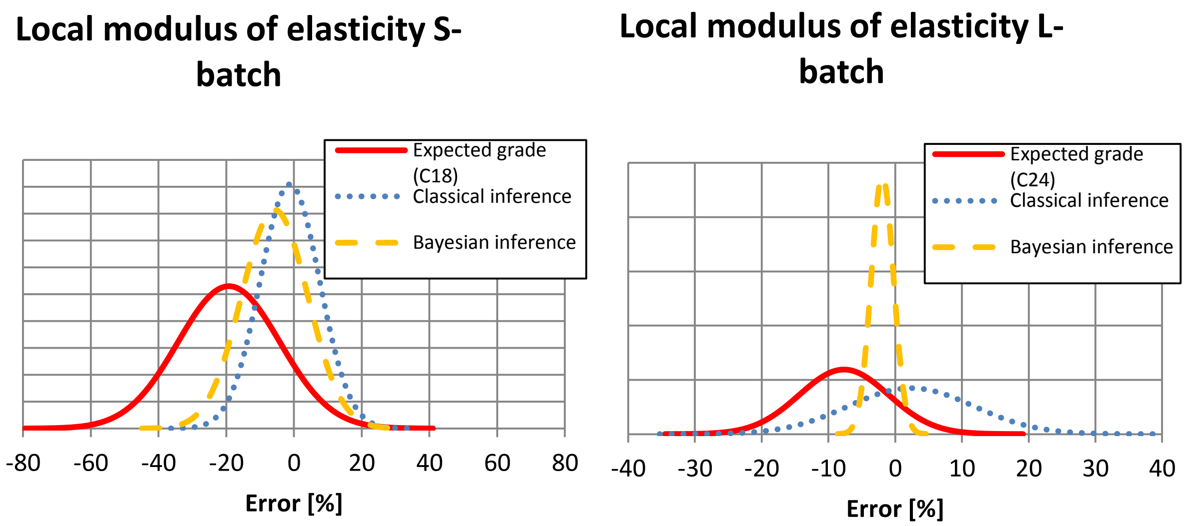


Figure 3-16: Error between the applied strategy and the true value of the local modulus of elasticity

Conclusion

The application of the different strategies allow for higher loads with every step. As expected, strategy 3 is the most beneficial because information about the current strength is attained. Comparing this strategy with strategy 0,the bending strength was increased with a factor 1,5 in case of Kerkhofstraat.

In this case study it becomes clear that low weight green roofs (1 kN/m²) can be applied while a heavy green roof (3,4 kN/m²) needs more attention. A solution between these two extremes is also possible. Time dependent factors seem to be the main problem in all strategies. The duration of load may cause excessive deflections or even creep rupture. Limits to the deflections are not legally established and can be concealed with a lowered ceiling.

In strategy 3a it was seen that the strength class has a higher MOE than was measured. Therefore it is better to use the MOE from strategy 3b/c. The Bayesian inference becomes more reliable when only a limited amount of beams are tested.

The case of Kerkhoflaan showed that all strategies lead to satisfied result. Reasons might be that either the initial assumptions were wrong or older beams are overdimensioned because they are based on experience. The latter conclusion can only be made when more older roofs are tested.

3.7 VISUAL ASSESSMENT OF INSPECTED BEAMS

During the research three groups of roof beams were inspected: the in-situ roof beams (paragraph 2.3.1), the obtained beams (paragraph 3.5.3) and the in-situ garage beams (appendix G.5.2). In table 3-22 the visual assessment of these beams is given. Attention is particular paid to the degradation mechanisms as described in paragraph 2.8.

Mechanism	In-situ roof beams	Obtained beams	Garage beams 1	Garage beams 2
Mechanical	NP	See appendix G.5.1	NP	NP
Physical	A fire occurred / drying cracks		NP	NP
Chemical	NP		NP	NP
Insects	NP		NP	NP
Rot/disfiguring	NP		NP	NP
Discoloring	Black due to fire / other parts are gray		Some gray	NP
Water damage	NP		Visible near supports	NP
Mycelium's	NP		NP	NP

Table 3-22: Aspects of visual assessment, NP = not present

The fire decreased the in-situ roof beams cross section. Apparently the remaining cross section can still withstand the loads. Another observed aspect is discoloring. Some parts of the beams become grayer due to ageing. This is not destructive. At last the garage beams showed some water damage, if the moisture content is too high than there is a chance of biological attacks.

4. STRENGTHENING OF EXISTING ROOF STRUCTURES

In chapter 2 and 3 it was concluded that there is often extra strength available. When an intensive green roof is preferred and thus higher loads need to be taken into account, some roofs might need strengthening depending on the roof function. A distinction must be made between reinforcing the roof for extra capacity and reinforcing for restoring capacity (repair). The latter is needed when degradation processes have taken place.

The consequences of a green roof and the failure mechanisms were discussed in chapter 2.5. A timber beam in bending will always fail at the brittle tension side due to ductile behavior of the compression zone. Design methods in the Eurocode are based on elastic theory and doesn't take the extra plastic resistance into account. Furthermore the tension zone is often weakened by the presence of knots.

4.1 OPTIONS

Over the years different methods have proven to work for reinforcing a timber beam. These methods can be divided into four groups:

Replacing structure (parts)

Additional structure (parts)

- Adding beams
- Adding supports
- Change support
- Increase cross section
- Transverse brace
- Tie rods

Composite systems

- Timber-timber
- Timber-steel
- Timber-concrete
- Timber-plastic

Inserting reinforcing elements

- Bars/plates
- Self-tapping screws
- FRP

The different methods are described in appendix H. Some of these options have a high impact on the existing structure.

4.2 CONSTRAINTS

The choice for a method of reinforcing an existing structure depends on the constraints. An engineer and user should discuss the possibilities that satisfies both. Different aspects should be considered:

- The economic aspects like the costs of extra material and man-hours. The chosen method should be easy and cheap.
- The geometry and boundary materials which determines the structural behavior. Other parts like the decking and bearing wall must be suitable with the intervention.
- The extra capacity needed, reinforcement is only needed until a certain stress level can be resisted. Also the method must be able the increase the resisting bending moment in the middle.
- The demands of the user. In some cases the members have aesthetical value and the intervention option must then be carefully chosen to respect the users wishes. For instance the user may object to a lowered ceiling or an additional structure.
- The durability of a reinforcing method. This concerns not only the lifetime and environmental factors but also the maintenance needed after intervention has taken place. Associated with this is the accessibility of the structure.
- The preference aspects. A solution that ensures water and root resistance over the full lifetime is preferred so that expensive monitoring equipment is redundant. Also a green roof increases the lifetime of the roof covering. It is preferred to leave the decking and covering untouched. At last is not preferred to conceal the structure from the bottom, this makes visual monitoring harder.
- The time that is needed to realize the intervention.

Cultural heritages sometimes require the reinforcement method to be reversible. A roof of a house is not expected to become a monument and it is considered that a green roof will be present over the remaining lifetime of a building which makes this criterion unnecessary. Furthermore there are some local constraints that must be considered. For instance a skylight or chimney might be present.

The last column in appendix H shows if the reinforcing method is suitable with a green roof considering the constraints.

4.3 SOLUTIONS

The most optimal solution will depend on the existing timber structure because every situation is unique. A distinction can be made between individual or overall reinforcing. The former is used when only a few beams do not meet the requirement. Overall reinforcing can best be applied when the total roof structure is incapable of transferring the load. Strengthening on the bottom side of a beam is not preferred because large deflections can occur and a lowered ceiling is necessary. This already reduces the available height.

Individual reinforcing

The most favorable solutions for reinforcing individual beams is increasing the cross section, bonding FRP to the tension zone or applying a steel strip on the bottom. Increasing the cross section with new timber elements has the preference in the context of sustainability.

Overall reinforcing

When no detailed assessment is desired or when test results do not lead to sufficient strength, the total roof structure can be reinforced. The most favorable solution is a composite system where timber and concrete or timber and timber work together. Here the new material is placed on top of the existing roof covering and horizontal shear forces need to be transferred through shear connecters. Due to the existing covering a gap is created between the beam and the new material. This leads to a moment in the connecter. To prevent high shear forces the connecters can be placed under an angle so that also a normal force occurs. In reality the existing decking will also contribute to the load transfer.

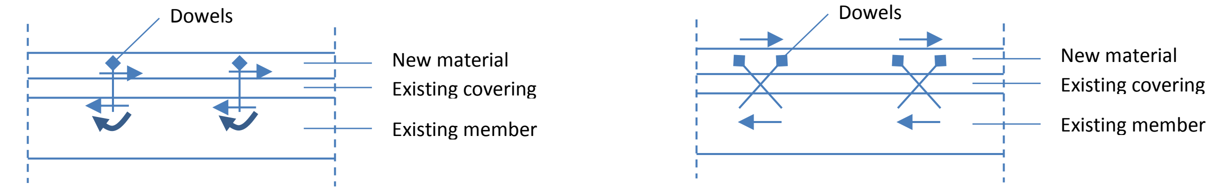


Figure 4-1: Longitudinal cross section of composite system

Both methods work optimal when the existing beam is unloaded before attaching the new parts. For individual reinforcing this can be achieved by placing jacks on the floor below. When overall reinforcing is chosen it should be determined if the floor below is capable to resists the forces from the jacks.

4.3.1 STRENGTHENING OPTIONS APPLIED ON TWO CASES

Strategy 3a allows for some beams in one roof to have a lower strength class than the adjacent beams. In the case study of Kerkhofstraat two beams were C18. These can be strengthened until the maximum load for C27 can be resisted. In this case two timber strips of 35 mm x 80 mm of C24 can be glued or bolted to the side in the tension zone, see figure 4-2. Appendix H.1 shows that the new beam can resist the same bending moment as C27. Another option is triplex plates on each side connected with glue or bolts. For Kerkhoflaan the same procedure can be followed.

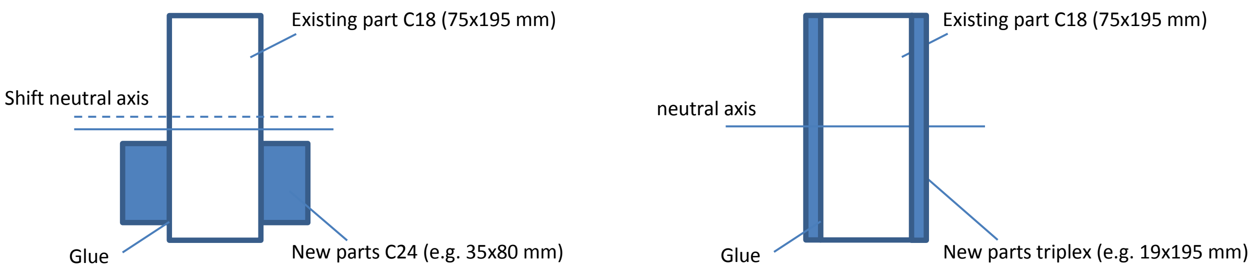


Figure 4-2: Cross section of strengthened beam with strips (left) or plates (right)

Full cooperation is achieved when the glue can resist the shear forces.

5. CONCLUSION AND RECOMMENDATIONS

Based on the information provided in this work, **the sub-questions** can now be answered:

1. How many different kind of timber roof structures were constructed in Rotterdam?

From a construction point of view , there is almost in all cases a beam simply supported by two masonry walls. A more interesting perspective is the timber itself. In the past, timber beams in constructions were chosen on experience and throughout the years more knowledge led to more economic solutions. In the introduction it was noted that the municipality of Rotterdam made potential maps for houses. This is their starting point for a decision tool. The criterion “year of construction” was based on experience with houses and not on timber. The new grouping of houses takes into account the design norms and is therefore more suitable for the decision tool.

2. What were the design procedures in the past since the norms changed through the years, starting from the first norm?

A timeline is visible in figures 2-20 and 2-21. The TGB-norms stayed more or less the same over the years and thus older roofs are not overdesigned. Visual grading norms became less strict over the years.

3. What happened to the strength of the timber over the years?

Wood is an organic material which is sensitive to time dependent processes that reduce the strength. Age is not necessarily a strength-reducing factor but is associated with strength-reducing processes. Four degradation mechanisms can be distinguished for timber: mechanical, physical, chemical and biological. The latter is the largest problem for roofs because insulation or treating the wood was not always done (correctly). It is expected that roof structures still have their initial strength.

4. What kind of (non-destructive) grading methods can be used to determine the strength?

A list with different grading methods is provided. Different authors reported that the three reference properties can best be determined by the resistograph (for bending strength and density) and stress waves (for modulus of elasticity). Furthermore, promising results were showed when different methods were combined.

5. What is the current strength of the existing timber beams?

A method was presented that can determine a new strength class that corresponds better to the actual used beam. For this, a strength prediction model was used that required the density and dynamic modulus of elasticity. The density can be measured with aid of a Resistograph. Next a vibration meter can be used to measure the wave speed which can be combined with the density to gain the dynamic MOE. A case study showed that the predicted bending strength is 1,5x higher than the initial strength.

6. What combination factors can be used for the new load occurring together with the current loads?

In this thesis the saturated weight of a green roof was considered as a permanent load because the purpose is to buffer and slow down the water drainage. Besides, the load has a maximum value (extra water is discharged by the emergency overflow) and thus it makes sense to use a smaller partial factor because the uncertainty of exceeding the maximum value is small. Even though the load can be predicted and controlled with good accuracy it is recommended to still use a load factor due to possible gardening in the future.

7. Do the timber beams comply with the current demands of the Eurocode standards?

The answer to this question depends on the actual situation. For the two discussed case studies it was shown that using non-destructive tests will lead to more strength. At first the extra load did not meet the requirements of the Eurocode but more load was allowed with test results.

8. How can the strength of the beams be increased by means of a reinforcing method?

Several options are presented. The most optimal solution will depend on the existing timber structure because every situation is unique. A distinction can be made between individual or overall reinforcing. The former is used when only a few beams do not meet the requirement. Overall reinforcing can best be applied when the total roof structure is incapable of transferring the load. An easy and cheap method is to increase to cross section with new timber elements.

9. What steps should be followed for future assessments?

A protocol is presented in the recommendations.

The **main research question** can now be answered:

How much water can be buffered on the existing timber roof structures, and how can this be increased when there is more knowledge about the uncertainties of the structure?

A single answer to this question is impossible to give because every structure and timber element is unique. Therefore a solution was searched that determines the strength of timber beams in existing structures. Another motivation for this solution is the lack of information in the city archive. Non-destructive tests are thus often inevitable. The municipality wants to buffer a minimum of 25 L/m². Based on the results, there is no reason to doubt this possibility once certain criteria are met. When more water needs to be stored the extra weight can lead to problems. This thesis provides several ways to increase the bearing capacity by reducing the uncertainty about the real strength and upgrading the strength class. A strengthening method can be applied when the upgrade is still insufficient. The new approach was applied on two case studies which showed that a heavy green roof (3,4 kN/m²) might be realized in Kerkhoflaan. ZinCo Benelux B.V. indicates that this roof type has a buffering capacity of 110 l/m². In conclusion, this research shows that strengthening of roof beams might not be needed for green roofs.

At last the expectations about **the limitations** can be given:

It is clear that the proposed methods for gaining extra strength in existing structures can also be used for other purposes than a green roof. The method is valid for any case where timber beams are in an existing structure and the strength needs to be determined. Furthermore this thesis was limited to the city of Rotterdam but can be applied on every house with a flat timber roof structure in the Netherlands because the building styles are more or less the same. The method can also be applied on sloped roofs but more research to the construction and the influence of the surroundings is needed.

5.1 FUTURE RESEARCH

This thesis is only the first step towards a Rotterdam with green roofs. From an engineering point of view it is interesting to research the following topics into more depth:

- What are the similar strength parameters in each housing group as defined in chapter 2.6? When non-destructive tests are performed on houses of each group, is there one strength class associated with a certain typology?

- How can the in-situ measurements for the dynamic MOE be improved and what coefficient should be used on the measured frequency in-situ? A simulation of the in-situ situation can be reconstructed and tested to determine the influence of the surroundings. This can also be determined for roofs with a slope where tiles are present.

- The NEN8700 gives more options for coping with extra load: determine load values on the actual use and not on values from the Eurocode, control the load by taking measures or adjust the safety margin by a precise probabilistic analysis. Are these methods suitable and how much strength can be gained?

- What solution is the most economic beneficial? A detailed assessment requires extra man-hours and expensive equipment. It might be financially better to directly strengthen the timber with cheap methods.

- The used prediction model for the strength was based on the density and dynamic modulus of elasticity. It was found that the variance is large in the estimation of the strength. A better prediction model or better measurement options might improve the estimation. An important aspect is that the existing beams are already graded once in the past and the origin of the roof beams might be the same. Further research can be done how this information improves the prediction accuracy.

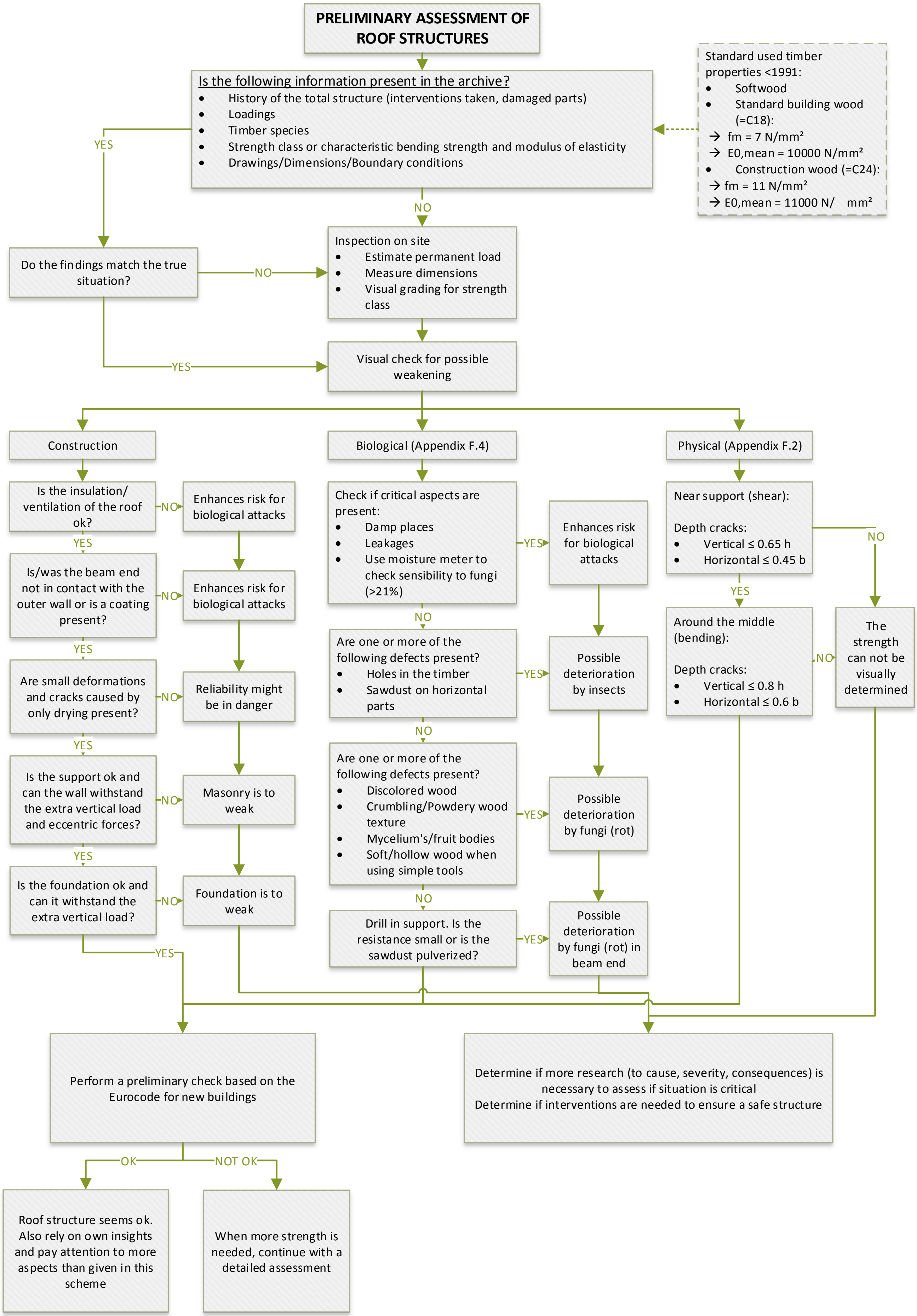
- In the case study it was shown that the beams can resist the extra load of a heavy green roof but can eventually lead to creep rupture. If this problem can be tackled than all requirements are fulfilled. A possible solution might be to perform a probabilistic analysis on the amount of water expected to be present. In this thesis it was considered as a permanent load which indicates a lower modification factor on the strength. When the duration of load is more clear this factor might become higher and thus more favorable.

- All tests were conducted with after the beams were placed inside a climate room. This resulted in a moisture content of 12%. The true moisture content is expected to be higher for in-situ situations. More research can therefore be done to the influence of moisture on the non-destructive test results.

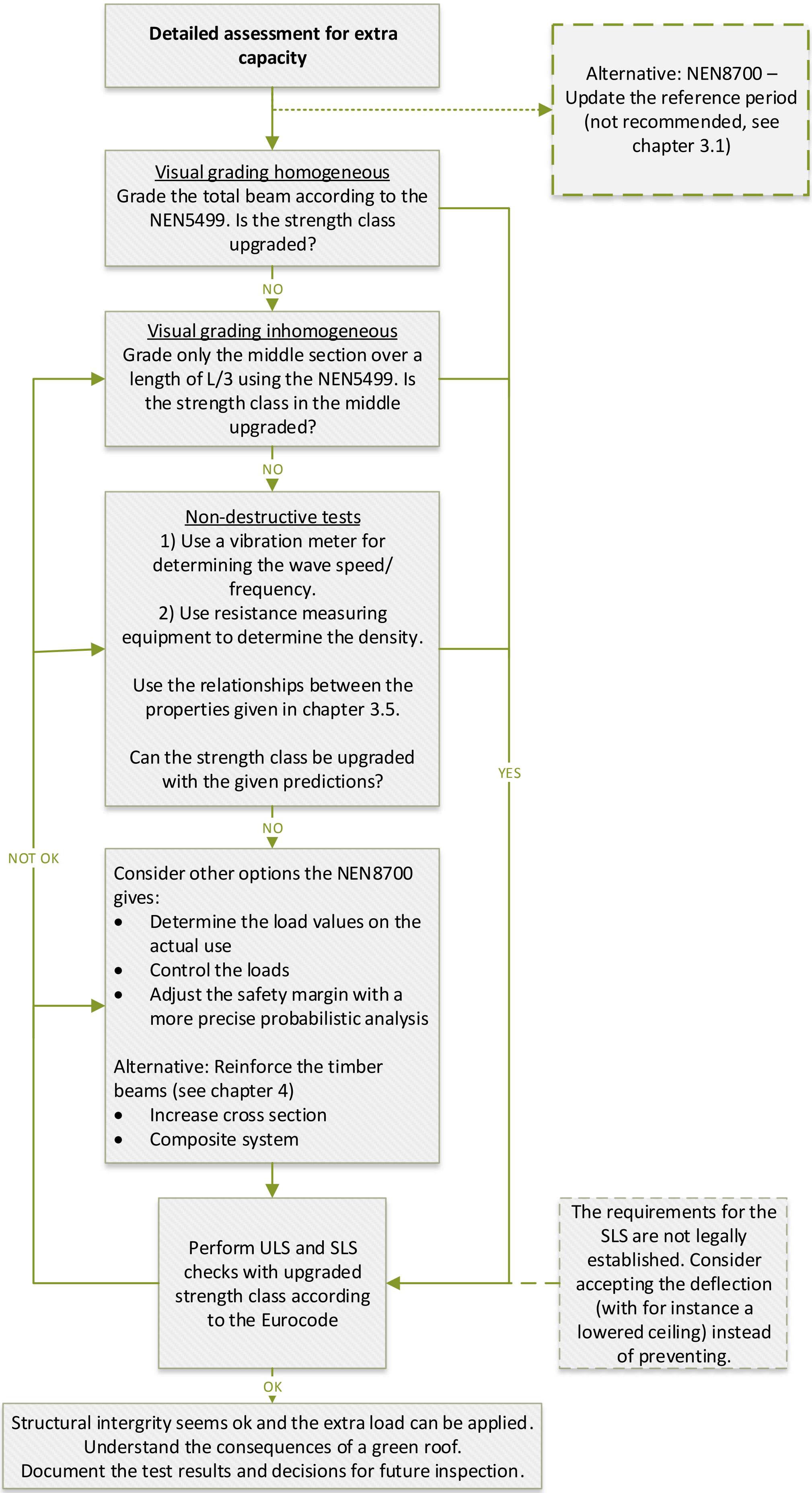
5.2 RECOMMENDATIONS

All of the research can be summarized in a protocol for future assessments. In 2003 van Reenen created an action plan for coping with older (oak) beams (van Reenen, 2003). This plan is adjusted and expanded to fit the purpose of houses and green roofs.

The assessment starts with a prior evaluation that checks if the existing structure is suitable and reliable. The first step is to collect all available data of the existing structure. This information is commonly found in the city archive and needs to be verified with the real situation. When no information can be found, visual grading during inspection can be performed for determining the strength class. Inspection of the existing structure is inevitable because different degradation mechanisms might have occurred. At last the structure with the extra load can be checked with the Eurocode for new buildings. Also keep in mind favorable aspects given in table 2-5. When more strength is required than the engineer can continue with the detailed assessment. The steps here are based on the different strategies. Starting with visual grading of the whole beam or parts of the beam according to the NEN 5499 for softwoods. Subsequently non-destructive tests can be performed. After each step the engineer must decide if enough strength is gained or must continue with the next step.



PRELIMINARY ASSESSMENT OF ROOF STRUCTURES
Is the following information present in the archive?
History of the total structure (interventions taken, damaged parts)
Loadings
Timber species
Strength class or characteristic bending strength and modulus of elasticity
Drawings/Dimensions/Boundary conditions
Standard used timber properties <1991:
Softwood
Standard building wood (=C18):
→ fm = 7 N/mm²
→ E0,mean = 10000 N/mm²
Construction wood (=C24):
→ fm = 11 N/mm²
→ E0,mean = 11000 N/ mm²
YES
NO
Do the findings match the true situation?
NO
Inspection on site
Estimate permanent load
Measure dimensions
Visual grading for strength class
YES
Visual check for possible weakening
Construction
Biological (Appendix F.4)
Physical (Appendix F.2)
Is the insulation/ ventilation of the roof ok?
NO
Enhances risk for biological attacks
YES
Is/was the beam end not in contact with the outer wall or is a coating present?
NO
Enhances risk for biological attacks
YES
Are small deformations and cracks caused by only drying present?
NO
Reliability might be in danger
YES
Is the support ok and can the wall withstand the extra vertical load and eccentric forces?
NO
Masonry is to weak
YES
Is the foundation ok and can it withstand the extra vertical load?
NO
Foundation is to weak
YES
Check if critical aspects are present:
Damp places
Leakages
Use moisture meter to check sensibility to fungi (>21%)
YES
Enhances risk for biological attacks
NO
Are one or more of the following defects present?
Holes in the timber
Sawdust on horizontal parts
YES
Possible deterioration by insects
NO
Are one or more of the following defects present?
Discolored wood
Crumbling/Powdery wood texture
Mycelium's/fruit bodies
Soft/hollow wood when using simple tools
YES
Possible deterioration by fungi (rot)
NO
Drill in support. Is the resistance small or is the sawdust pulverized?
YES
Possible deterioration by fungi (rot) in beam end
Near support (shear):
Depth cracks:
Vertical ≤ 0.65 h
Horizontal ≤ 0.45 b
NO
YES
Around the middle (bending):
Depth cracks:
Vertical ≤ 0.8 h
Horizontal ≤ 0.6 b
NO
The strength can not be visually determined
Perform a preliminary check based on the Eurocode for new buildings
Determine if more research (to cause, severity, consequences) is necessary to assess if situation is critical
Determine if interventions are needed to ensure a safe structure
OK
NOT OK
Roof structure seems ok. Also rely on own insights and pay attention to more aspects than given in this scheme
When more strength is needed, continue with a detailed assessment



Detailed assessment for extra capacity
Alternative: NEN8700 – Update the reference period (not recommended, see chapter 3.1)
Visual grading homogeneous
Grade the total beam according to the NEN5499. Is the strength class upgraded?
NO
Visual grading inhomogeneous
Grade only the middle section over a length of L/3 using the NEN5499. Is the strength class in the middle upgraded?
NO
Non-destructive tests
1) Use a vibration meter for determining the wave speed/ frequency.
2) Use resistance measuring equipment to determine the density.
Use the relationships between the properties given in chapter 3.5.
Can the strength class be upgraded with the given predictions?
YES
NO
NOT OK
Consider other options the NEN8700 gives:
• Determine the load values on the actual use
• Control the loads
• Adjust the safety margin with a more precise probabilistic analysis
Alternative: Reinforce the timber beams (see chapter 4)
• Increase cross section
• Composite system
Perform ULS and SLS checks with upgraded strength class according to the Eurocode
The requirements for the SLS are not legally established. Consider accepting the deflection (with for instance a lowered ceiling) instead of preventing.
OK
Structural intergrity seems ok and the extra load can be applied.
Understand the consequences of a green roof.
Document the test results and decisions for future inspection.

5.3 ALTERNATIVE SOLUTION FOR BUFFERING WATER ON ROOFS

In the past research was also done to steel and concrete roofs. Most of the roof structures in Rotterdam are made from timber materials which has proven to be suitable for extra loads. All materials have their disadvantages but in every situation the residual capacity is the largest uncertainty. One of the driving forces behind green solutions is C.M. Ravesloot[3]. During his latest research he designed an additional structure that does not make use of the existing roof structure. The so called Facility Roof Rack (FRoRa, see figure 5-1) is a lightweight truss that spans over the existing structure from wall to wall. Solar panels or solar heathers can be applied on the new system and orientated for optimal performance so that sustainable energy is generated. Barrels can be used to buffer the rain water. This water can then be discharged with some delay or be used for personal use. Green roofs can now be lighter since only the dry condition is present.

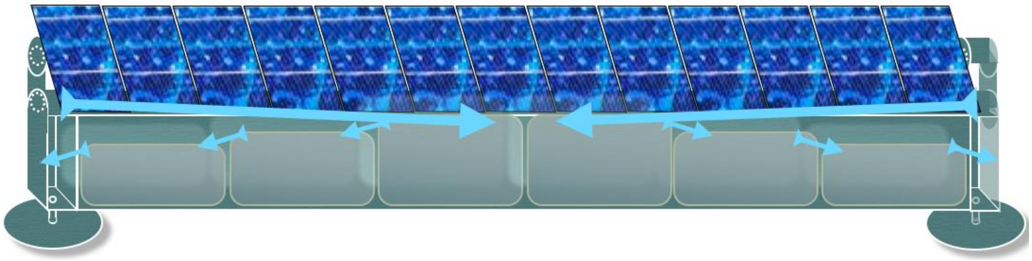


Figure 5-1: Facility Roof Rack designed by Ravesloot

[3] Personal and written contact on 22-06-15. Dr.drs.ir. Christoph Maria Ravesloot is a lector of Inholland University and Rotterdam University. His goal is to accelerate the introduction of sustainability.

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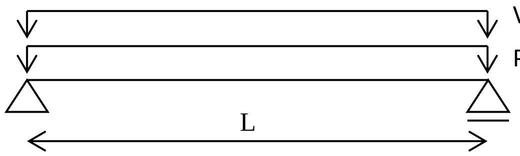
Appendices

TABLE OF CONTENTS

A. A SIMPLIFIED CALCULATION

The following strength calculation gives an indication of how great the problem is. Some assumptions[4,5], based on a flat roof, are needed for the uncertainties. The level of uncertainty and reason is presented in the brackets behind each variable.

Constructive scheme:



Boundary conditions = Simply supported (Low: most roof structures are simply supported, however some rotation might, unintentionally, be restrained. There might also be an extra support in the middle)

Strength = 7 N/mm² (High: current standard flat roof varies between C16 – C30. This value is used in the two cases[2] for the strength check of the old structure and to design the new structure)

Dimensions:

L = 4500 mm (Medium: depends on the size of the building. The beam lengths of the two cases varied between 4000 mm and 5000 mm)

BxH = 75x200 mm (Medium: depends mostly on the load which depends on the used norm.)

Distance between beams = 610 mm (Medium: this is a standard value for roofs, this might deviate from previous standard values)

Loads:

Permanent load:
- Self weight = 450 kg/m³ (High: depends on strength class)
- Roof structure = 0,60 kN/m² (High: depends on structure, some might have gravel or an air conditioner on top)
- Vegetation = 100 kg/m² (Low: own choice based on average value for extensive green roof)

Variable load:
- Water = 0,25 kN/m² (Low: own choice based on buffered water)
- Maintenance = 1,0 kN/m² over 10 m² (High: it is not yet certain how buffered water in combination with other loads occurs)

The importance of finding out how the variable loads should be combined is high. The maintenance load of 1,0 kN/m² is probably never present when the water load is the extreme value (the soil is fully saturated), here a factor of 0,2 is used. When the 1,0 kN/m² is seen as the extreme value, the water load gets a factor of 0,5 since they probably won't be working during a heavy rainfall. The importance of these different perspectives comes clear in the following unity checks where both situations are considered.

Case 1: Extreme value: maintenance; Combination factor water load: 0,5

4 Based on www.houtinfo.nl
5 Based on 2 houses (Heemraadstraat 21-35 and Herlaerstraat 5-13) that were built around 1900, however the roof structure is renewed around 1982.

Crosssection check for bending

$$> \mathrm{b} := 0.075 : \mathrm{h} := 0.200 : \mathrm{fm} := 7\cdot 10^{3} : \mathrm{L} := 4.5 : \mathrm{qp} := 450\cdot 10^{-2}\cdot \mathrm{b}\cdot \mathrm{h} + 0.610\cdot \left(0.6 + 100\cdot 10^{-2}\right); qv := 0.610\cdot (0.5\cdot 0.25 + 1);$$

$$qp := 1.043500000$$
$$qv := 0.686250 \quad (1)$$

$$> My := \frac{1}{8}\cdot (qp + qv)\cdot L^{2}; \mathrm{Wy} := \left(\frac{1}{6}\right)\cdot b\cdot h^{2}; sigmy := \frac{My}{Wy};$$

$$My := 4.378429688$$
$$Wy := 0.0005000000000$$
$$sigmy := 8756.859376 \quad (2)$$

$$> evalf\left(\frac{sigmy}{fm}, 3\right) \leq 1;$$

$$1.25 \leq 1 \quad (3)$$

Case 2: Extreme value: water; Combination factor maintenance load: 0,2

Crosssection check for bending

$$> \mathrm{b} := 0.075 : \mathrm{h} := 0.200 : \mathrm{fm} := 7\cdot 10^{3} : \mathrm{L} := 4.5 : \mathrm{qp} := 450\cdot 10^{-2}\cdot \mathrm{b}\cdot \mathrm{h} + 0.610\cdot \left(0.6 + 100\cdot 10^{-2}\right); qv := 0.610\cdot (0.25 + 0.2\cdot 1);$$

$$qp := 1.043500000$$
$$qv := 0.27450 \quad (4)$$

$$> My := \frac{1}{8}\cdot (qp + qv)\cdot L^{2}; \mathrm{Wy} := \left(\frac{1}{6}\right)\cdot b\cdot h^{2}; sigmy := \frac{My}{Wy};$$

$$My := 3.336187500$$
$$Wy := 0.0005000000000$$
$$sigmy := 6672.375000 \quad (5)$$

$$> evalf\left(\frac{sigmy}{fm}, 3\right) \leq 1;$$

$$0.953 \leq 1 \quad (6)$$

B. AN OVERVIEW OF ROTTERDAM

This appendix shows aspects of houses in Rotterdam. Figures and numbers give an overview of the distribution of built houses. The data used is based on a document of municipality Rotterdam (Arcadis, 2008) *and the following websites:*

- *www.rotterdamincijfers.nl*
- *www.mappinghistory.nl*
- *bagviewer.geodan.nl*

B.1 ROTTERDAM IN NUMBERS

In 2014 the number of citizens are 618.109 which are accommodated in 299.773 houses. These were built in different time period as can be seen in the next chart:

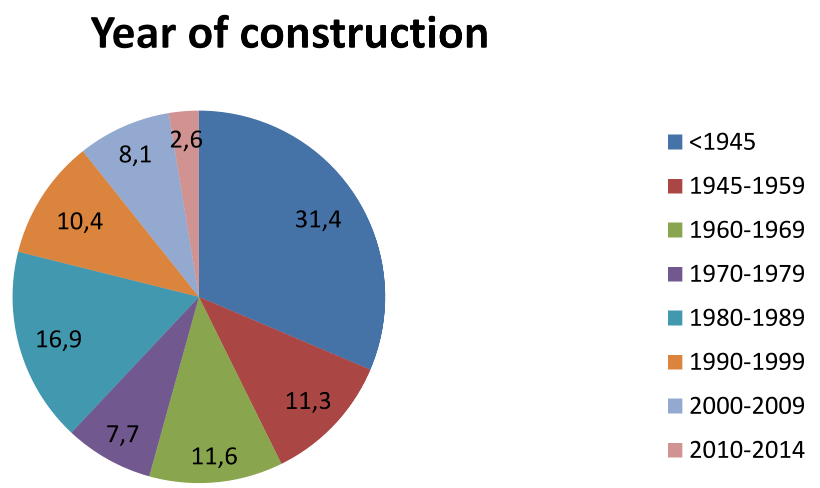


Figure B-1: Distribution of percentage houses in construction years

The roof surface is divided as followed:

Type eigenaar	Dakoppervlak (m^2)	%
Gemeente	669.933	2,9
Grote woningbezitters	9.521.287	40,6
Kleine woningbezitters	9.551.319	40,7
Overheid	282.284	1,2
Onbekend	3.444.736	14,7
Totaal	23.469.559	100

Bouwjaar	Dakoppervlak deelgemeenten (%)
< 1946	25
1946-1955	4
1956-1967	17
1968-1979	9
1980-1994	18
> 1995	8
Onbekend	19
Totaal	100

Figure B-2: Total roof surface of Rotterdam to owner and year of construction (exclusive business area) in 2007

B.2 ROTTERDAM IN FIGURES

Rotterdam around 1850:

Figure B-3: Rotterdam's buildings around 1850

Rotterdam around 1940, note the destroyed city center:

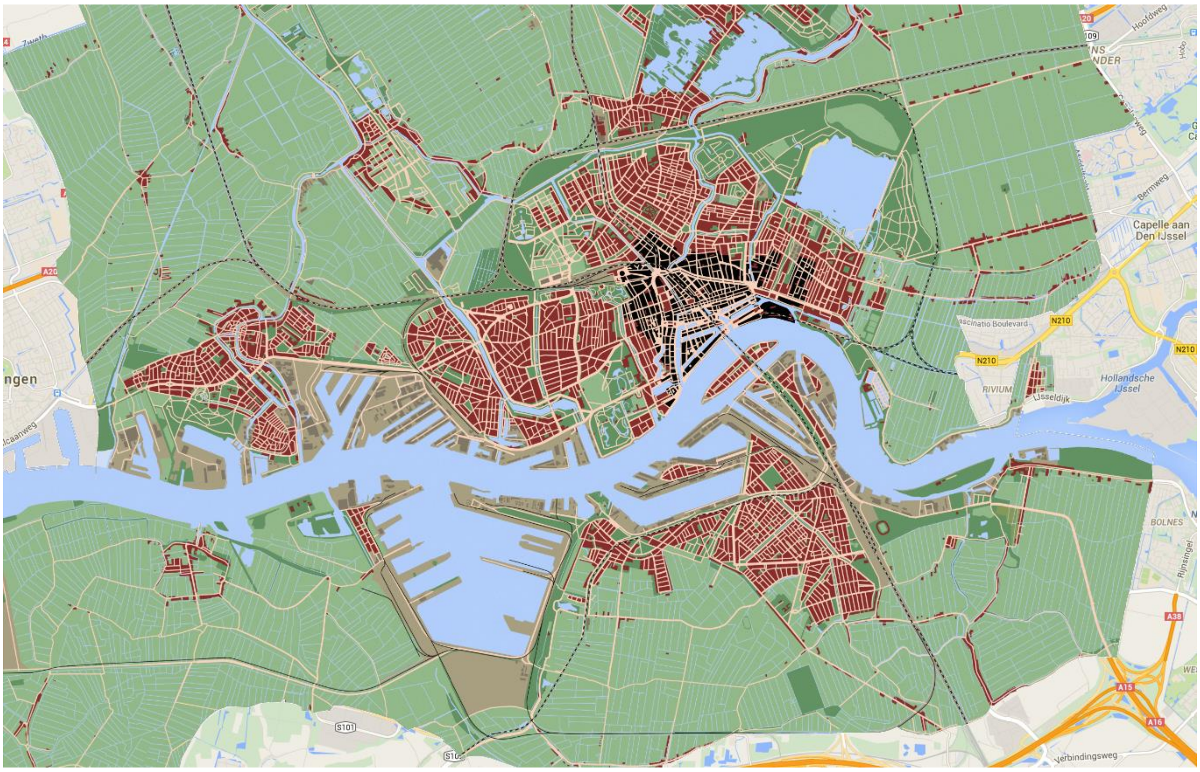


Figure B-4: Rotterdam's buildings around 1940

Rotterdam around 1960:

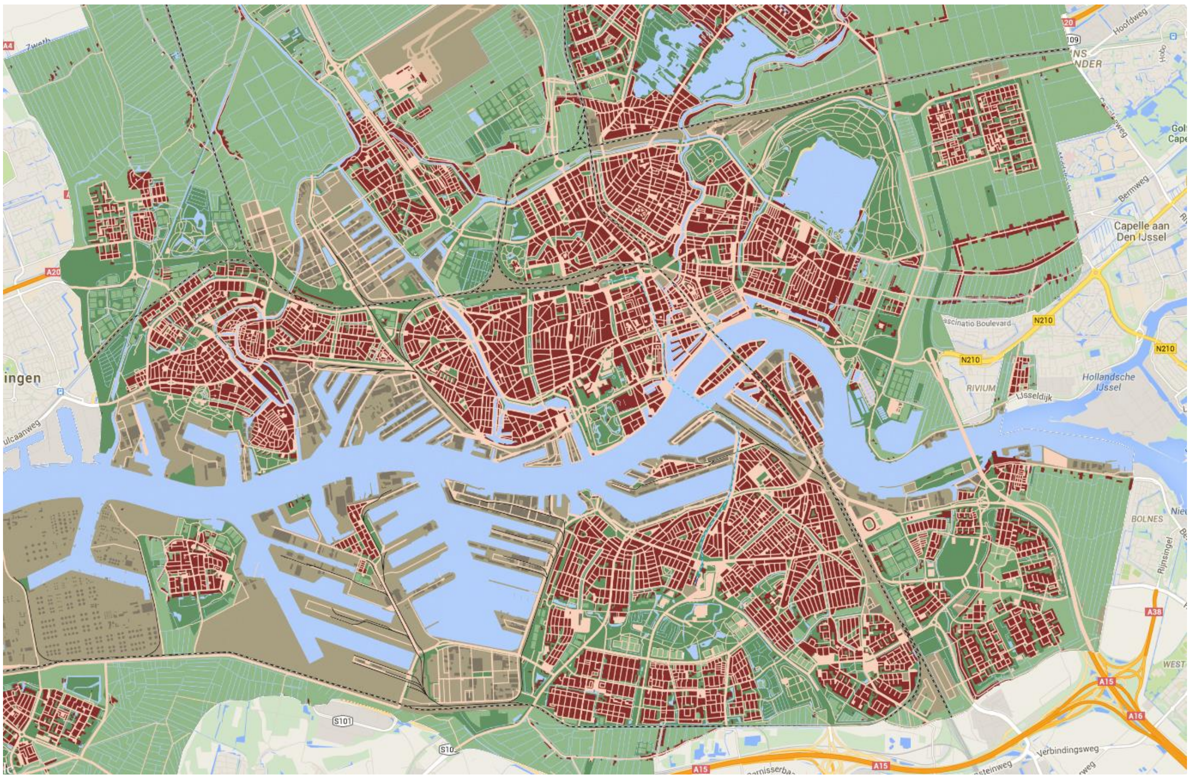


Figure B-5: Rotterdam's buildings around 1960

Rotterdam around 1975:

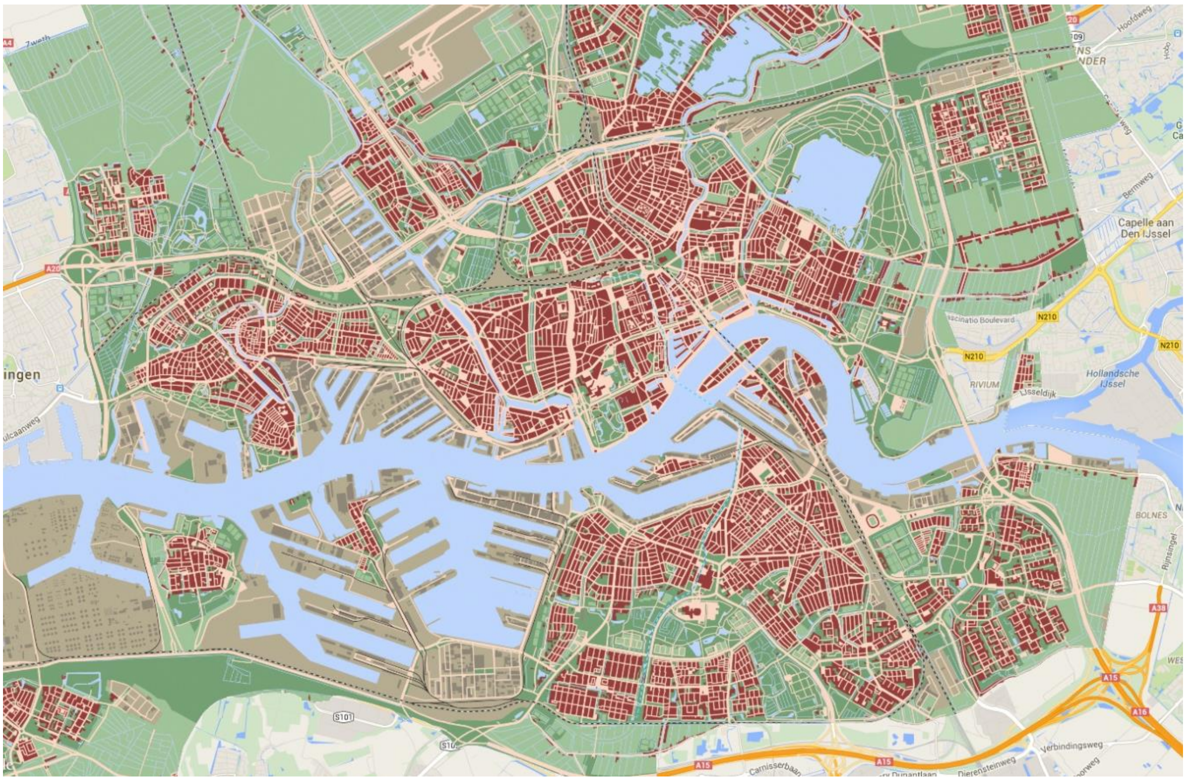


Figure B-6: Rotterdam's buildings around 1975

Rotterdam around 2009:

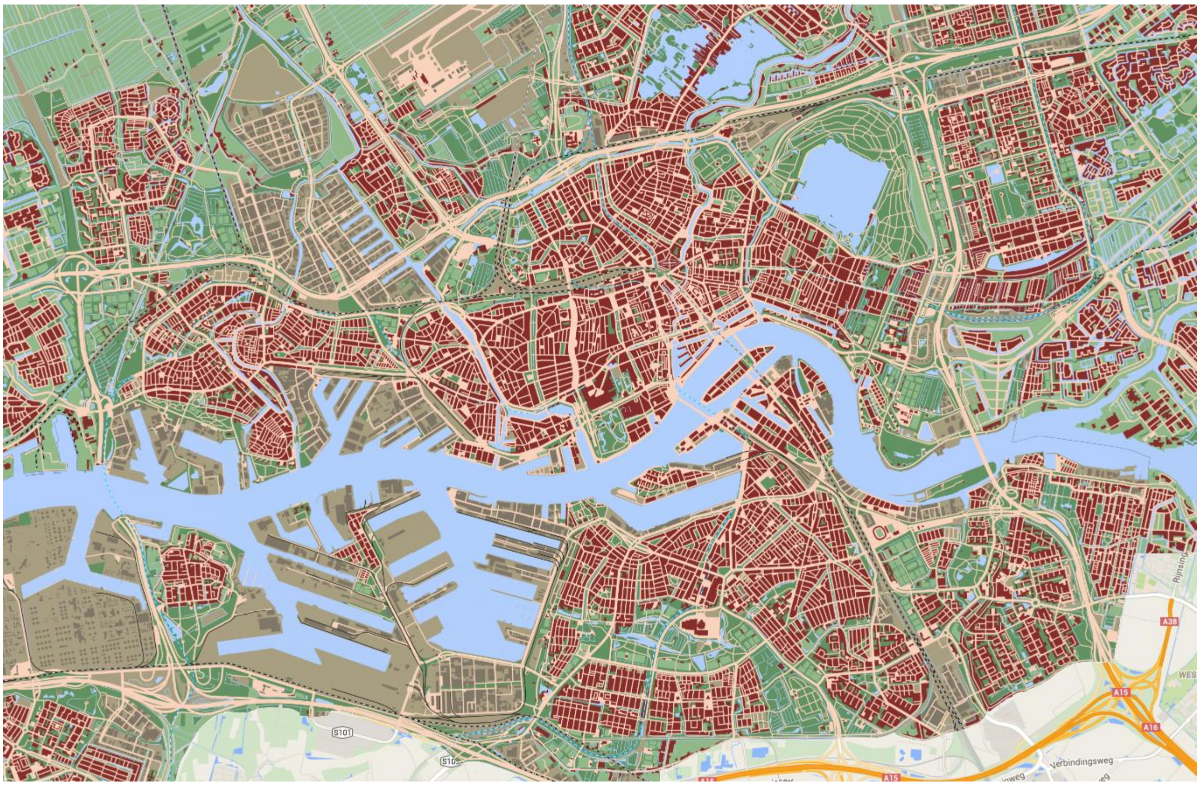


Figure B-7: Rotterdam's buildings around 2009

B.3 SUBMUNICIPALITIES IN NUMBERS:

Rotterdam is divided into 13 submunicipalities. The following figures show where there is potential for green roofs.

	Feijenoord	IJsselmonde	Pernis	Prins Alexander	Charlois	Hoogvliet	Hoek van Holland	Bedrijven terreinen
Voor 1906	3.416	138	299	83	417	54	140	
1906-30	9.620	554	370	323	3.878	26	217	
1931-44	2.991	1.563	239	315	7.615	149	87	
1945-59	831	756	246	137	12.973	1.651	790	
1960-69	92	12.745	700	9.273	2.690	4.732	622	
1970-79	572	3.557	47	8.567	747	2.997	802	
1980-89	4.462	5.737	18	12.889	1.487	3.031	736	
Na 1990	7.523	3.159	228	9.996	4.297	3.709	787	
Totaal	**29.507**	**28.209**	**2.147**	**41.583**	**34.104**	**16.349**	**4.181**	**35**

	Totaal	Stads-centrum	Delfshaven	Overschie	Noord	Hillegersberg-Schiebroek	Kralingen-Crooswijk
Voor 1906	16.248	1.954	2.646	341	4.434	177	2.149
1906-30	48.039	680	13.454	794	8.728	4.009	5.386
1931-44	32.521	719	3.005	1.125	10.110	2.625	1.978
1945-59	36.120	2.997	2.836	3.867	1.082	4.938	3.016
1960-69	35.546	103	25	341	177	3.551	495
1970-79	21.148	580	114	100	699	737	1.629
1980-89	50.035	5.963	2.917	244	3.661	779	8.111
Na 1990	48.399	3.407	4.098	841	3.921	2.845	3.588
Totaal	**288.056**	**16.403**	**29.095**	**7.653**	**32.812**	**19.661**	**26.352**

Figure B-8: Roof surface Rotterdam per submunicipality to year of construction (from 2007)

Type dak	Totaal (m^2)	Stads-centrum	Delfshaven	Overschie	Noord	Hillegersberg-Schiebroek	Kralingen-Crooswijk
Plat	10.451.333 65%	1.055.536 87%	877.353 70%	421.747 66%	817.534 74%	739.202 55%	995.038 76%
Niet-plat	5.698.543 35%	154.626 13%	369.571 30%	219.167 34%	285.724 26%	603.764 45%	317.953 24%
Totaal	**16.149.876**	**1.210.163**	**1.246.925**	**640.914**	**1.103.258**	**1.342.966**	**1.312.991**

Type dak	Feijenoord	IJsselmonde	Pernis	Prins Alexander	Charlois	Hoogvliet	Hoek van Holland	Bedrijven terreinen
Plat	908.604 63%	1.181.343 76%	64.548 46%	1.493.409 68%	1.012.357 67%	588.517 59%	296.145 21%	877.758 99%
Niet-plat	541.545 37%	367.603 24%	75.296 54%	719.485 32%	491.703 33%	406.343 41%	1.145.762 79%	9.709 1%
Totaal	**1.450.149**	**1.548.946**	**139.844**	**2.212.894**	**1.504.060**	**994.860**	**1.441.907**	**887.466**

Figure B-9: Roof surface Rotterdam per sub municipality to roof type (from 2007):

Delfshaven				
District	Building periods of larger housing blocks (Source: BAG)	Year of construction (Source: rotterdamincijfers.nl)	Total number of houses: 33.857 (Source: rotterdamincijfers.nl)	Note
Bospolder	1910-1930 (mixed roofs) 1950-1955 (mostly flat roofs) 1990-2000 (flat roofs)	<1945: 55,5% 1945-1970: 7,9% 1970-2000: 22,6% >2000: 14%	Number of houses: 3167	Many old houses are demolished and replaced by new buildings in the 90's.
Middelland	< 1900 (sloped roofs) 1900-1930 (mixed roofs) 1985-2000 (flat roofs)	<1945: 71,7% 1945-1970: 3,1% 1970-2000: 21,5% >2000: 3,7%	Number of houses: 5328	
Nieuwe Westen	< 1900 (flat roofs) 1900-1930 (mixed roofs) 1990-2014 (flat roofs)	<1945: 80% 1945-1970: 1,7% 1970-2000: 9,4% >2000: 8,9%	Number of houses: 8219	
Oud Mathenesse	1930-1940 (mixed roofs) 1950-1953 (flat roofs) 1990-1993 (flat roofs)	<1945: 50,7% 1945-1970: 38,6% 1970-2000: 9% >2000: 1,7%	Number of houses: 4035	
Delfshaven	1900-1920 (mixed roofs) 1920-1940 (flat roofs)	<1945: 63% 1945-1970: 7,1% 1970-2000: 25,2% >2000: 4,7%	Number of houses: 2873	
Schiemond	1980-1990 (flat roofs)	<1945: 6,6% 1970-2000: 53,7% >2000: 39,8%	Number of houses: 2588	Before 1980 it belonged to the harbor.
Spangen	1918-1940 (mixed roofs) 1990-2014 (flat roofs)	<1945: 74,1% 1945-1970: 3,2% 1970-2000: 11,8% >2000: 10,8%	Number of houses: 4240	Since 1990 many houses were demolished, renovated or rebuilt.
Tussendijk	1920-1930 (mostly flat roofs) 1950-1960 (flat roofs) 1900-2014 (flat roofs)	<1945: 52,4% 1945-1970: 27,2% 1970-2000: 12,1% >2000: 8,3%	Number of houses: 3335	Bombed in 1943.

Table B-1: Houses in districts of Delfshaven

B.4 OVERVIEW OF REQUESTED DRAWINGS

Cases	Number	Timber roof	Permit	Built	Renovated	Executor	Streetname	Submunicipality	Ditstrict	Digital
<1900	1	x	B2 246 74	1867	1974	A. van der lek architect	Schiedamsesingel 187	Centrum	Cool	C+D+P
	2		P8 26 46	1872	1946/later		Eendrachtsweg 67	Centrum	Cool	P
	3		B2 294 86	1890	1986	Woningstichting "onze woning"	Korenaarstraat 61/63	Delfshaven	Nieuwe westen	P
	4					Architektenbureau Post en Eekelen	Korenaardwarstraat 6-14			
	5	x	B2 1058 82	1889-1898	1982	Architektenbureau H. v. Straalen b.v.	Herlaerstraat 5-13	Noord	Agniesbuurt	C+D+P
1901-1920	6	x	B2 168 84	1902-1903	1984	Architektenbureau H. v. Straalen b.v.	Heemraadstraat 29-33	Delfshaven	Nieuwe westen	P
	7		B2 1097 87	1904-1913	1987		Lambertusstraat 57-71	Kralingen-Crooswijk	Kralingen west	P
	8						Lusthofstraat 39-45			No
	9		B2 840 87	1909	1987	Woningstichting "onze woning" / H. v. Straalen b.v.	Davidstraat 32-64	Delfshaven	Nieuwe westen	No
	10						Messcherstraat 7-29			No
1920-1940	11	x	B2 1394 81	1923	1981	Gemeentelijk Woningbedrijf Rotterdam	Kerhoflaan 74-92	Kralingen-Crooswijk	Crooswijk	C+D+P
	12	x					Kerkhofstraat 1-21			
	13	x					Rusthofstraat 3-15			
	14	x	B2 757 83	1923	1983	Gemeentelijk Woningbedrijf Rotterdam	Rusthofstraat 71-117	Kralingen-Crooswijk	Crooswijk	C+D+P
	15	x				Architectenbureau Wout Putter	Kerkhofstraat 4,8-16,18-50			
	16	x	B2 844 85	1925-1926	1985	Goverts hoogstad architekt	Taanderstraat 108-120	Delfshaven	Tussendijken	D
	17	x				Constructeur van Hattem bv	Rosener manzstraat 91-93			D
	18	x				Woningbouwvereniging "de combinatie"	Haringpakkersstraat 21-37			D
	19			1935	2009	Cardo Architecten	Balkenstraat 20	Delfshaven	Spangen	No
1940-1970	20	x	P25 35 42	1942			Schieweg 88-120	Noord	Bergpol	P
	21	x	P11 32 48	1950			Suiestraat, etc.	Delfshaven	Oud-Mathenesse	C+P
	22	x	P21 56 50	1952			Schiedamseweg 252-270	Delfshaven	Bospolder	C
	23		B2 520 53	1953-1955			Fransbekkerstraat 100	Charlois	Oud-Charlois	No
	24	x	B3 18 55	1956			Van Drimmelenstraat 12-41	Pernis		C+D
	25		B3 52 57	1958-1959			Posweg 92-258	Hoogvliet	Hoogvliet	No
1970-2000	26		B2 1259 78	1978-1981			Rembrandtstraat 109-188	Noord	Oude-Noorden	No
	27		B2 643 86	1988		Woningstichting "onze woning" / H. v. Straalen b.v.	Van Heusdestraat 80-88	Delfshaven	Nieuwe westen	No
	28		B2 488 89	1989		Woningbouwvereniging Vreewijk/Lombardijen	Olmendaal 33-51	Feijenoord	Vreewijk	No
	29		B2 464 89	1992		Gemeentelijk Woningbedrijf Rotterdam	"Witte dorp"	Delfshaven	Oud-Mathenesse	No
>2000	30		T1999/592	2001		Maaswerken architecten	Gerrit jan mulderstraat 100-114	Delfshaven	Nieuwe westen	P
	31		T2005/2008	2008		Jorissen simonetti architecten	Omloopdijk	Ijsselmonde	Groot-Ijselmonde	No
	32		T2008/3198	2011		4D architecten	Molgerdijk	Ijsselmonde	Groot-Ijselmonde	No
	C = Calculation		D = Drawing		P = Photo					

	Old					New					
Number	Length [mm]	BxH [mm]	Distance [mm]	σ [N/mm²]	E [N/mm²]	Length [mm]	BxH [mm]	Distance [mm]	σ [N/mm²]	E [N/mm²]	Notes
1						2800	50x150	580			New roof storey / Check new+old structure / Calculation shows roof covering
2											Recover war damage + new garage / garage beams 3".4" and DIN10
3											First a slope now flat with steel profiled roofplates
4											
5				7	10000	4000-5000	75x200	610	7	10000	First a slope now flat
6				7	10000	4000-5000	75x225	600	7	10000	First a slope now flat
7											First a slope now flat with concrete slab (SIPOREX) or steel plate (SAB)
8											Roofs were demolished so not in calculation
9											First a slope now flat with SAB plates
10											
11						3000			10	10000	140 houses merged to 89 / roof structure untouched
12											No roof drawings/calculations
13											Photo roof covering
14						4020+2370	70x275	500	7	10000	85 houses merged to 52 / make flat roof / Rusthofstraat roof might not be renovated / springy roof
15		90x200 (floor)				4130-4460	75x200(71x196)	605	7	10000	Note of a rot beam / nr 35 is same as 39 existing structure is checked
						Till 5000	75x225(71x221)				
						Till 6100	100x250(96x246)				
16		70x175				4400	63x200	600	7	10000	First a slope now flat / used 'liplassen' / Not in project 70x224 distance 630 / warm roof structure
17						2x4050	75x200	600			Beam with possible intermediate support
18		70x144				4200-4500	63x200	600			First a slope now flat / used 'liplassen'
19											No roof details found but strengthclass K17 is mentioned
20			500?								No measurements known, scale 1:100
21						2300-3520	90x165	700	7		Photo of isolation (1948) looks like cold roof / calculation shows roof covering
22						4600	65x180				40 houses; coupling and stormanchor / Calculation shows roof covering
23											Concrete roof
24						4260	80x200	650-680	7		72+61 houses. Flat roof with 3 cm gravel on top
25											Concrete roof
26											Concrete roof
27											Steel plate
28											Drawings do not match the plan. Uncertain what is done here. European spruce class C is mentioned
29											Concrete roof
30											Concrete "breedplaatvloer" / Photo roof covering
31											Concrete "kanaalplaat"
32											Sloped prefab concrete roof

Municipality Delft

Number	Timber roof		Built	Renovated	Executor	Streetname	Municipality		Digital
1	x		1916	1977/1983	H. van Straalen	Simonsstraat 1 - 77	Delft		C+P
2	x		1920	1985		Warmoezierstraat 34	Delft		P
3	x		1923			Hugo de grootstraat 4 - 34	Delft		P
4	x		1923	1976		Jan de wittstraat 2 - 56	Delft		P
5			1999			Kloosterkade 1 - 131	Delft		
C = Calculation		D = Drawing		P = Photo					

	Old					New					
Number	Length [mm]	BxH [mm]	Distance [mm]	σ [N/mm²]	E [N/mm²]	Length [mm]	BxH [mm]	Distance [mm]	σ [N/mm²]	E [N/mm²]	Notes
1		80x150					63x125	600			Old was sloped / balcony used bankirai / showing roof loadings
2											Sloped roof
3							80x180				drijfsteen' walls
4											Cross section shows timber beams
5											Concrete roof

Roof loadings

Some of the requested dossiers showed the loadings on the roof. A house from 1952 (Schiedamseweg) has a gravel layer of 3 cm and a timber decking of 22 cm. This resulted into the following permanent load on the roof beams:

▪ Mastic + gravel	50 kg/m²
▪ Effective load	100 kg/m²
▪ Ceiling	40 kg/m²
▪ Decking + self weight	40kg/m²

Total : 2,30 kN/m² (230 kg/m²)
In this calculation the maximum allowable deflection is checked with the demand of L/400.

Another house from 1956 (van Drimmelenstraat) used a decking of 22 cm covered with mastic and a gravel layer:

▪ Effective load	100 kg/m²
▪ Self weight + finishing	140 kg/m²

Total: 2,40 kN/m² (240 kg/m²)

The Kerkhofstraat that was renovated in 1983 has the following loads:

▪ Roof covering (Mastic)	0,15 kN/m²
▪ Decking + self weight	0,25 kN/m²
▪ Ceiling	0,20 kN/m²
▪ Effective load	1,00 kN/m²

Total: 1,60 kN/m² (160 kg/m²)
Also here the maximum allowable deflection is L/400.

The calculation of Schiedamsesingel (1867 and renovated in 1974) contained a check for both the old and new state.

The old roof structure which had a slope of 37° and 51° used the following values:

▪ Effective load (snow)	50 kg/m² ↓
▪ Tiles	75 kg/m² ↓
▪ Ceiling (37°)	175 kg/m²↓
▪ Ceiling (51°)	215 kg/m²↓

Total of 20 000 kg.

New flat roof structure:

▪ Effective load	100 kg/m²
▪ Gravel + roofing	30 kg/m
▪ Self weight	30 kg/m
▪ Ceiling + insulation	80 kg/m²

Total: 1,80 kN/m² (180 kg/m²)

In these calculations floor wood class 1 and φ: 0,58 are mentioned.

C. STANDARD ROOF STRUCTURES

This appendix belongs to chapter 2.3 and gives important aspects of roof structures according to the literature.

Building typologies

Older houses are typically built with a traditional building method. A masonry wall consisting out of bricks and floors made of timber beams or elements. Later on a cavity wall was used to keep the bearing wall dry. With this method the houses are simple to construct and are flexible during the construction.

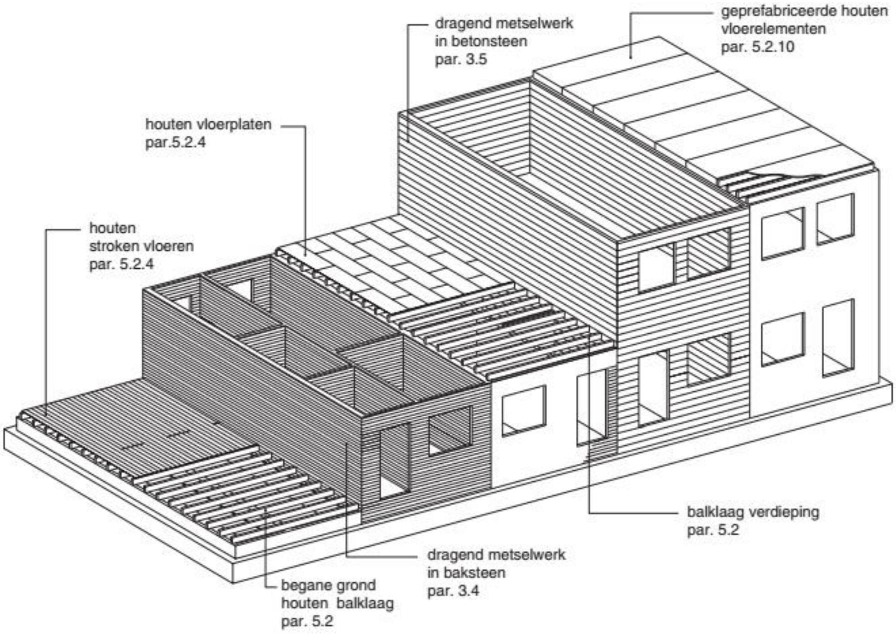


Figure C-1: Traditional building (Jellema 3, 2004)

Modern build techniques often use concrete for the construction. The roofs will then consist out of a hollow core slab or a wide slab floor. These elements have a fast construction time. Especially the use of precast concrete elements or poured concrete as walls and floors are popular when making series of houses.

Flat roof structures

More durable roof coverings came on the market around 1900 which was used for flat roofs on a larger scale. A timber roof structure exist of beams that carry the roof covering.
Generally the timber beams are made from sawn timber however laminated or composite beams might have been used.

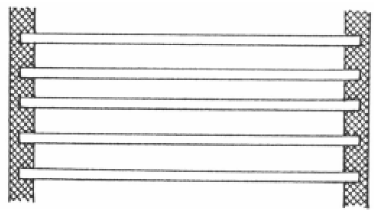


Figure C-2: Single beam layer (Arends, van Eldik, & Janse, 1989)

Two ways of supporting a beam is commonly used :

1. Single beam layer (figure C-2)
   The beams span from wall to wall.

2. Multiple beam layer (figure C-3)
   Larger beams span from wall to wall while supporting smaller beams that carry the covering. These beams can be from timber however the bottom beams might be from steel.

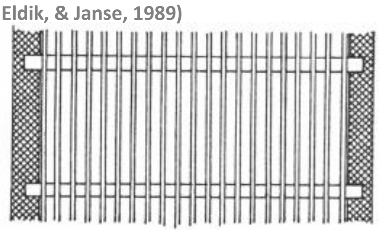


Figure C-3: Multiple beam layer (Arends, van Eldik, & Janse, 1989)

Roof covering

Four kind of roof coverings are standard (Jellema 4a, 2005). They all have the same function: to shield the residents from weather conditions, animals and burglars. However insulating buildings was not commonly done in the past. This led to the following cross sections:

1) Traditional warm roof
   The insulation is on top of the joists which keeps the heat under the decking. The roof covering keeps the water out of the structure. See figure C-4.

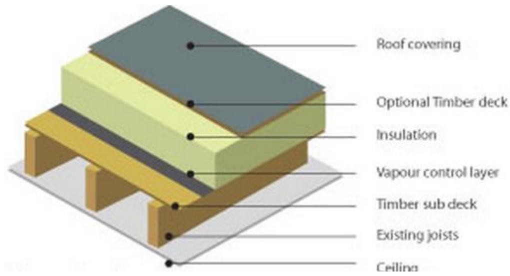


Figure C-4 – Warm roof cross section (Proshield, 2015)

2) Reverse roof
   This is a variant of the warm roof. Here the insulation is on top of the covering. To prevent it from blowing away it is covered by a gravel layer or another heavy material. This extra weight will be removed when making a green roof and thus can be replaced by heavier vegetation or more water in the buffer zone.

3) Cold roof
   The insulation is placed between or below the joists. The space between the insulation and decking is air which has the same temperature as outdoor. Although this area is ventilated, problems related to high humidity can occur like timber rot. See figure C-5.

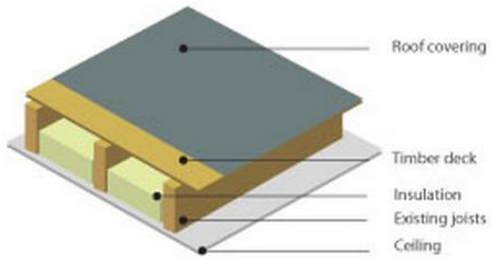


Figure C-5 – Cold roof cross section (Proshield, 2015)

4) Uninsulated roof
   No insulation is present. This option is only chosen when the function of the covered structure allows it. These were used in the past but is unaffordable these days due to high energy bills.

Sometimes there is a gravel layer or tiles present to protect the covering from aging and wind suctions. Other materials than timber have also been used for flat roofs. Although timber is very common, one may find roof structures made of concrete, profiled steel plates or a box structure.

The decking on top of the beams have the main function of carrying the covering and spreading the loads over the beams. Furthermore these slabs or planks protect the lower structure from different weather conditions. At last they can be used as a plate for extra stability. The decking is attached to the beams by means of (wire)nails or staples and are not designed to work together. In the past these decks were planed and consisted of a tongue and groove. Triplex, particleboards or OSB-plates with tongue and groove are popular choices. These should be connected with at least two wire nails or staples. In the past these products often were made of spruce and placed parallel or perpendicular to the direction of the beams. The thickness for planks is minimal 21 mm while boards have a thickness between 14-22 mm (Jellema 3, 2004).

The greatest attention point designing a flat roof is the discharge of water. Residual water can cause problems like water accumulation or frost damages. For this reason a flat roof is never completely flat but has a gradient between 2° and 5°.

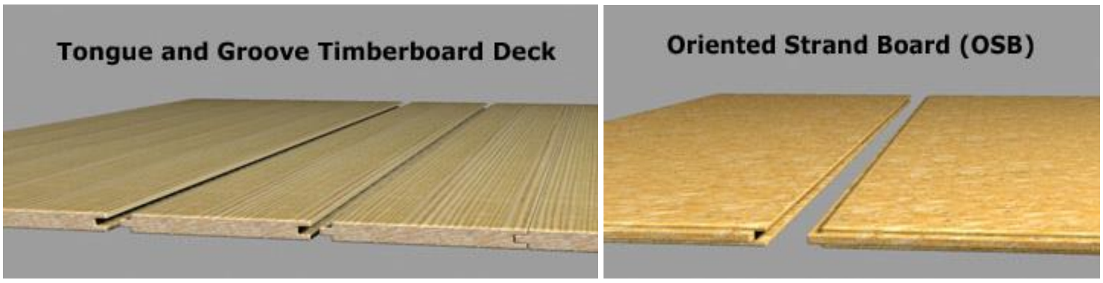


Figure C-6: Standard decking of the roof (BuildingRegs4Plans, 2015)

Anchoring and connecting

A beam simply supported on two walls is common practice. The way they are connected to the wall is important for the evaluation. Standard is to lie them cold on the masonry wall by means of a notch. The anchoring needs to be in vertical and horizontal directions to ensure stability. If there are more beams in a row then they are coupled by a coupling anchor above the supports (figure C-7). When the beams are not in a row but at the edge they can be fastened with different connectors, see figure C-9 where C is used for timber to timber connections. However it may occur that the span is longer than the standard-lengths of timber beams. Instead of making an extra support or using larger beams a joint can be made. In the past three different joints were used to connect beams outside a support, see figure C-8. This was very labor-intensive and thus nowadays the joint is made with a shoe (Jellema 4a, 2005). Making use of a nipped or hooked scarf joints is also a poplar solution when decayed parts have to be replaced.

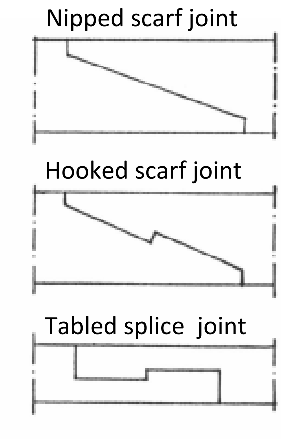


Figure C-9 – Nipped scarf joint, hooked scarf joint and Tabled splice joint (Jellema 4a, 2005)

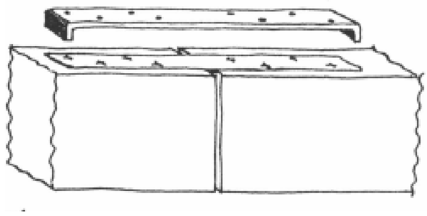


Figure C-7: Coupling anchor (Arends, van Eldik, & Janse, 1989)

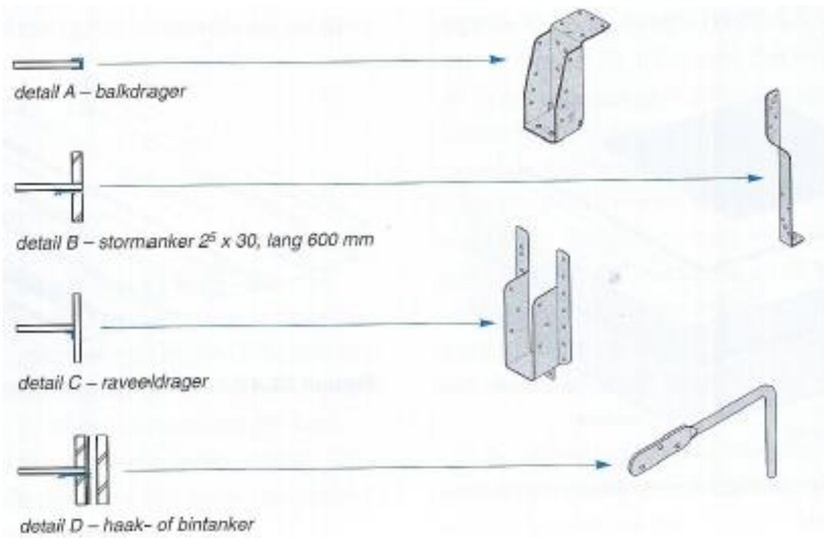


Figure C-8: A = Joist shoe; B = Storm anchor; C= Joist hanger; D= Hook anchor (Bone, 2007)

Whenever the beams are lied into the masonry it is important whether it is an inside or outside wall. The outside walls are wet after a rainfall so that moisture penetrates into the masonry and eventually also into the timber. Here rot can occur and the connection between the timber and wall becomes a critical point. On average the bearing walls are one stone thick.

The decking is attached to the beams by means of double nails.

Figure C-10 shows a standard detail of roof with overhang (left) and without overhang (right).

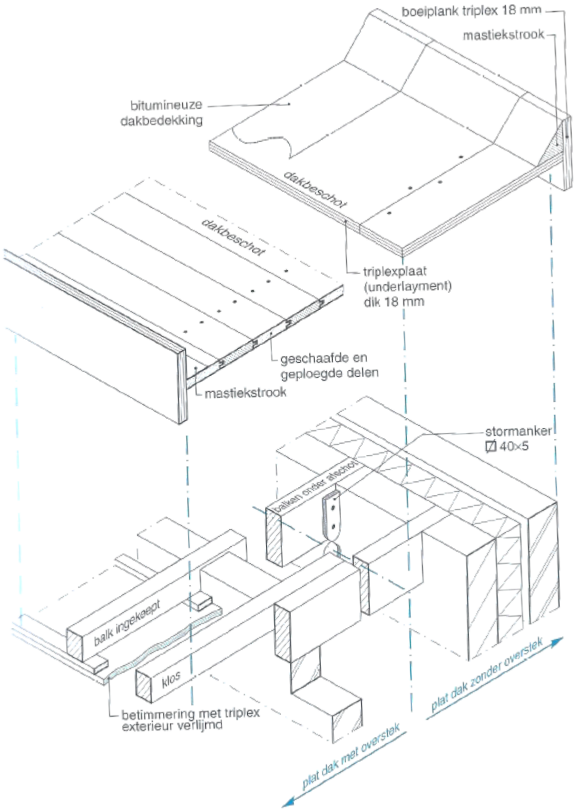


Figure C-10: Detail of roof structure (Bone, 2007)

The dimensions

The length of a beam cannot be standardized. One should keep in mind that timber is a natural product therefore a span of six meter or more is already unusual for roof structures since the size of the beam would get to large. A timber trader uses standard sizes, these are called nominal sizes. The real delivered size may deviate from the nominal within a certain margin. These nominal sizes are also most likely to be found in a timber roof structure. For European softwoods, which is commonly used in housing, a table is given with a moisture content of 20% (see figure C-11).

Gangbare handelsmaten Europees vuren en grenen													
	Breedte (mm)												
Dikte (mm)	38 (34)	50 (45)	63 (58)	75 (70)	100 (95)	125 (120)	150 (145)	160 (156)	175 (170)	200 (195)	225 (220)	250 (245)	275 (270)
16 (12)					X	X							
19 (15)					X	X							
22 (18,5)					X	X	X		X	X			
25 (21)						X							
32 (28)	H	H		H	X	X	X		X	X	X		
38 (34)		H			X	X	X		X	X	X		
44 (40)		H		H	X	X	X		X	X	X		
50 (44)		H	H	H	X	X	X		X	X	X		
63 (58)			H	H	X	X	X	X	X	X	X		
75 (70)				H	X	X	X		X	X	X	X	X
95 (90)					X	X	X						
100 (95)					X	X	X			X	X	X	

De houtdikten 95 en 100 mm zijn in de kwaliteitsklasse A volgens NEN 5466 niet verkrijgbaar en als gedroogd hout in kwaliteitsklasse B moeilijk verkrijgbaar (door grotere kans op droogscheuren).
X = Gangbare handelsmaat, H = Herzaagmaat, ( ) = Afmetingen geschaafd hout

Figure C-11 : Standard trading sizes (Centrum hout, 2005)

These conventional dimensions didn't always existed. They are specified in the NEN 5466 for the first time in a norm. This norm came out around 1983. Before this, different measures or even logs were imported which was then resawn in the desired dimensions. The distance between beams are by default 600/610 mm.

The timber species

During the reconstruction period the standard timber species were harder to get. The largest group of timber species to be found in roofs in the Netherlands is spruce, fir and pine. Some centuries ago oak was popular but this is not expected to be found in houses from the past century. The named softwoods are imported from the following countries (Source: various timber traders):

Spruce: Scandinavia, Russia, Poland, Czech Republic and the Baltic states.
Pine: Scandinavia and Germany (American Pine from USA)
Fir: Europe, USA and Canada

Atmosphere

The temperature in the room below the roof strongly depends on its function (the heat), the insulation and the outside temperature. When no insulation is present the temperature inside is almost equal to the outside. The roof covering is waterproof and thus water vapor inside cannot leave through the ceiling. A good combination between insulation and ventilation is needed to prevent a high humidity.

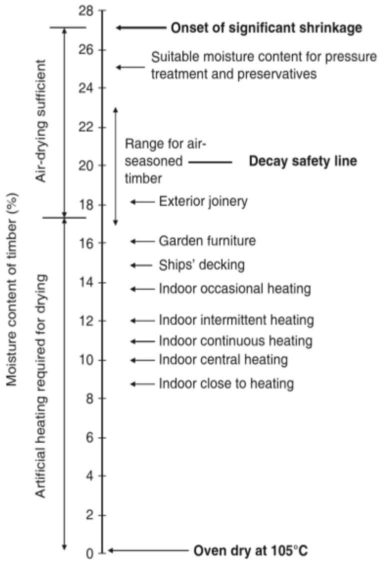


Figure C-13: Equilibrium moisture content (Domone & Illston, 2010)

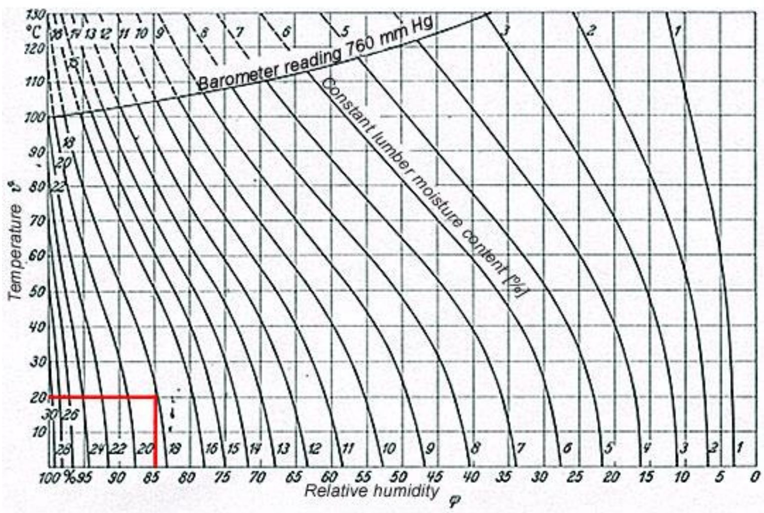


Figure C-12: Sorption isotherms for pine wood (Duken, Fieberg, Schieder, & Topp, 2015)

A moisture content between 10% and 14% is expected to be present in an insulated timber roof.

Defects in houses

Association "eigen huis" shows on their website (Vereniging eigen huis, 2014) which building parts can expect different defects. (joostdevree) used this data and coupled it to certain build periods. The most common relevant defects are listed below.

Period	Common defects
General	▪ Bad water drainage and no/insufficient emergency drains ▪ Low gradient in flat roof ▪ Leakage green roof ▪ Bad condition of roof covering
< 1960	▪ Timber decay due to biological attacks ▪ No or insufficient isolation in walls or roof ▪ No cavity or cavity insulation
1960-1979	▪ No/bad cavity or roof decking insulation ▪ Roof covering in bad condition
1980-present	▪ Roof covering in bad condition ▪ Bad ventilation caused bad condition insulation

Table C-1: Common defects in houses (joostdevree)

Thijssen and Meijer

Thijssen and Meijer concluded that the most common flat roof structure between 1946 and 1965 consisted out of reinforced concrete. Furthermore timber roof structure were almost never insulated in this time. Whenever there was insulation, which wasn't often, it was done poorly. Houses that are built in a row before 1960 usually have only a cavity on the end walls but not in the longitudinal direction. 1960 was a wet year and seems to be the turnaround for better insulation methods, also the making of a cavity wall became mandatory which wasn't always done before this time. Flat roofs are mostly covered with mastic and topped with a gravel layer. Often the roofs don't have an overhang but are finished with edge pieces. A slighty sloped roof in Rotterdam either consists out of timber or lightweight concrete hollow slabs. The type of material is often based on whether the roof is accessible or not. The bearing walls for seperating houses was contucted with 0,5 stone, 1 stone or 1,5 stone sizes in limestone. Bearing walls for seperating rooms were usually 0,5 stone or 1 stone sizes also in limestone. Other materials less common material for walls are concrete blocks or red bricks (Thijsen & Meijer, 1988).

Wittmann and Verhoef

Wittmann and Verhoef did research on roofs structures of 80-100 years old in Slovenia. They note that the first step is visual inspection. During inspection attention is paid on important locations like supports, chimneys, gutters, etc. and on biological attacks. One conclusion becomes clear, no visual deterioration is found when the ventilation of the structure is good. A hammer is used to check the condition of the wood. When deterioration is spotted, the main cause is a high level of moisture. This occurs when there is poor detailing at critical points. The most critical point is the wood which is enclosed in a wall, the damage can only be assessed when parts of the wall are removed. Another reason for a bad roof structure is the interference with the original design. These constructional changes are sometimes the cause for unfavorable deformations. Some of the damaged specimens were tested on their strength. The results showed that the remaining strength was still high despite the damage (Wittmann & Verhoef, 2000).

D. STRESSES AND CONSEQUENCES OF A GREEN ROOF

This chapter belongs to paragraph 2.5.5 and shows the calculations of beams with and without interaction of the decking.

<u>Overview:</u>

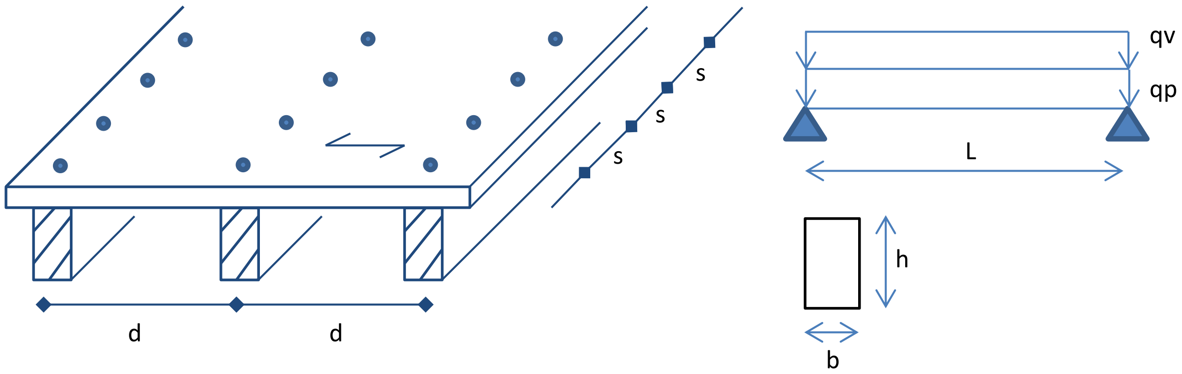


Figure D-1: Overview of roof decking on beams

<u>No interaction</u>

Single beam (according Eurocode)

> $b := 75 : \mathrm{h} := 200 : \mathrm{l} := 4150 : \mathrm{d} := 605 : \mathrm{fm} := 18 : \mathrm{kmod} := 0.90 : \mathrm{gammam} := 1.3 : \mathrm{E} := 9000 : qp := (0.60+1)\cdot d\cdot 10^{-3};\ qv := 1\cdot d\cdot 10^{-3};$

$$qp := 0.9680000000$$

$$qv := \frac{121}{200} \qquad (1)$$

> $qd := 1.2\cdot qp + 1.5\cdot qv, My := \frac{1}{8}\cdot(qd)\cdot l^2;\ V := \frac{1}{2}\cdot qd\cdot l;\ Wy := \frac{1}{6}\cdot b\cdot h^2;\ Iy := \frac{1}{12}\cdot b\cdot h^3;$

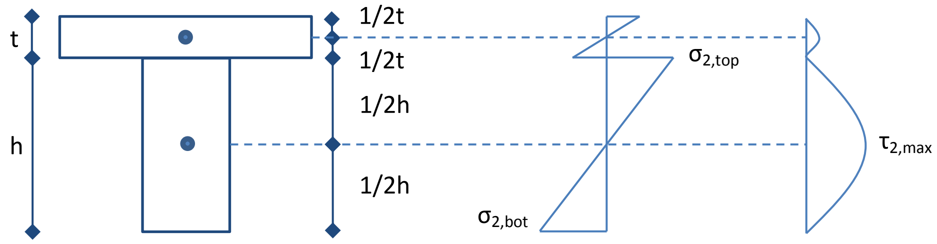
$$qd := 2.069100000$$
$$My := 4.454384344\ 10^6$$
$$V := 4293.382500$$
$$Wy := 500000$$
$$Iy := 50000000 \qquad (2)$$

> $sigm2top := \frac{My\cdot(-0.5\cdot h)}{Iy};\ sigm2bot := \frac{My\cdot 0.5\cdot h}{Iy};$

$$sigm2top := -8.908768690$$
$$sigm2bot := 8.908768690 \qquad (3)$$

> $tau2max := \frac{3}{2}\cdot\frac{V}{b\cdot h};$

$$tau2max := 0.4293382500 \qquad (4)$$



<u>Full cooperation between decking and beam</u>

Beam with decking (according Eurocode)

> $t := 19 : Eply := 4200 : rhoc18 := 380 : rhoply := 520 : rhom := \sqrt{rhoc18 \cdot rhoply}; s := 100 : dia := 4 :$

$$rhom := 20\sqrt{494} \tag{4}$$

> $Bcef1 := 0.1 \cdot l; Bcef2 := 20 \cdot t;$

$$Bcef1 := 415.0$$
$$Bcef2 := 380 \tag{5}$$

> $Beff := b + Bcef2;$

$$Beff := 455 \tag{6}$$

> $kser := \frac{rhom^{1.5} \cdot dia^{0.8}}{30}; ku := \frac{2}{3} \cdot kser;$

$$kser := 947.0369930$$
$$ku := 631.3579953 \tag{7}$$

> $gamma1 := \frac{1}{1 + \frac{3.14^2 \cdot Eply \cdot t \cdot Beff \cdot s}{ku \cdot l^2}}; gamma2 := 1;$

$$\gamma1 := 0.2329743799$$
$$\gamma2 := 1 \tag{8}$$

> $a2 := \frac{gamma1 \cdot Eply \cdot Beff \cdot t \cdot (h + t)}{2 \cdot (gamma1 \cdot Eply \cdot Beff \cdot t + gamma2 \cdot E \cdot b \cdot h)}; a1 := 0.5 \cdot (h + t) - a2;$

$$a2 := 6.456669704$$
$$a1 := 103.0433303 \tag{9}$$

> $EIeff := Eply \cdot \left(\frac{1}{12}\right) \cdot Beff \cdot t^3 + E \cdot \left(\frac{1}{12}\right) \cdot b \cdot h^3 + gamma1 \cdot Eply \cdot Beff \cdot t \cdot a1^2 + gamma2 \cdot E \cdot b \cdot h \cdot a2^2;$

$$EIeff := 5.465380157\ 10^{11} \tag{11}$$

> $sigm2top := \frac{gamma2 \cdot E \cdot \left(a2 - \frac{h}{2}\right) \cdot My}{EIeff}; sigm2bot := \frac{gamma2 \cdot E \cdot \left(a2 + \frac{h}{2}\right) \cdot My}{EIeff};$

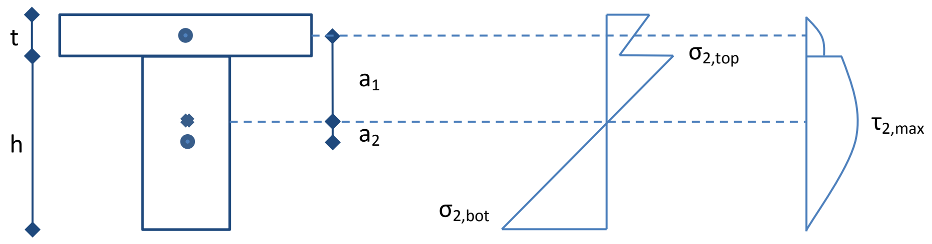
$$sigm2top := -6.861556573$$
$$sigm2bot := 7.808771179 \tag{12}$$

> $tau2max := \frac{0.5 \cdot E \cdot b \cdot (0.5 \cdot h + a2)^2 \cdot V}{b \cdot EIeff};$

$$tau2max := 0.4006244693 \tag{13}$$

> $sigm1top := \frac{gamma1 \cdot Eply \cdot \left(-a1 - \frac{t}{2}\right) \cdot My}{EIeff}; sigm1bot := \frac{gamma1 \cdot Eply \cdot \left(-a1 + \frac{t}{2}\right) \cdot My}{EIeff};$

$$sigm1top := -0.8975208138$$
$$sigm1bot := -0.7459978810 \tag{14}$$



E. DESIGN PROCEDURES

Since 1992 the Building Act became active in the Netherlands which gave regulations about different aspects for all new structures and renovations. This includes rules about the strength and stiffness. For these rules the Building Act refers to a norm and can also overrule that norm. In 2003 a new Act became active and again in 2012. The most important change is regarding to the deformations. The Act of 1992 notes that the end deformation of a floor is determined in the TGB 1990. It is uncertain whether a roofing falls under this category, however when the roof is used intensively it should be considered as a floor. In 2003 and 2012 no requirements are given for the deformations because the regulations should be as simple as possible. Demands for the maximum deflections are therefore often specified in a contract between the client and the construction team.

E.1 THE NORMS

Beginning

Around 1920 the start of normalizing construction principles had begun. This was based on a German example, the first German timber norm was the DIN 104 which prescribed cross sections and dimensions for houses. In these beginning years mainly steel and concrete norms for bridges existed in the Netherlands. Around 1927 the first timber related norm came on the market: "N 1012:1927 Keuringsvoorschriften voor hout als bouwmateriaal en voorschriften voor houtbereiding". This is now known as the NEN 5467 for pine and the NEN 5466 for spruce. The main concern in the N 1012 is the moisture content which caused many problems. Often the required dryness was not applied. These concerns were addressed in an design code V 1004 in 1926 (Comissie Normalisatie Nederland, 1926 nummer 19). This code distinguishes five climate classes. Timber that is used in roof structures is submitted to class 4 which states: Wind dry timber and an average moisture content less than 35%. Furthermore the code notes that when no timber specie is prescribed spruce and fir should be used for inside work and (European) pine for outside work. Regulations of sustainability of the wood was already addressed in this norm. No strength properties are given.

N788 – N795

This N-serie consists out of separated norms that include different aspects. It is divided into the following parts:

- N788: Self-weight
- N789: Effective load and snow load
- N 790: wind load and final provision
- N 791: Stresses
- N 792: Deflections
- N 793: Buckling of steel
- N 794: Buckling of timber
- N 795: Particular rules for steel structures

The values and procedures are the same as the TGB 1955.

TGB 1949

The TGB 1949 is a collection of the N-serie. The main difference is the wind load, there was no distinction between moderate and high wind loads in the N-serie. There are only minor changes in the TGB 1955 compared with the TGB 1949. Actually the TGB 1955 is an revised version of the TGB 1949. The main change for flat roof structures is the calculation of the snow load.

TGB 1955 (NEN 1055)

This norm from 1955 distinguishes six loads and three load combinations. The average self-weight of the common wood species is represented in a table. This value is the minimum value which must be used. Whenever a specie is not in the table the weights should be determined separately. This also counts for the roof finishing. Noted is that the moisture content should always be accounted for. The effective load consists out of a uniformly distributed load or a concentrated load that is caused by persons on a roof. Two kind of wind loads exist, the moderate and the high wind. However because no house is higher than 16 meters the wind load may be neglected.

Timber that is used in construction must comply with the N 1012. Also the maximum allowable stress is based on the N 1012. A table with strength values is given for the common wood species. Additionally a reduction factor is given when rot can occur. A higher stress of maximum 1.5 times the stress in the tables is allowed when three criteria are fulfilled. For the deflection due to the effective load a ratio is given. At last one modulus of elasticity is given for all wood species. The given strength values were planned to be revised shortly after 1955, new aspects like quality classes and the negative influence of moisture must be included.

TGB 1972 (NEN3850)

Due to new constructive aspects and more accurate calculation procedures the TGB commission decided to change the format of the norms. This means a separation of a general part with the loads and the materials.

The TGB 1972 makes a clearer distinction between permanent load, variable load and wind load. The permanent load consists out of self-weight and dead load, also here tables can be used and other values must be determined separately. The variable load is based on snow and persons which is combined into one value. When a roof is intensively used it should be calculated as a floor. Further distinction is made for the decking plates and the beams and also for the edge parts. The last load is the wind load. Like the TGB 1955 it is allowed to neglect this part when a certain criteria is met. It is mentioned that one should take water accumulation into account but no load value is given.

Two main groups of load combinations exist, one for the maximum allowable stress and one for the ultimate guaranteed capacity. The latter consists again out of two parts, an increased load and a reduction of strength. The load and strength both get a coefficient (load factor or material factor) that is based on the required safety.

The TGB 1972 for timber gives an additional reduction method for a concentrated load on the decking of a flat roof. The reduction takes into account the spreading of the load to more beams. This kind of additional information also occurs for determining the strength. Timber has good resistant against short load durations from the variable load it is allowed to use a reduction factor on the variable load because the allowable stresses are based on the permanent loads. Next, the N 1012 which gave demands for timber in constructions in the TGB 1955 is updated to NEN 3180. Two strength classes are distinguished: standard building wood and construction wood. Additionally there are five drought classes from I till V which is characterized by the moisture content and humidity. Class I being the driest. Furthermore the norms distinguishes elastic deformations and creep. The latter is based on the elastic deformation of the permanent load plus one-third of the deformation from the variable load. At last the minimum thickness of the decking for European softwoods is given as 16 mm.

TGB 1990 (NEN6700)

Many things have changed since the last edition, most important is that the deterministic approach has changed to a probabilistic approach. Also for the first time the reference period of a construction is determined which is related to a certain safety target. This norm is not uniform throughout the years, different editions came out with sometimes changes in the calculation procedures. The table below shows the evolution of the TGB 1990.

	1st edition	2nd edition	3rd edition	4th edition
NEN 6700 - General	1991 (C in 1992 and A in 1997)	2005		
NEN 6702 - Loads	1991 (C in 1993 and A in 1997)	2001 (A in 2005)	2007 (C in 2007 and A in 2008)	
NEN 6760 - Timber	1991 (C in 1994)	1997 (A in 2001)	2001 (C in 2002)	2008
A = Supplement sheet C = Correction sheet				

Table E-1: Evolution of TGB 1990

The latest editions were released during the transitional period. These versions became more uniform with the Eurocode which was expected replace the TGB 1990 shortly after. Comparing the different editions, only one important change was found for roof structures. The first edition of the NEN 6702 gave a high uniform distributed load for the decking of 2,5 kN/m². This was replaced in 1997 by 1 kN/m².

One new load, water load, is added to the variable loads due to structure that failed because of water accumulation. In most cases the engineer will prevent this load by making enough emergency drains, use a gradient or a making the roof edge lower.

New are also the load and material factor. In the previous norms these safety factors were already included in the maximum allowable stress. Roof structures have a combination factor of 0 which means that only one variable load needs to be considered. In the combinations there is a distinction between ultimate and serviceability limit state which represent the strength and deformations/vibrations respectively. In the SLS a further distinction is made between deformations with and without creep.

More knowledge about constructing with timber is gained which is visible in the norm by means of a modification factor. The factor takes into account the climate conditions and load duration of a structure. Modern grading methods are also possible that resulted into more strength classes. This makes more economic structures achievable. The third edition of the NEN 6760 changed the Dutch K-strength classes to the European C- and D-strength classes which caused some changes in material properties.

The design value of the strength is determined in the following way:

$$f_d = \frac{f_{rep}}{\gamma_m} * k_{mod} * k_h$$

Where: f_{rep} = Representative value of the strength in N/mm²
γ_m = Material factor
k_{mod} = Modification factor
k_h = Size factor

Eurocode

The first edition of the Eurocode exists since 2002 but became mandatory in 2012. Remarkable is that the wind load can now also cause pressure on the roof due to gusts landing on the far end of the roof. A flat roof is divided into zones with different suction coefficients, only the zone farthest away from the wind side can get pressure. However this will not be governing when also considering snow. Furthermore all of the Ψ-factors are 0 which means no combination between the variable loads is needed.

A table of timber strengths is no longer given in the timber part but redirects the reader to a specific specie norm. Creep factor k_{def} is now dependent of the climate class and the type of material. The different Ψ_2 factors make the relation between creep and load duration possible. The modification factor k_{mod} directly relies on the climate class and the load duration.

The design value of the strength is determined in the same way as the TGB 1990.

E.1.1 WATER ACCUMULATION

An observation can be made about the water load. Common practice is to prevent water accumulation instead of taken the load into account. In the past mainly steel roofs have failed due to this phenomenon.

The main question is whether older houses are equipped with an emergency overflow and high edges. Before the first Building Act and the TGB 1990, no calculation procedure was present concerning water accumulation. However the TGB 1972 mentions to take a water load into account and to use a gradient of 1,5%. The engineer determined if emergency overflows should be present and thus a variety exists. The amount of water that can be stored on a roof depends on the distance between the roof covering and the lowest point of the roof edge or emergency overflow. A risk of failure due to water accumulation is than determined by the stiffness of the roof.

In general no water accumulation is expected when one of the three criteria are fulfilled:

- Sufficient gradient
- Sufficient stiffness
- Sufficient amount of emergency overflows

The NEN 6702 refers to the NPR 6703 for a detailed consideration of the criteria. A calculation procedure for when regular drainage is impossible is given in the NEN 6702 and the EC 1-3 NB.

NPR 6703 gives two roof failure categories, strength and stability. This depends on the critical factor n, which is determined by the ratio between stiffness and critical stiffness of the (cooperative) structure.

E.1.2 IMPACT LOAD

Falling or slipping of a person on top of the roof may not lead to failure of the decking. NEN 6702 mentions this load in 2001 for the first time and is nowadays found in the national annex of EC 1. The idea is that the energy from such an impact must be withstand by the area that is not supported by bearing members. Two methods are given to assess its integrity.

- A practical method: a 0,7 meter drop of a 50 kg filled leather bag
- A conservative method: $F_{rep} = \sqrt{\frac{490}{u}}$ with u = deflection in mm under static design load 0,7 kN

E.2 COMPARISON WITH THE DRAWINGS

The calculation of Van Drimmelenstraat (1955), Schiedamseweg (1950) and Suiestraat (1948) used an effective load of 1 kN/m² from persons on the roof. This complies with the norms from 1949/1955. There is no indication that the tables for the permanent loads were used.

Schiedamsesingel checks in 1974 an existing structure from 1867 with the calculation procedures of the TGB 1972. The new structure is calculated with the variable load of 1 kN/m² along with a reduction for the spreading and for the short load duration. These reductions are not used in Kerkhofstraat which raises the question how many roof structures, that used the TGB 1972, actually have a reduced variable load. The latter case also showed a calculation for the deflection, however creep is not taken into account.

Some retrieved dossiers showed the permanent loads. These obtained loads are used for designing a beam with the Eurocode to check if the structures are overdimensioned.

Schiedamsesingel (1974)

- L = 2800 mm
- Distance between beams = 580 mm
- Permanent load = 0,80 kN/m² (including assumption self-weight)
- Bending strength = C18 (standard building wood)

ULS	Eurocode
Governing variable load	2 kN in middle
Load combination	0,580 * 1,2 * 0,80 = 0,56 kN/m 1,5 * 2 = 3 kN
Moment	2,65 kNm
Maximum stress	0,90 * 18/1,3 = 12,46 N/mm²
Minimal section modulus needed **Section modulus used**	**212584 mm³ (63x150 mm)** **187500 mm³ (50x150 mm)**

Table E-2: ULS calculation of Schiedamsesingel according to the Eurocode

Kerkhofstraat (1983)

- L = 4400 mm
- Distance between beams = 605 mm
- Permanent load = 0,60 kN/m² (including assumption self-weight)
- Bending strength = C18 (standard building wood)

ULS	Eurocode
Governing variable load	2 kN in middle
Load combination	0,605 * 1,2 * 0,60 = 0,44 kN/m 1,5 * 2 = 3 kN
Moment	4,36 kNm
Maximum stress	0,90 * 18/1,3 = 12,46 N/mm²
Minimal section modulus needed **Section modulus used**	**350305 mm³ (75x175 mm)** **500000 mm³ (75x200 mm)**

Table E-3: ULS calculation of Kerkhofstraat according to the Eurocode

Van Drimmelenstraat (1956)

- L = 4260 mm
- Distance between beams = 680 mm
- Permanent load = 1,40 kN/m² (including assumption self-weight)
- Bending strength = C18 (standard building wood)

ULS	Eurocode
Governing variable load	2 kN in middle
Load combination	0,680 * 1,2 * 1,40 = 1,14 kN/m 1,5 * 2 = 3 kN
Moment	5,78 kNm
Maximum stress	0,90 * 18/1,3 = 12,46 N/mm²
Minimal section modulus needed **Section modulus used**	**463967 mm³ (75x200 mm)** **533333 mm³ (80x200 mm)**

Table E-4: ULS calculation of Van Drimmelenstraat according to the Eurocode

Keep in mind that the 2 kN is a load that can occur during the construction. So the comparison here is when the structure is newly build.

A few reasons can be given why the minimal needed section modulus is lower than the used modules even though table 2-2 showed that the norms get stricter. The first reason is that an engineer does not want a unity check close to 1, some extra tolerance is often chosen. Another reason is that the strength classes are not the same. Furthermore in some calculations the deformation is also checked. It is uncertain if the beam sizes are adjusted to fit the deflection requirements. A reason why Schiedamsesingel showed a higher needed section modulus is due to the reduction factors that are used in the existing calculation.

E.3 THE FIRST QUALITY DEMANDS

The first Dutch timber related regulation from 1927 (N1012) gives quality demands for timber as structural material. Different aspects are addressed: general quality, drought condition, wood to be delivered, round and cleaved wood, square-edged wood, wane at sawn wood, heart in sawn wood, sawn sapwood, dimensions of sawn timber and at last the regulations concerning preparation for modified wood.

General quality

Wood needs to be visibly healthy so that defects will not lead to rejecting. The assessment concerns (loose) knots, cracks, chalk rings, rust stains, blue stain and other defects like red streaking. However no specific demands are given.

Drought conditions

Five classes are distinguished:

- Class 1: Room dry wood with 12% moisture or less
- Class 2: Dry wood with 12% - 15% moisture
- Class 3: Air dry wood with 15% - 18% moisture
- Class 4: Wind dry wood with 18% - 35% moisture
- Class 5: Wet wood with more than 35% moisture

Class 1 and 2 must be free from internal dry cracks and have limited hardening crust. The contract documents will prescribe the needed class but if no demands are given class 4 is used for roofs.

Wood to be delivered
The contract documents can prescribe the species. It is noted that in most cases spruce and fir may be mixed. If no specie is prescribed than spruce and fir must be used for inside work and (European) pine for outside work.

Round an cleaved wood
These qualities are related to poles and not to roof beam.

Square-edged wood
These qualities are related to poles and not to roof beam.

Wane at sawn wood
Timber for roofs that is not painted, varnished or stained may be squared sawn. In this square sawn wood some wane is allowed. Square edged wood must be delivered unless the wane is removed during process. The maximum allowed wane for a cross section of 2,5 m^2 is 25% and no more than 2 cm on a surface. For cross sections smaller than 2,5 m^2 this is also 25%. Besides it is allowed on two corners, may not be bigger than 1/20 of the perimeter and not longer than 1/3 of the length.

Heart in sawn wood
Timber for roofs that isn't painted, varnished or stained may be delivered heart cleaved. For members that are modified, heart free wood must be delivered.

Sawn sapwood
For heart free and heart cleaved wood, two sapwood corners may be present. The top side of the decking must be free of sapwood.

Dimensions of sawn timber
The dimensions given are standard for unsawed wood. A table is presented for standard dimensions for trading sawn timber inland and in Mid-European wood. Another table gives standard lengths for spruce and pine from Russia and East sea harbors. At last dimensions for round wood is given which also mentions American pine.

E.4 BACKGROUND INFORMATION ALLOWABLE STRESSES

Background of standard building wood and construction wood

Around 1955 TNO did research (Govers, 1966) about the values for the working stress of Middle European coniferous wood and North European Spruce. The stresses set in the TGB 1972 are 7 N/mm² or 10 N/mm² given that the maximum moisture content is 21% and the quality is according to the N 1012. The derivation of the values for European softwood differ from other species.

It was proposed to let the Middle European coniferous wood represent the applied wood in the Netherlands. A lower probability value of 1/1000 for bending stress was considered sufficient to meet the requirements for structural purpose. Before the timber was tested, they were first graded according to NEN 3180 (KVH 1958) into the two strength classes. Subsequently they were tested in a four point bending test. This led to a bending strength of 13 N/mm² for construction wood and 7 N/mm² for standard building wood.

The same procedure was performed with the North European Spruce. Here the stresses in bending are 11 N/mm² and 7 N/mm² for construction wood and standard building wood respectively.

It is also noted that a lower moisture content comes with a higher bending strength, however, this effect was only visible in the mean values. The lower important values only showed small differences. A remark can be made that beams with low moisture content (12-14%) can increase the standard deviation a lot since the strength is significant higher than for a moisture content of 21%. Therefore it is important to also consider the frequency distribution.

At last the average modulus of elasticity for the North European is spruce 12000 N/mm² and 10800 N/mm² for construction wood and standard building wood respectively. For the Middle European wood these values were measured at 12000 N/mm² and 10100 N/mm². The values are fixed for construction wood at 11000 N/mm² and for standard building wood at 10000 N/mm² due to safety reasons.

A remark can be made about the method for determining the allowable bending stress for other wood species. These are determined with the following formula (CHR, 1982):

$$\bar{\sigma} = t \left(\frac{1 - k * v}{w}\right) \hat{\sigma}$$

Where: t = Duration of load factor: 9/16
k = Accepted failure probability: 2,33 for 1% or 1,96 for 2,5% or 1,64 for 5%
v = Coefficient of variation
w = Safety factor: 1,25 in TGB 1972
$\hat{\sigma}$ = Average stress

Background of characteristic values of material properties

The NEN 3180 (KVH 1958) has been updated throughout the years due to better understanding of the material properties. Nowadays the quality demands for European species are collected in the NEN 5466 (KVH 2010). This norm specifies four quality classes A till D which is based on visual grading aspects. When comparing these classes to the TGB 1972 it shows that quality class B corresponds to construction wood and quality class C with standard building wood. The TGB 1990 and the Eurocode distinguish more strength classes for soft and hardwood. The softwoods are grouped in strength class C. Timber that satisfies the criteria of quality class B may be assigned in a strength class of minimal C24, for quality class C this is C18. The first edition of the TGB 1990 used strength classes K, here standard building wood is assigned to strength class K17 and construction wood to K24. The number in the strength class represents the characteristic bending strength in N/mm². The properties are at a temperature of 20°C and a relative humidity of 65%.

Three criteria of material properties determine a strength class: the 5-percentile value of the bending strength, the average of the elasticity modulus and the 5-percentile value of the density. In other words only 5% of a graded batch has a bending strength or density below a certain value. These values can be determined by visual or machine strength grading methods. The grading requirements are specified in the NEN-EN 14081, NEN 5499 and NEN-EN 1912 and the classification in the NEN-EN 384 and NEN-EN 408.

Loads	TGB 1949	TGB 1955 (NEN 1055)
a) Self-weight	See TGB 1955	Average values given in table:
		- Pine and spruce: 550 kg/m³ (air dry)
		- European pine: 600 kg/m³ (air dry)
		- American pine: 800 kg/m³
		Deviation is allowed only when this has an unfavorable effect
		Notes:
		Average values are given for finishing's.
		10% reduction for self-weight when it works favorable for stress/stability
b) Effective	Persons on roof:	Persons on roof:
	1) 1 kN/m²	1) 1 kN/m²
	2) 1 kN per girder and plate	2) 1 kN per girder (for roof plates 1 kN per plate)
		3) If edge girder is not sufficient supported: 2kN
c) Snow	S = 0,5 kN/m²	Angle of 0…30°: S=0,5 kN/m²
d) Moderate wind	See TGB 1955	Rotterdam wind load: 0,4 kN/m² * - 0,4 = - 0,16 kN/m² (downwards)
		15% reduction possible if one size is >10 m
		Houses with height ≤16m may neglect wind load
e) High wind	See TGB 1955	Rotterdam wind load: 0,7 kN/m² * - 0,4 = - 0,28 kN/m² (downwards)
		15% reduction possible if one size is >10 m
		Houses with height ≤16m may neglect wind load

Load combinations	TGB 1949	TGB 1955 (NEN 1055)
A	a+b+c	a+b+c
B	a+b+d	a+b+d
C	a+b+e	a+b+e
Effective load for roofs does not need to be combined with snow or wind		

Maximum allowable stress	TGB 1949		TGB 1955 (NEN 1055)	
for pine and spruce in N/mm²	Bending σb	MOE E	Bending σb	MOE E
Fir, spruce and European pine	7	10000	7	10000
American pine	10	10000	10	10000

Notes:

1) When the timber is exposed to water and air and isn't protected against rot, the values need to be multiplied with a factor 0,67
2) A higher allowable stress (maximum of 1,5x) is permitted when complied with the following:
- When extra attention is paid to the grading and quality of the timber
- The moisture content isn't higher than class 2 (10-15%) of N 1012
- The moisture content will not rise considerable

Maximum allowable deflections	TGB 1949	TGB 1955 (NEN 1055)
Span of ≥ 5250 mm	1/500	1/500

Loads	TGB 1972 - General	TGB 1972 - Timber	
Permanent Loads (a)			
Self-weight	Table:	Table:	
	- Softwood: 400-650 kg/m3	- European softwood: 500 kg/m³	
	- Hardwood: 500-1000 kg/m3	- Spruce: 450 kg/m³	
		- Western hemlock: 500 kg/m³	
		- American pine: 600 kg/m³	
Dead load	Average values in table		
Variable Loads (b)			
Uniform distributed load for decking and beams (max 10m²)	1 kN/m². Reduction possible with minimum of 0,5 kN/m²		
Line load of 1 meter for decking	2 kN/m. Reduction possible with minimum of 1 kN per plate		
Concentrated load for beams	2 kN. Reduction possible with minimum of 1 kN	Reduction possible for spreading of the load	
Combination between these loads is not needed.			
Wind load (c)			
Under pressure	800 N/m² * (0,4 + 0,3) = 560 N/m²		
It is allowed to neglect the wind load when torsion and biaxial bending is not present.			
The wind load is based on:			
- q = 800 N/m² for a height of 12 meters.			
- cd = +0,4 for 0° - 65°			
- co = -0,3 for under pressure			

Factors	TGB 1972 - General	TGB 1972 - Timber
Load	γ1 = 1,3 – 1,5	
Material	γm = 1,0 – 1,4	
Combination		0,85 for variable load
		0,70 for wind load

Load combinations	TGB 1972 - General	TGB 1972 - Timber
1) Maximum allowable stress	a+b+c ≤ σ	σpermanent + 0,85 * σvariable ≤ σ
		σpermanent + 0,70 * σwind ≤ σ
2) Ultimate guaranteed capacity	γ1(a+b+c) ≤ U	
	U = U*/γm	
	U* = allowable force	

Maximum allowable stress σ in N/mm²		TGB 1972 – Timber (drought classes I,II,III)	
		σb	E//
Group 1	Standard building wood	7	10000
	Construction wood	10	11000
Group 2	Construction wood	12	12000
For drought class IV: all values should be 90% ; For drought class V: 80%			
Higher values are possible for laminated timber			

Maximum allowable deflection	TGB 1972 - General	TGB 1972 - Timber
End deflection	≤0,004 L	
Elastic deflection		≤0,0035 L (beams)
		≤0,0025 L (decking)
Beware of water accumulation, use a camber or a gradient of 1,5%		

Safety	TGB 1990 – NEN 6702	TGB 1990 - NEN 6760
Reference period	50	
Safety class	3	
Loads	TGB 1990 – NEN 6702	TGB 1990 - NEN 6760
Permanent load (Grep)		
Self-weight	Table:	
	- Softwood: 550 kg/m3	
	- Hardwood: 500/750 kg/m3	
Dead load	Average values are given in a table:	
	Flat roof with beams and decking (no gravel): 36 kg/m²	
Variable load (Qrep)		
Uniformly distributed load (max of 10m² and for decking plates/planks)	Prep = 1,0 kN/m²	
Concentrated load (only UGT and on area of 0,1 m x 0,1 m)	Frep = 1,5 kN (2 kN if beam directly under load)	
Line load (only UGT on decking and length: 1m width: 0,1m)	qrep = 2 kN/m	
Wind load	0,20 kN/m²	
Based on : Cdim = 0,96 ; Cpi = 0,3 ; Ceq = 1 ; pw = 0,68 (Area II, high built density) ; ϕ = 1		
Water	0,50 kN/m²	
Snow	0,8 * 0,7 kN/m² = 0,56 kN/m²	
Water can be neglected when a gradient is used, the roof has enough stiffness or there are enough emergency drains.		
The water load strongly depends on the amount and height of emergency drains, the given value is based on a case.		
Factors	TGB 1990 – NEN 6702	TGB 1990 – NEN 6760
Load factor ultimate limit state	Permanent: γf;g;u = 1,2 / 1,35	
	Variable: γf;q;u = 1,5	
Load factor serviceability limit state	Permanent: γf;g;ser = 1,0	
	Variable: γf;q;ser = 1,0	
Correction factor for the load	Ψt = 1 (t = 50 years)	
	Roofs: Ψ = 0	
	Ψk = 0,6 (creep)	
Material factor		γm = 1,2 (ULS) ;γm = 1,0 (SLS) ;
Modification factor		kmod = 0,85 (ULS); kmod = 1,0 (SLS);
Size factor		kh = (150/h)0,2 (for 40 mm ≤ h < 150mm)
		kh = 1,0 (for h ≥ 150 mm)
Creep factor		Ψkrp = 1,0 (load duration I)
Material factor and size factor are based on sawn timber.		
The modification factor is based on climate class I and load duration III short.		
Load combinations	TGB 1990 – NEN 6702	TGB 1990 - NEN 6760
ULS: Fundamental combination	γf;g;u * Grep + γf;q;u * Ψt * Q1;rep + Σ γf;q;u * Ψi * Qi;rep	
SLS: Incidental combination	γf;g;ser * Grep + γf;q;ser * Ψt * Q1;rep + Σ γf;q;ser * Ψi * Qi;rep	
SLS: Momentaneous combination (for creep)	γf;g;ser * Grep + Σ γf;q;ser * Ψi * Ψk * Qi;rep	
Representative values in N/mm²	TGB 1990 – NEN 6702	TGB 1990 – NEN 6760
Most common softwood		14,16,18,20,22,24
Maximum allowable deflection	TGB 1990 – NEN 6702	TGB 1990 – NEN 6760
Additional	≤0,004 L	
End	≤0,004 L	

Safety	NEN-EN 1990	NEN-EN 1995
Design class	3 (50 years)	
Consequence class	CC2	
Roof class	H (only accessible for maintenance)	
Climate class	2 (average moisture content not higher than 20%)	
Loads	NEN-EN 1991	NEN-EN 1995
Permanent load		
Self-weight (dry)	Tables:	
	C14: 3,5 kN/m^3 - C16: 3,7 kN/m^3	
	C18: 3,8 kN/m^3 - C22: 4,1 kN/m^3	
	C24: 4,2 kN/m^3	
Dead load	Tables give values for individual materials.	
Variable load		
Uniformly distributed load (maximum area 10m^2)	q_k = 1,0 kN/m^2	
Concentrated load (0,1 m x 0,1 m)	Q_k = 1,5 kN (2 kN if beam directly under load)	
Line load (length: 1 meter width 0,1 meter)	2 kN/m	
Wind load	0,36 kN/m^2	
Only half of the roof feels pressure of the wind, the other half suction. The value is based on:		
C_sC_d = 1,0 ; q_p = 0,72 kN/m^2 (area II; high built density) ; C_{pe} = 0,2 ; C_{pi} = 0,3		
Water load	Same as TGB 1990	
Snow	0,8 * 0,7 kN/m^2 = 0,56 kN/m^2	
Snow and water load are based on:		
μ_i = 0,8 & s_k = 0,7 kN/m^2 & C_e = C_t = 1,0		
Factors	NEN-EN 1990	NEN-EN 1995
Load factor ULS	Permanent: 1,2 / 1,35	
	Variable: 1,5	
Load factor SLS	Permanent: 1,0	
	Variable: 1,0	
Ψ-factor	Roofs: Ψ_0 = 0 and Ψ_2 = 0	
	Snow,water,wind: Ψ_0 = 0 and Ψ_2 = 0	
Material factor		γ_m = 1,3 (sawn timber)
Modification factor		k_{mod} = 0,90
Size factor		k_h = min of $(150/h)^{0,2}$ and 1,3
		k_h = 1,0 (for h ≥ 150 mm)
Creep factor		k_{def} = 0,80 (sawn timber)
The modification factor is based on climate class 2 and load duration short		
Load combinations	NEN-EN 1990	NEN-EN 1995
ULS: Fundamental combination	$\Sigma \gamma_{G,j} G_{k,j} + \gamma_{Q,1} \Psi_{0,1} Q_{k,1} + \Sigma \gamma_{Q,i} \Psi_{0,i} Q_{k,i}$	
SLS: Characteristic combination	$\Sigma G_{k,j} + Q_{k,1} + \Sigma \Psi_{0,i} Q_{k,i}$	
SLS: Quasi-permanent combination	$\Sigma G_{k,j} + Q_{k,1} + \Sigma \Psi_{2,i} Q_{k,i}$	
Strength	NEN-EN 1990	NEN-EN 1995
Strength class		
The norm no longer supports a table but recommends values in specific norms. These strength values are the same as the TGB 1990.		
Maximum allowable deflection	NEN-EN 1990	NEN-EN 1995
Additional + long term	≤0,004 L	W_{inst} = l/300 – l/500
		$W_{net,fin}$ = l/250 – l/350
		W_{fin} = l/150 – l/300

F. DETERIORATION OF THE STRENGTH

Not every degradation mechanism is important for roof structures. This appendix will focus on those degradations that can affect the structural safety. Information about how to prevent, notice and repair the mechanism is given. Service life models in combination with the damage accumulation models can predict the residual life time. For this, information about the expected loads and the state of the structure is needed. This appendix belongs to chapter 2.8.

F.1 MECHANICAL DEGRADATION

Wood strength is susceptible to time and loads. The loss of strength over time in combination with long term loading is known as the duration of load effect (DOL). Failure due to this effect is referred to as creep rupture. Literature can be found for modeling this effect on timber materials. These models are based on empirical data, cumulative damage theories, fracture mechanics, deformation kinetics or energy based models. When the load combination with only a permanent load is used, the strength gets a modification factor of 0,60.

<u>Impact</u>
Timber can resist higher loads for a short period of time better than a long period. It is expected that timber roofs still have most of the full strength. High loads come from snow or maintenance but are only present for a short time. Furthermore the rate of loading is also low.

<u>Prevention</u>
Preventing the DOL behavior is not possible in timber, it is better to anticipate the negative effects. The duration of the variable load is, according to the Eurocode, short for roof structures which indicates that there will be no significant loss in strength. The TGB 1990 and the Eurocode both give a modification factor (k_{mod}) on the resistance to prevent failure due to load duration. The maximum allowable stress in the norms before the TGB 1990 took a factor of 9/16 into account.

<u>Visual</u>
The degradation happens in the material on molecular level. Bonds break which leads to extra deformation. Excessive deformation of the beam could indicate a reduced strength.

<u>Repair</u>
If repair is needed, the beam can be replaced or strengthened.

F.2 PHYSICAL DEGRADATION

High temperatures, wind, UV radiation and drying can cause physical degradation. In relation with roof structures only drying cracks are expected. These cracks occur when during construction the moisture content of the applied timber is higher than the moisture equilibrium. Due to its anisotropic property the timber swells and shrinks different in the transverse, radial and longitudinal direction. This leads to different deformations and stresses in the three directions. Eventually these stresses will cause cracks in radial direction which are usually safe provided that a specific crack depth is not exceeded (Fech, 1987). The crack depth and width depends on the wood quality. Not all cracks are caused by drying, other reasons can be mechanical damage or cracks during growth period of the tree.

Figure F-1: Cracks in timber

Impact
The exposed cracks are vulnerable for fungal spores or insect eggs. Furthermore the available cross section is reduced by (unexpected) dry cracks that results in a decreased resistance for bending and shear stresses. The rupture strength is based on test results.

Bending strength: A crack depth of 60% of the width is harmless for bending stresses. This is valid for individual cracks or the summation of multiple cracks in the horizontal direction.
Shear strength: The shear strength is more susceptible for cracks. The allowable shear stress is based on the ratio full cross section/cracked cross section. A crack depth of 45% is harmless, higher percentages will reduces the allowable stress (Fech, 1987).

Figure F-2 and F-3 show the harmless crack depths in relation with stresses and angels. For one sided cracks the same rules apply.

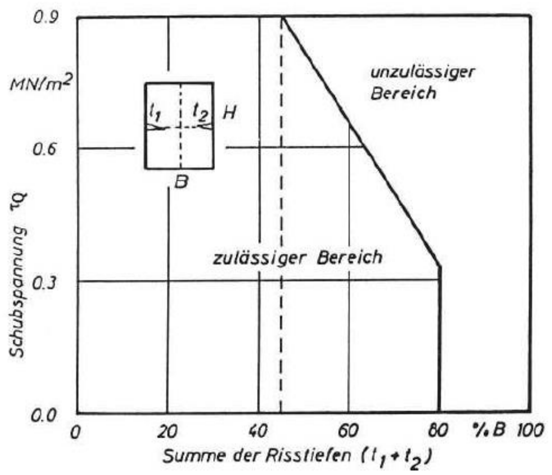


Figure F-3: Safe zone for crack depth and shear stress (Fech, 1987)

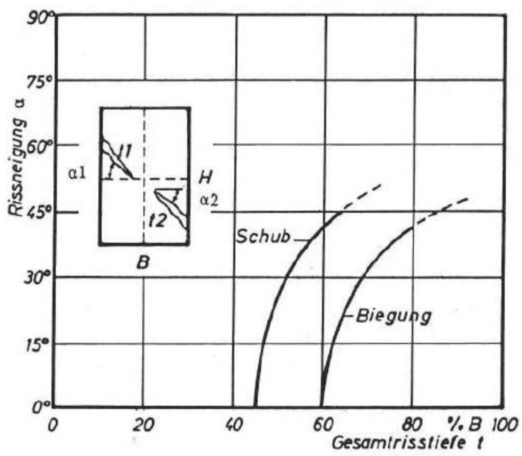


Figure F-2: Safe zone for crack depth and shear (Fech, 1987)

Prevention
During the design of a timber structure it is necessary to estimate the equilibrium moisture content. The timber should than only be installed when its moisture content is close to the expected equilibrium so that only the seasonal moisture variation occur during its lifetime.

Visual
By means of visual inspection the cracks can be identified. Measuring their depths will indicate if a critical situation is present.

Repair
Repair is only needed when the crack depths are too high. Strengthening of replacing the affected beam is necessary to extend the service life of the total structure.

F.3 CHEMICAL DEGRADATION

The Eurocode 5 states that connections that make use of metal should be protected against corrosion. High humidity (or water) plus oxygen leads to corrosion of metals. The minimal protection depends on the climate class. Class I requires no protection for nails, screws, bolts, dowels or steel plates thicker than 3 mm. Corrosion gives the metals a larger surface which pushes the surrounding timber apart. Furthermore iron stains attack the cellulose components in timber which can lead to loss of strength in the joint (Li, Marston, & Jones, 2011). Other chemicals like strong acids (PH < 2) or strong alkalis (PH>10) can also cause degradation, however these are not expected in roof structures.

Figure F-4: Chemical stain due to fasteners (Renovate, 2015)

Impact
This defect occurs very locally and only slightly weakens the timber beam. If the beams and decking were working together, this could now be damaged.

Prevention
It is not expected that timber roof structures come in touch with chemicals other than metallic salts. The metal connections can be protected against corrosion and the surrounding conditions can be specified. Especially when a green roof is placed on top the water resistance must be intact.

Visual
The iron stains are a dark stain around the nails but are hard to spot because the affected side is protected by the decking and thus not visible. If the corrosion process is in an advanced stage than the structural integrity of the joint might be in danger.

Repair
Replacement of the metals is needed and the cause for high humidity must be fixed.

F.4 BIOLOGICAL DEGRADATION

An important aspect in timber engineering is the biological durability. Because wood is an organic material, the most common degradation is by living organisms. The European standard (EN-335) and the Eurocode (EC5) use different classes to define the durability. One parameter that is essential for these classes is the moisture content. Other important parameters for biological degradation are temperature, oxygen and pH-values. These are harder to influence since they are based on the living circumstances for persons. Three types of attacks can be distinguished: insects, fungi and bacteria. Only the relevant organisms for roof structures are considered here. The natural durability determines the resistant against a biological attack without treatment, this is specie dependent. Furthermore the heartwood has a higher natural durability than the sapwood. Table F-1 gives an overview of the different attacks.

Insects

The main type is the beetles with a larvae cycle. Eggs are laid in the cracks or splits and eventually a larvae will bore tunnels into the timber. When the larvae is in the adult stage, a metamorphosis takes place and the beetle will exit the timber through a hole. This hole is often the only sign of a beetle attack. The damaged caused inside the beam is not visible which makes the assessment of the damage hard. Important living conditions are the temperature and the present of nutrient. Secondary is the moisture content.

There is a wide range of aggressive beetles but only a few are active in the Netherlands. Most of the insects only attack the sapwood however some species also attack the heartwood. Spruce and fir are expected to have degradation in the full cross section. Furthermore a higher natural durability is expected for softwoods before

1900 than wood from the 20th century. This is due to the slower growth of the tree and the later cutting down. This led to a less sensitive sapwood and a more toxic heartwood (RDMZa, 2001).

Impact

The larvae reduces the weight and thus also the strength of a timber beam. Extra attention should be given to the house longhorn beetle which can cause much damage in a short period of time. These beetles also prefer timber in roof spaces.

Fungi

Fungi that feeds from timber can cause loss in weight and strength of beam. This is known as rot. Two types of fungi exist: wood-destroying and wood-disfiguring. The difference lies in their effects. Wood-destroying fungi (Brown rot, white rot and soft rot) attacks the cellulose and lignin which eventually reduces the strength of a timber beam. Wood-disfiguring fungi (mould and blue stain) only affects the appearance and does no mechanical damage, however, the coating of a beam can be deteriorated (Blass, Timber engineering step 1: Basis of design, material properties, structural components and joints, 1995).

The 'Rijksdienst voor de Monumentenzorg' notes that there is sometimes a misinterpretation for timber under a lead roofing. Here 'vervilting' can occur which has the same characteristics as white rot. However vervilting is caused by acid and only affects the aesthetical appearances, not the mechanical resistance (RDMZb, 2001).

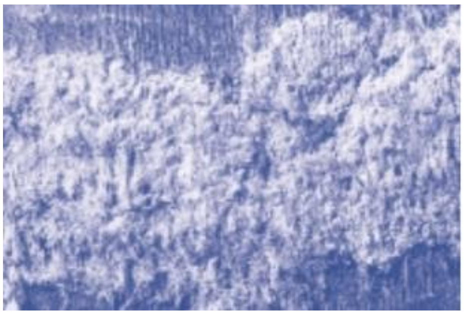


Figure F-5: Vervilting, (RDMZb, 2001)

Certain living conditions need to be fulfilled in order for fungi to grow. The conditions depend on the fungal type. In general the moisture content should be between 20% and 30%. If the high moisture content is of short duration, than fungi is not expected. The ideal temperature is between 20°C and 30°C. Temperatures lower than 10°C or higher than 35°C cause a slower decay whereas <2°C or >38°C completely stops the decaying process (Clausen, 2010). Other important conditions are pH-value (5-6) and free oxygen.

Impact

The organisms are fed by the available nutrients in the wood. This leads to a change in the cross section and a lowered weight which results into loss of strength. In 2002 van de Kuilen (van de Kuilen, 2004) gave graphs of the relation between weight loss and strength loss based on a research of Wilcox in 1978 (see figure F-6). Unfortunately this was only done for brown rot since the data of white rot was insufficient. Note that the loss is strength is significant to the weight loss.

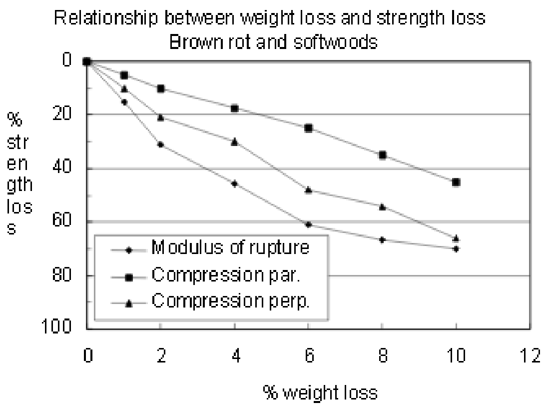


Figure F-6: Relationship between weight loss and strength loss
Brown rot and sofwoods (van de Kuilen, 2004)

Another study (Winandy & Morrell, 1992) tested Douglas-fir on two types of brown rot and two types of white rot. The results are showed in figure F-7. It becomes clear that brown rot causes more mechanical damage than white rot.

Fungus	Type	Incubation time (days)	Sample size	MOE (GPa)		MOR (MPa)		WML (kJ/m³)		EMC (%)	Stress wave (m/s)	Weight loss[b,c] (percent)
				Mean	SD	Mean	SD	Mean	SD			
P. placenta	Brown-rot	3	12	12.91	1.33	83.63	6.96	3.11	1.13	11.7	29.6	4.2
		34	12	12.26	2.30	79.07	15.68	2.64	1.16	11.9	30.0	2.7
		49	12	12.43	1.53	79.72	14.08	3.41	1.18	11.9	31.0	2.1
		84	12	10.60	1.80	62.88	13.03	2.05	0.86	11.9	30.5	7.4
		119	12	10.67	2.03	60.65	5.87	1.92	0.79	12.2	31.5	12.2
		177	12	7.32	1.34	33.16	8.18	0.53	0.18	12.7	35.1	19.8
G. trabeum	Brown-rot	3	12	12.71	2.01	87.39	16.33	2.73	0.95	11.8	29.9	2.5
		34	12	12.27	1.38	73.02	9.02	2.56	1.10	11.8	31.9	3.6
		49	12	12.74	1.07	76.99	5.47	2.97	0.98	11.8	29.0	7.0
		84	12	10.65	1.48	61.08	9.57	1.75	0.94	12.0	30.1	7.9
		119	12	10.56	2.21	53.10	10.73	1.06	0.45	12.0	31.5	11.5
		177	12	6.65	1.58	25.10	7.79	0.48	0.40	12.3	33.5	18.0
T. versicolor	White-rot	7	12	13.31	1.57	88.28	10.17	3.22	1.22	11.8	30.2	3.7
		35	12	13.33	1.28	85.19	8.88	3.29	1.41	11.5	30.0	5.2
		50	6	12.56	2.38	91.34	16.21	3.90	2.21	11.9	31.8	0.7
		85	12	13.22	1.55	85.26	9.86	2.91	1.60	11.7	30.5	4.8
		120	12	12.67	1.14	78.40	7.06	2.41	0.96	11.9	32.0	6.5
		178	12	13.84	1.71	87.86	6.54	3.70	1.44	12.2	33.3	2.5
B. adusta	White-rot	7	12	12.85	1.74	82.60	12.99	3.01	2.49	11.9	32.2	3.8
		35	12	13.59	1.55	89.17	9.41	2.69	0.86	12.0	31.3	2.6
		50	12	13.35	1.78	90.06	14.95	2.91	0.98	11.9	32.0	1.7
		85	12	13.08	1.15	86.09	6.93	3.25	1.19	11.9	32.4	5.1
		120	12	13.32	1.39	88.35	7.84	3.51	1.45	12.0	32.2	5.6
		178	12	14.24	2.04	87.44	7.74	2.97	0.96	12.0	30.9	4.1
(Control)	(Steam sterilized)		12	13.37	1.88	83.13	11.74	2.40	0.98	11.8	31.5	3.9
(Control)	(Untreated)		36	12.60	1.65	93.94	12.20	3.60	1.47	12.1	31.8	—

[a] MOE is modulus of elasticity, MOR modulus of rupture, WML work to maximum load, EMC equilibrium moisture content, and SD standard deviation.
[b] Basic specific gravity (oven-dry weight to oven-dry volume) of unsterilized controls = 0.468.
[c] Compared to unsterilized control.

Figure F-7: Effect of brown and white rot on Douglas-fir with different incubation times (Winandy & Morrell, 1992)

Bacteria

Bacteria is generally not seen as a cause of degradation but more as a contributor for fungi decay. Generally it changes the color and texture. Furthermore it can increase the permeability. When bacteria is present over a longer time, than excessive absorption of moisture is possible. Mainly timber poles suffer from bacterial decay (Clausen, 2010).

Degradation

Insects (1)	Bionomial name	Preferred living conditions (1)	Attacks (2,3)	Characteristics (2,4)	Control (3,4)
Dead Watch Beetle 	xestobium rufovillosum	MC: 30% and higher Temperature: 30°C larvae stage: 8-10 years	Attacks sap- and heartwood. Mainly oaks are affected but also some softwoods. Partially-decayed hardwood is preferred.	Exit holes circular of 3 mm diameter. The bore dust is bun-shaped, cream-colored and has a grainy feeling. Mainly the supports with high moisture content are affected.	Only sapwood affected: Spraying or brushing (repeatedly) Locally affected: Injection (repeatedly)
Common Furniture Beetle 	Anobium punctatum	MC: Fibre saturation point Temperature: 22°C-23°C Larvae stage: 4-8 years	Attacks mainly sapwood of soft- and hardwoods that is in use for a longer time. Also heartwood that is affected with fungi.	Exit holes circular of 1-2 mm diameter. The bore dust is lemon-shaped, cream-colored and has a grainy feeling.	Spraying or brushing
House Longhorn Beetle 	Hylotrupus bajulus	MC: 28%-30% Temperature: 28°C-30°C Larvae stage: 3-5 years	Attacks sapwood of softwoods mainly in roof spaces of houses. Pine only sapwood is affected, for spruce and fir sap- and heartwood. Severe damage possible.	Exit holes oval of 6-10 mm diameter. The bore dust is sausage-shaped, cream-colored and has grainy feeling.	Spraying or brushing Sometimes replace rot parts
Powder-post Beetle 	Lyctus brunneus	MC: 16% Temperature: 26°C-37°C Larvae stage: 5-18 months	Mainly sapwood from hardwood	Exit holes circular of 1-2 mm diameter. The bore dust is fine, cream-colored and talc-like feeling.	Pesticides (difficult to reach affected zone) Replacement of parts

1)	Lecture service life prediction of structures & wood durability/deterioration; Gard, W.; Timber Structure 2 2014
2)	Construction Materials; Domone, P. and Illston, J.; 4th edition 2010
3)	Beoordeling en restauratie van historische (eiken)houten balklagen; van Reenen, M.J.; 2003
4)	Insecten in hout: beoordeling en bestrijding; Rijksdienst voor de monumentenzorg; 2001

(Continues on next page)

Fungi (1)	Bionomial name	Preferred living conditions (1)	Attacks (2,3)	Characteristics (2,5)	Control (3,5)
Brown rot (6) 		MC: 30%-60% Temperature: 24°C-35°C pH: 4-6	Attacks cellulose and hemicellulose. Mostly softwood is affected but sometimes also hardwoods.	Color becomes dark brown with cuboidal cracking. Timber with high moisture is vunarble, especially in damp masonry.	Reduce the humidity Replace infected parts Protect the uninfected parts Check for attacks of dead watch beetle
White rot (7) 		MC: 30%-60% Temperature: 24°C-35°C pH: 4-6	Attacks cellulose, hemicellulose and lignin in hardwoods	Color becomes white and bleached. Timber becomes fibrous but doesn't crack. White rot is hard to detect because of the appearance staying long intact.	Reduce the humidity Replace infected parts Protect the uninfected parts
Soft rot (8) 		MC: 30%-200% Temperature: 24°C-35°C pH: up to 11	Attacks cellulose of the s2 layer. Mainly soft- and hardwoods that are in contact with the ground	Disfigures almost the same as brown rot	
Blue stain 	Grosmannia clavigera	MC: 30%-40% Temperature: 28°C-40°C	Attacks cell contents like starch or extractives. No mechanical damage is done but coating can be deteriorated.	Disfigures the wood by leaving a stain	

1)	Lecture service life prediction of structures & wood durability/deterioration; Gard, W.; Timber Structure 2 2014
2)	Construction Materials; Domone, P. and Illston, J.; 4th edition 2010
3)	Beoordeling en restauratie van historische (eiken)houten balklagen; van Reenen, M.J.; 2003
5)	Schimmels in hout: oorzaken en oplissingen; Rijksdienst voor de monumentenzorg; 2001
6)	Examples of fungi for brown rot: Dry rot (Serpula lacrimans), Wet rot (Coniophara puteana), Poria vaillantii, Gloeohyllum spp.
7)	Examples of fungi for white rot: Coriolus versicolor, Fomes fomentarius, Stereum spp.
8)	Examples of fungi for soft rot: Chaetomium, Ceratocystis, Kretzschmaria deusta

Table F-1: Biological degradation processes

G. ASSESSMENT OF EXISTING STRUCTURES

This appendix belongs to chapter 3 and shows background information about the case studies, test setups and test results.

G.1 BACKGROUND CASES

Figure G-1: Overview of situation before demolishment (Source: Bing.com/maps)

Green area: Rusthofstraat, built in 1923. Roof structure is not renovated but in the calculations, it is noted that a beam is rot. In chapter 2 it was shown that this can occur due to direct contact with the outer wall. The case study shall not focus on these houses.

Figure G-2: House number 10, original (left) and renovated (right) view of Kerkhofstraat

Yellow area: Kerkhofstraat, built in 1923 and a renovated roof structure in 1983. Note the new part on top. The original structure was a sloped roof, see figure G-2. Ten roofs beams of the new structure are obtained. Unfortunately no detail or cross section drawing was found that showed the roof covering. However during site visit insulation on top of the beams was visible.

Blue area: Kerkhoflaan, built in 1923. Three roof beams of the original structure are obtained. Renovation took place in 1983, this did not affect the structure but the window frames were renewed, the internal layout changed and the roof covering changed. This original covering was topped with insulation – bituminized glass fleece – gravel. The ceiling consisted out of reed. Assumed is that during the renovation the beams are reinforced with a wooden bar on each side in the tension zone attached with nails.

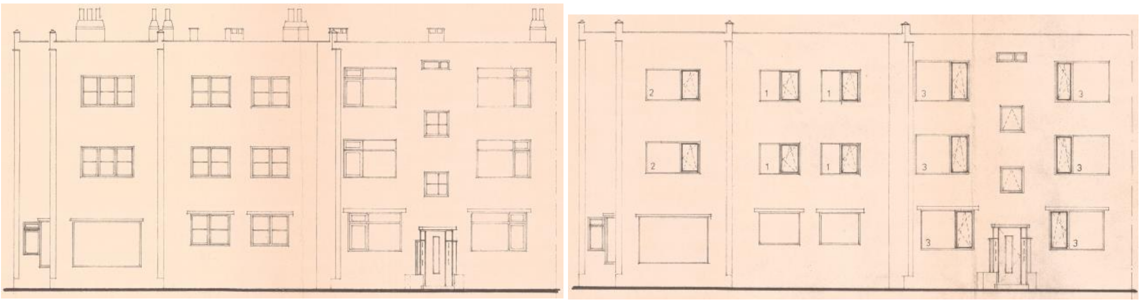


Figure G-3: Original (left) and renovated (right) view of Kerkhoflaan

Roof structure

Both houses were built with the traditional method. The roof structure consist out of a single beam layer that is simply supported in a notch of the wall. For Kerkhofstraat the anchoring consisted out of storm anchors which were sometimes still attached to the obtained members. Figure G-4 shows the beam plan of Kerkhofstraat. This drawing was not found for Kerkhoflaan but based on a photo, the ground plan and the length of the beams a likely situation can be sketched (figure G-5). The insulation in both cases is placed so that a warm roof arises. Before this, Kerkhoflaan probably had an uninsulated roof.

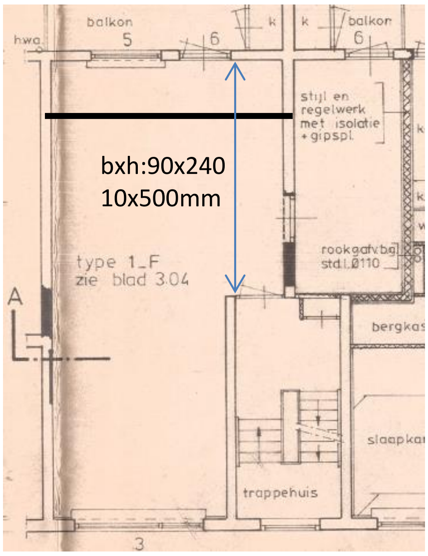


Figure G-4: Beam location on Kerkhoflaan

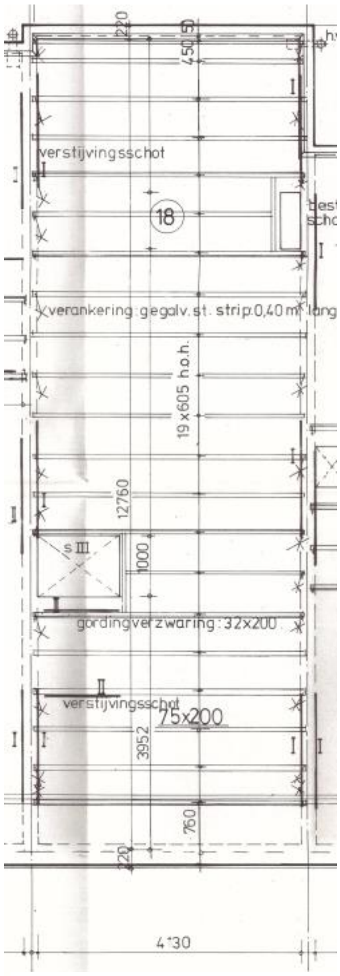
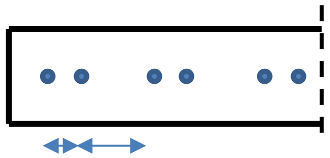


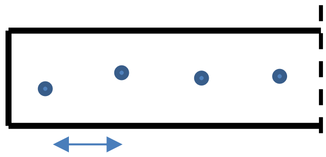
Figure G-5: Beam plan Kerkhofstraat

Decking

The decking is unknown but was attached by nails of ϕ4mm to the member. There were still some nails in the obtained beams which can be used to estimate the situation. The protruding length of the nails is around 19 mm which should equal the height of the decking. A different pattern in the nailing is observed:



4-5 cm 10 cm



≈10 cm

The decking of Kerkhoflaan might have existed out of planks. Modern timber suppliers sell tongue and groove planks with a width of 11 – 15 cm. This matches the measured distances of the nails.

Kerhofstraat probably had timber plates as decking due to single nails in row. Also the nails aren't in the middle of the beam but varies a lot which makes sense when a wide plate blocks the sight of the beam.

Anchoring

In both cases the members spanned between two inner walls which protects the ends from weather conditions. However members from the Kerkhoflaan have an orange-red color on the beam ends. This is caused by red oxide primer which is used in the past for protection against rot and rust. Furthermore signs of a coupling anchor are visible. The storm anchor of Kerhofstraat is placed 10 cm from the end which equals the bearing length.

Dimensions

<u>Kerkhofstraat:</u>
Dimensions: ≈75x200 mm
Length: ≈4000 mm
Distance between beams: 605 mm

<u>Kerkhoflaan:</u>
Dimensions: ≈90x240 mm
Length: ≈4600mm
Distance between beams: Unknown, estimate 500 mm based on photo.

Species

The specie is unknown with the given information, this will be determined later on. However in the calculation of Kerkhofstraat the maximum allowable stress of 7 N/mm² is used which indicates coniferous wood.

Loads during service life

The norms prescribe the loadings that need to be taken into account however it is hard to predict the loads that actually occurred during its life time. Especially the variable load has a high uncertainty, the permanent load can often still be found on drawings or can be measured on situ. The load history can have an influence on the strength.

The permanent loads were as followed:

<u>Kerkhofstraat:</u>

- Roof covering (Mastic) 0,15 kN/m²
- Decking + self weight 0,25 kN/m²
- Ceiling 0,20 kN/m²

Total load = 0,60 kN/m²

<u>Kerkhoflaan:</u>

No load is found, an estimation is made:

- Self weight + finishing 0,80 kN/m² (based on standard values for reed roof in TGB 1949)
- Added part 0,60 kN/m² (insulation + gravel layer)

Total load = 1,40 kN/m²

Variable loads are from maintenance or weather related. The former is usually governing but also hard to predict. Weather conditions over the years are documented and extreme years can be found. When the extreme value of either loads has (multiple times) occurred, it can result in deterioration of strength by means of cracks. For roof structures from houses is almost every time governing, roofs are prepared for loads of 1 kN/m².

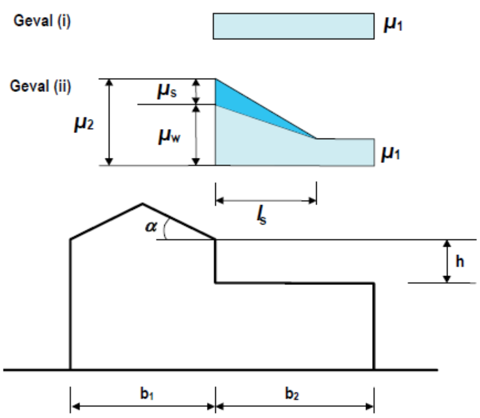
Water

The water load is prevented by making use of drains. When these are clogged by leaves or dirt, the water level rises until the emergency overflow. For both cases it's uncertain what height is used for the overflow or whether the drains were ever clogged. A minimal needed height can indicate if overloading could occur. A roof structure that is designed for the maintenance load can take 1 (kN/m²) / 10 (kN/m³) = 0,10 m of water assuming the beams do not deflect. In practice an emergency overflow height of 3 cm is common but also can be determined by the engineer.

Snow

In 1956 Rotterdam started to keep track of the snow thickness. Weather stations Westerkade and Waalhven reported in February 1969 a snow thickness of 23 cm (KNMI, 2015). This thickness is only measured one time in history, in other years a thickness of 15 cm was measured. A thickness of 50 cm is needed for the snow load to equal the prescribed maintenance load. A high snow thickness is only expected to be present in the north of the Netherlands. Actually only a small part in the north is determined in EC 3 as 0,70 kN/m² (35 cm) on ground level, however the National Annex requires to use this value for the entire Netherlands. This allows 12 cm more snow in Rotterdam than the one time maximum. Keep in mind that the values are from ground level. Snow on roofs can accumulate near a raised edge, an obstacle or higher adjacent roofs. Depending on this height the beams may be subjected to locally high forces. However keep in mind that an edge beam is only loaded from one side and therefore has already some rest capacity.

A higher adjacent roof is present on Kerkhofstraat, but the obtained beams are not from this location(!). The following shows how a snow accumulation calculation according to the Eurocode can lead to excessive values. This calculation is purely informative and is a warning for these kind of situations.



$\mu_1 = 0{,}80$
$\mu_2 = \mu_w + \mu_s = 0 + 2{,}86 = 2{,}86$

$l_s = 2\ h = 2 * 1 = 2$ m → minimal 5 m (thus the full length of the beam)

$s = \mu_2 * C_e * C_t * s_k = 2{,}86 * 1{,}0 * 1{,}0 * 0{,}70 = 2$ kN/m^2

Note that his is higher than the maintenance load of 1 kN/m². Some deterioration of the strength might have occurred depending on the actual accumulation and duration of time.

Figure G-6: Snow accumulation according EC1

In general the load history is not expected to cause any damage. Critical parts were accumulation of snow or water is possible should get extra attention.

G.2 NEN 8700

Safety	NEN 8700
Residual life time	min. 15 years
Consequence class	CC2 (House with 4 layers or more)

Loads	NEN 8701
Permanent load	
Self-weight (dry)	The real values from measurements or weightings may
Dead load	be used.
Variable load	Adjustments are possible (constraints in loads or lower
	realistic weight)
Wind load	Measurement of pressure coefficient
Water load	No reduction allowed
Snow	No reducion allowed

Factors	NEN 8700
Load factor ULS	Permanent: 1,3 (6.10a), 1,15 (6.10b)
Rebuilding level	Variable: 1,3
Load factor ULS	Permanent: 1,2 (6.10a), 1,1 (6.10b)
Rejection level	Variable: 1,15
Ψ-factor	Roofs: Ψ0 = 0 and Ψ2 = 0
	Snow,water,wind: Ψ0 = 0 and Ψ2 = 0
Material factor	May be reduced due to in-situ measurements. This is
	often neglectable due to other uncertainties.

Load combinations	NEN-EN 1990
ULS: Fundamental combination	$\Sigma \gamma_{G,j} G_{k,j} + \gamma_{Q,1} \Psi_{0,1} Q_{k,1} + \Sigma \gamma_{Q,i} \Psi_{0,i} Q_{k,i}$

G.3 IN SITU EVALUATION OF TIMBER

G.3.1 STATE OF THE ART IN-SITU TESTING METHODS

In 2003 a thesis was made by van Reenen called "Beoordeling en restauratie van historische (eiken) houten balklagen". In this research a list is provided with different non-destructive grading methods for timber. Distinction is made between the application of the methods. A summary of this is list is given below and adjusted with new modern grading techniques (Kasal & Tannert, 2010) (Monk, 2011). Not all methods are used for determining the strength, some only give an indication of the decayed part. One should keep in mind that wood is an anisotropic material and thus the properties are directional dependent. Furthermore some techniques give only local results, the reliability of these results for a global level should then be researched. All members should be checked separately because local constrains might be different and a high variety within the timber specie can exist.

The third column indicates whether the method is suitable for roof structures. Due to local constraints a method might not be applicable. Constrains can be: accessibility, time to evaluate, non-user friendly method or if the method reduces the effective cross-section. Reasons for (not) recommending the method is given in the last column.

Physical methods				
Method	Procedure	Quantifiable property	Suitable for roof structures	Notes
Visual inspection (1)	During inspection attention is paid to grain slope, knots, etc.. Visual grading is possible and defects can be observed.	Strength class and surface decay	x	Simple and good for first impression. Visual grading norms must be adjust for older beams. Only exposed parts are visible.
Awl / Screwdriver (1)	A sharp object is struck into the member. Based on the diffuculty and fracture of the fibers, information abuot the condition is gained.	Surface decay	x	Simple and good for first impression
Holes drilling (1)	Holes are drilled into the member, the resistance determines softer or hollow parts. Furthermore the sawdust gives indications about the condition.	Internal decay	(x)	Needs experienced user for identifying sawdust
Core drilling (1)	A bore extracts a specimen from the member. This sample can be visually evaluated.	Internal decay	(x)	Semi-destructive and location dependent but internal parts become visible.
Borescope (Endoscope) (1)	An optical device is used for inspection inaccessible places.	Level of surface decay (/ experts might be able to assess rest strength)	x	Recommended when beams are covered by ceiling
Species Identification (2)	Macroscopic: The surface layer is removed and specie characteristic (color, size growth rings, etc.) are observed. Micropscopic: Sample is taken for miciscopic evaluation.	Wood specie	x	Sampling is often necessary but might be to destructive for this phase. Checking historical records and making an educated guess is prefered.
Dendrochronology (2)	Cores are extracted and studied for the tree rings	Age and specie of the wood		The age of a roof structure can often be determined by studying archive dossiers.
1) van Reenen	2) Kasal & Tannert	3) Monk		

Acoustic and dynamic				
Method	Procedure	Quantifiable property	Suitable for roof structures	Notes
Sounding (with hammer) [1,2]	Member is struck with an object, the resulting sound indicates the condition.	Location of decay	x	Fast and easy but only serious levels of decay are detected.
Velocity Measurement [1,2]	Sonic stress waves from impact of an object. Accelerometers detect stress waves and record time.	Level of global decay	x	Calibration with sound material is needed. Also possible when end face is unreachable.
Frequency Spectrum Analysis [1,2]	Hamer induces stress wave, the accelerometer detects the wave and an oscilloscope transforms it into a frequency spectrum.	Strength and level of decay	x	Multiple scans can produce maps of extent and location. Good basis for determining residual strength. Only one member face required. Calibration with sound material is needed.
Acoustic emission technique [1]	A pressure test measures the acoustic emissions, which is related to the loss of weight.	Strength		Uses pressure tests so not applicable in situ.
Dynamic stiffness measurement [1]	Member is struck with hammer to induce a wave while the propagation speed and the damping is measured	Dynamic E-modulus, density and level of decay	(x)	The surroundings influence the measurements
SASW [1]	A graphic representation of a vibration is made	Location of decay		High investment costs considering non detailed results
Ultrasonic Technique [2]	A transducer converts an electrical current into a wave signal, the recieving parts analyse the wave.	Level of decay	x	Contact and non contact options exist. Configuration near beam end is possible but results will neglect the condition of the ends.
Ultrasonic Echo Technique [2]	A sensor measures sonic waves due to reflection of acoustic waves on material inhomogenities.	Level of decay	x	A clear echo indicates no damage while a unclear echo is hard to interpret.

Electromagnetic methods				
Method	Procedure	Quantifiable property	Suitable for roof structures	Notes
Pulse radar (e.g. GPR) [1,2]	A device generates electromagnetic waves pulse and are reflected when contrast of permittivity is interfered.	Detects defects (e.g. cracks, holes,)	(x)	Results are difficult to interpret, gives more qualitative information
Moisture content meter [1,2]	A device generates a magnetic field, moisture content can be measured because water has a higher dielectric constant than timber	Risk of decay	x	Fast and easy but density must be guessed

Electrical methods				
Method	Procedure	Quantifiable property	Suitable for roof structures	Notes
Moisture content meter [1,2]	The meter is driven into the member and the resistance from moisutre is measured between two pins.	Risk of decay	x	Fast and easy but surface is affected
Shigometer (Vitalometer) [1,3]	First a small hole is drilled, the meter is inserted and generates an electric current. The electrical resistance is measured by a probe with two wires.	Level of decay		Detects decay in early stages. Predrilled hole is needed. Reliability is debatable. Used for standing trees.
Radiographic (Source: gamma rays and X-rays) [1,2]	A source sends radiation through the beam. On the other side a recording medium is placed. The resulting image shows the internal structure.	Local density, condition, flaws, hidden internal material and composition	(x)	Good for assessing interal parts, however there are limitations like a 2D image representing a 3D situation.
Tomography (CAT scan) (Source: gamma rays) [1]	A source sends radiation through the timber while moving along the beam. The energy loss is measured.	Density, internal condition and hidden defects	(x)	Same notes as Radiographic plus it takes much time to perform.
Infrared [1]	A source generates heat into the member. The heat flows easier to sound parts (with higher density) than affected parts.	Location of decay		Not practical due to low conductivity of wood.

Mechanical methods				
Method	Procedure	Quantifiable property	Suitable for roof structures	Notes
Splinter test [1]	A sharp tool is struck under an angle into the wood to pry out a splinter. The sound indicates the condition.	Surface decay	x	Simple method for first indication. Member is damaged.
Compression test [1,2]	A sample is extracted from the member and compressed parallel to the grain. The results are correlated with the modulus of rupture.	Strength, modulus of elasticity and density	(x)	Semi-destructive but gives good indications of local conditions. The test setup must be present.
Penetration resistance (Pildoyn) [1,2]	A steel rod is shot with a spring onto the member. The depth depends on the impact energy. A correlation exists between the depth and the density.	Surface decay and density	(x)	Only surface properties. Best for poles and standing trees. The density is debatable.
Penetration resistance (Decay Detection Drill or resistograph) [1,2,3]	A device drills with constant force into the member while the resistance of the drill is measured.	Local profile, internal/surface decay and density	x	Quantative results. Cracks and decay is detected. Only local results, more measurements needed for global results.
Penetration resistance (resistance diagram, variant DDD) [1]	A device drills with constant force into the member while the resistance of the current is measured.	Internal decay	(x)	Semi-destructive.
Penetration resistance (direct correlation with strength) [1]	A rod is struck into the wood by a hamer with constant energy, the amount of strucks needed for 1 cm penetration is tracked. Correlation exists with laboratory tests.	Strength	(x)	Penetrations need to be perpendicular and parallel to the grain for valid results. Laboratory results need to be present.
Drilling speed (Silbert drill) [1,3]	A device drills with constant force into the member while the speed of penetration is measured.	Level of Decay	x	Makes small holes.
Fractometer (I or II) [3]	A sample is extracted from the member and compressed/bend until failure. The measurements are the fracture moment, angle and energy of failure.	Location of decay and parameter for bending (and compression) strength	(x)	Aim for the center of the heartwood. Semi-destructive method, test setup must be present. Must be compared with decay-free samples.
Screw withdrawal (several techniques)[1,2]	The required force to pull out a screw is measured. Correlation exists between screw withdrawal and MOR/MOE.	Level of decay and density	x	Simple to perform but it's semi-destructive (small holes) and the measuring points might be limited.
Extensometer [1]	A device applies a bending moment while the deflection is measured.	Bending stiffness		Long setup time and requires enough space.
Static stiffness measurement [1]	A known load is applied (dead load technique) and the deflection is measured.	Static E-modulus global/local	(x)	Good indication of bending stiffness but requires enough space for setup and the influence of the surroundings.
Stress distribution [1]	The strains are measured under no load and a known load and the E-modulus is guessed. Stresses can be caclulated.	Stresses in member	(x)	Semi-destructive and the E-modulus must be guessed.
Tension micro-specimens [2]	A saw cuts out a triangular specimen along the fibres of the member. The sample is tested on tension until failure.	E-modulus and maximum tensile strength	(x)	Semi-destructive and test setup needed. Sensitive to grain deviation and other aspects but a good specimen has a high correlation.
Hardness test (Piazza and Turrini) [2]	A steel rod is pushed into the member while the required force is measured. Five measurements are needed for the average.	E-modulus	(x)	A specie dependent coefficient must be known. The correlated properties are test dependent.

G.3.2 LITERATURE REVIEW OF IN-SITU TESTING

Literature about in situ assessment of older timber beams can be found. Different researchers reported good correlations of NDT and SDT with destructive tests. Because not every method is available for the author and a recommendation is essential, recent results of popular methods are described below. A standard procedure is using regression analysis for correlation between NDT and destructive tests.

(Tannert, et al., 2013) note that the best results are gained when different methods are combined. SDT is often necessary to gain reliable results. A list with ND and SD methods is provided for their effectiveness to assess structural timber (see table G-2).

Methods	Determine species	Measure MC	Locate deterioration	Quantify deterioration	Assess strength	Determine stiffness	Identify hidden details
Visual inspection			Limited				
Remote visual inspection			Limited	Limited			Yes
Species identification	Yes						
Moisture measurements		Yes					
Digital radioscopy			Yes	Limited			Yes
Ground penetrating radar		Limited	Limited				Limited
Infrared thermography		Limited	Limited				Limited
Stress waves			Limited	Limited	Limited	Estimate	
Resistance drilling			Yes	Yes	Limited		Limited
Core drilling			Yes		Estimate	Estimate	
Tension micro-specimens					Estimate	Estimate	
Glue line test		Limited	Limited		Limited		
Screw withdrawal			Limited		Limited		
Needle penetration			Limited		Limited		
Pin pushing			Yes	Limited	Estimate		
Surface hardness			Limited		Limited		

Table G-1: Effectiveness of NDT and SDT methods to assess structural timber (Tannert, et al., 2013)

Table G-3 shows how different methods are correlated with the wood properties. Their effectiveness is expressed in the coefficient of variation (R) or the coefficient of determination based on regression analysis. The table is informative because many of the used reports were unclear about the testing conditions (e.g. sound wood or structural timber) and the results (e.g. global or local MOE). Other factors that play a role and are uncertain: different wood species, procedures, age and sizes. It is therefore not recommended to directly use the values in the table. Despite this, a global indication about good working methods in situ can still be gained. These shortcomings are also encouraging for doing test independent of other results.

The UNI is the Italian standard. The UNI 11035 is for visual strength grading and the UNI 11119 is developed for grading timber on-site. The findings with UNI 11119 are not reliable but can be improved when combined with NDT values (Cavalli & Togni, 2011).

	Visual grading	Hardness tests				Stress waves			Core drilling
Method	UNI 11119	Resistograph	Drill resistance (equipment unknown)	Pilodyn	Piazza and Turrini	Free vibration	Various methods	Ultrasonic	
Property									
Density		R^2 = 0,442 [1]		R^2 = 0,4866 [1]				R = 0,40 (longitudinal) [1]	R^2 = 0,62 [6]
		R^2 = 0,722 (with biological damage) [1]		R = -0,86 (P.6J) [3]				R = -0,31 (transversal) [1]	
		R^2 = 0,0039 [5]		R = -0,83 (P.4JR) [3]				R^2 = 0,00 (longitudinal) [7]	
		R^2 = 0,67 (longitudinal) [7]	R = 0,06 - 0,88 [6]	R^2_{adj} = 0,77 [4]				R^2 = 0,02 (transversal) [7]	
		R^2 = 0,43 (transversal) [7]		R^2 = 0,0003 [5]					
				R = 0,02 - 0,89 [6]					
				R^2 = 0,24 (longitudinal) [7]					
				R^2 = 0,30 (transversal) [7]					
Compression strength		R^2 = 0,581 [2]		R^2 = 0,57 (transversal) [7]					
Bending strength	R^2 = 0,1374 [5]	R^2 = 0,58 (longitudinal) [7]	R = 0,86 - 0,93 [6]	R^2 = 0,2169 [1]		R^2 = 0,675 [3]		R^2 = 0,556 [3]	
		R^2 = 0,49 (transversal) [7]		R^2 = 0,27 (longitudinal) [7]				R = 0,32 - 0,80 [6]	
				R^2 = 0,20 (transversal) [7]				R = 0,42 (longitudinal) [1]	
								R = -0,25 (transversal) [1]	
Global MOE		R^2 = 0,33 (longitudinal) [7]		R^2 = 0,00 (longitudinal) [7]		R = 0,91 [3]	R^2_{adj} = 0,56 [4]	R = 0,85 [3]	
		R^2 = 0,56 (transversal) [7]		R^2 = 0,01 (transversal) [7]			R^2_{adj} = 0,80 [4]	R = 0,61 (longitudinal) [1]	
							R^2_{adj} = 0,70 [4]	R = -0,38 (transversal) [1]	
							R^2 = 0,76 (longitudinal) [7]	R^2 = 0,16 (longitudinal) [7]	
							R^2 = 0,51 (transversal) [7]	R^2 = 0,33 (transversal) [7]	
								R = 0,30 - 0,74 [6]	
Local MOE	R^2 = 0,0671 [5]	R^2 = 0,22 (longitudinal) [7]		R^2 = 0,00 (longitudinal) [7]	R^2 = 0,0601 [5]		R^2_{adj} = 0,66 [4]	R^2 = 0,0897 [5]	
		R^2 = 0,48 (transversal) [7]		R^2 = 0,00 (transversal) [7]			R^2_{adj} = 0,70 [4]	R^2 = 0,48 (longitudinal) [7]	
							R^2_{adj} = 0,63 [4]	R^2 = 0,25 (transversal) [7]	

Table G-2: Effectiveness of different methods to the wood properties

1) Specie unknown; (Teder, Pilt, Miljan, Lainurm, & Kruuda, 2011)
2) New spruce; (Calderoni, De Matteis, Giubileo, & Mazzolani, 2009)
3) Fir wood from 1400-1500; (Ceccotti & Togni, 1996)
4) Fir; (Cavalli & Togni, 2011)
5) Larch and spruce from 1879-1942; (Piazza & Riggio, 2008)
6) Martime Pine; (Machado, Lourenco, & Palma, 2011)
7) Chestnut; (Faggiano, Grippa, Marzo, & Mazzolani, 2009)

The best results are gained when different methods are combined. (Kasal & Tannert, 2010) reports that multiple researchers found good correlations of screw resistance in combination with other methods. Screw withdrawal alone underestimated the density because of decay. Screw withdrawal in combination with stress waves are a good indicator for the static MOE, certainly when there is less decay. The combination allows a predicting equation for the stiffness and strength.

	Joist ID										
E	1	2	3	4	5	6	7	8	9	MPD	SEE
										-----(%)-----	
Static E (10^9 pa)	7.27	6.34	10.07	11.65	8.01	8.44	10.39	7.24	5.91		
v (10^3 m/s)	5.03	4.34	5.10	4.96	4.75	4.93	4.97	3.96	4.24		
Top-E	6.87	9.14	9.19	9.49	7.20	8.69	10.71	7.36	6.66	2.40	18.4
Bot-E	7.93	7.29	9.90	10.83	9.34	8.29	9.85	5.92	5.96	0.86	11.2

Figure G-7: Measured and predicted MOE with combination screw withdrawal and stress waves; MPD = mean percentage deviation; SEE = percentage standard error; (Cai, Hunt, Ross, & Soltis, 2002)

(Cavalli & Togni, 2011) review a different combination. It is first noted that the UNI 11119 gives unreliable results for the local modulus of elasticity.

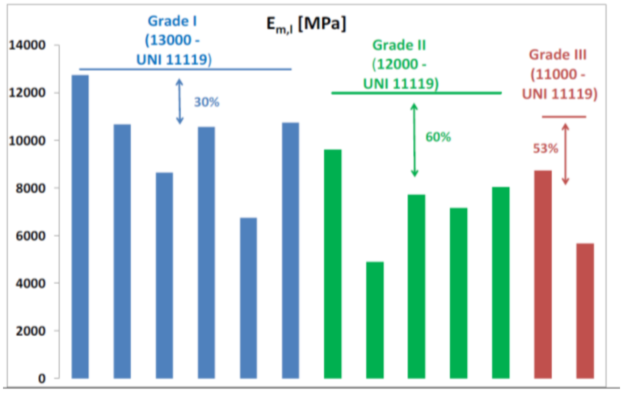


Figure G-8: Local MOE compared to UNI 11119 (Cavalli & Togni, 2011)

The most optimal solution is searched for a combination between different stress wave measurements (Esw, El and Ef), a Pilodyn and two parameters (knot index (KI) and slope of grain (SoG)) of the UNI 11119. The methods are used to predict the local or global MOE.

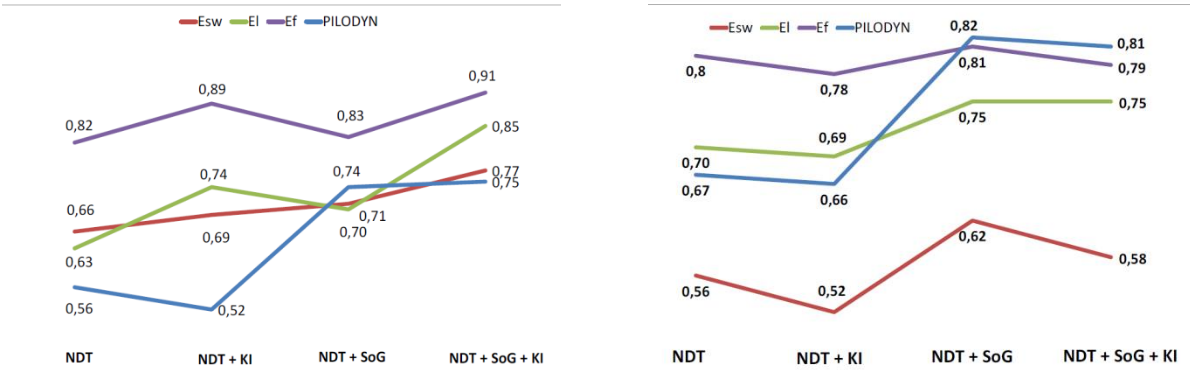


Figure G-9: $R^2_{adjusted}$ with different methods and multiple regression models compared to MOE local (left) and global (right) (Cavalli & Togni, 2011)

G.4 EXPERIMENTS PLAN AND SETUP

This paragraph describes the plan for the 13 obtained beams. The batch consists out of 10 roof beams from 1983 and 3 from 1923. Non-destructive testing has the preference since specimen requirements for semi-destructive tests are often unclear. The test methods have been chosen in consultation with the supervisors. After testing the following question can be answered:

<u>What non-destructive testing method can be used to predict the bending strength from destructive testing results and can thus best be used during site visit?</u>

Three methods are used to determine the reference properties (bending strength, modulus of elasticity and density): Visual grading, Resistograph and velocity measurements. Other methods that make use of simple tools are added to better understand wood behavior and its condition.

Hypothesis

The literature study (Chapter 3.2 and G-3) about different grading methods showed that the Resistograph works well for measuring growth rates and finding internal defects. This shows good correlation ($0,5 < R^2 < 0,7$) with the gross density and bending strength and medium correlations ($0,3 < R^2 < 0,5$) with elasticity modulus. Velocity measurements due to a vibration induced by a hammer is commonly used for measuring the elasticity modulus because of its high correlation ($0,7 < R^2 < 1,0$). Visual grading determines the strength class but these rules apply for fresh timber. At the end it is expected that a formula can be given that predicts the strength and stiffness based on measurements.

Strategy

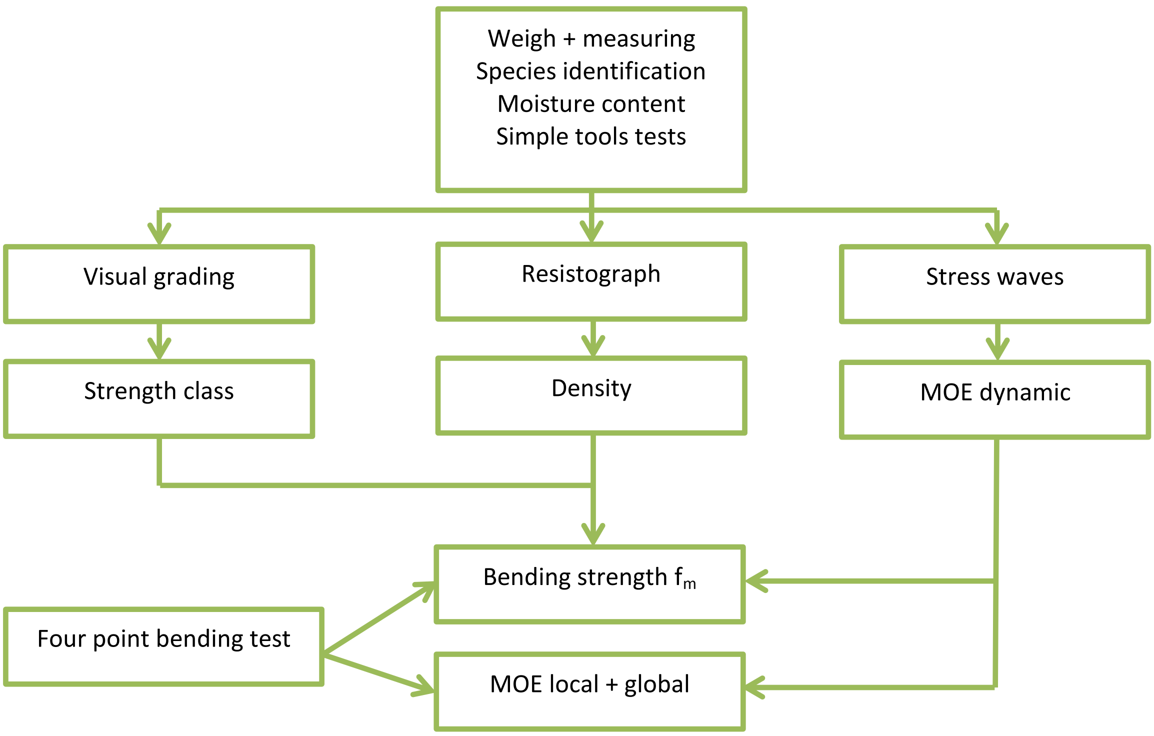


Figure G-10: Strategy to determine the reference properties

Step 1: Non-destructive testing on all members

- Species identification
  - ➔ Goal: Identify the specie
  - ➔ Needed equipment: Magnifying glass and handbook of wood anatomies.
  - ➔ Procedure: Macroscopic features are observed like color, size of growth rings, vessels, texture and rays.

- Density
  - ➔ Goal: Calculate the (average) density in kg/m³
  - ➔ Needed equipment: Measuring tape, scale, NEN-EN 408 and NEN-EN 348.
  - ➔ Procedure: According to NEN-EN 348 it is allowed to determine the mass and volume of the whole specimen and adjust the density to a small defect-free sample by dividing by 1,05. The average density is determined by dividing the total mass by its volume. According to NEN-EN 408, structural timber in strength tests a sample with a length of 25 mm needs to be extracted as close as possible to the fracture, free of knots and resin pockets.

- Moisture content
  - ➔ Goal: Measure the moisture content in %
  - ➔ Needed equipment: Moisture meter for measuring electric conduction (FMD) and dielectric constant (FMW), a scale, an oven, NEN-EN 408 and NEN-EN 13183-1.
  - ➔ Procedure: Members are placed inside climate room with temperature of (20± 2)°C and (65±5)% relative humidity. This is in compliance with NEN-EN 408. The moisture content is expected to be around 12%. Three places are measured: ¼ of the length from beam ends and in the middle of the member. The moisture is measured with two equipment's and from two sides (heartwood and sapwood). FMD and FMW are used for 2 cm depth while the FMD also measures the depth halfway. After destructive testing, samples with a length of 25 mm are taken close to the fracture to determine the actual moisture content. The sample mass is than determined before and during oven-drying (103±2)°C. All the water is removed when two successive weighings are the same. According to NEN-EN 13183-1 the following formula can be applied:

$$\omega = \frac{mass_{wet} - mass_{dry}}{mass_{dry}} x\ 100$$

- Visual inspection and grading
  - ➔ Goal: Assess strength class, surface decay and cracks
  - ➔ Needed equipment: NEN 5499, NEN-EN 1310, NEN-EN 336, NEN 3180, NEN 5466 and measuring tape
  - ➔ Procedure: Parameters to be measured are growth rings, deformations, knots, slope of grain (NEN 1310), cracks, wane (NEN-EN 336), discoloring, rot, bark, pressure wood, resin, insect damage, mechanical damage, overgrown defects and ageing. Tables in the NEN determine strength class.

- Awl/screwdriver, splinter test and sounding
  - ➔ Goal: Identify the surface condition and serious decay
  - ➔ Needed equipment: Screwdriver and hammer
  - ➔ Procedure: Member is struck with the hammer, the resulting sound indicates the condition. The screwdriver is struck under an angle into the wood to pry out a splinter. The sound indicates the condition.

- <u>Resistograph</u>
  - ➔ Goal: Measure the growth rates, make internal defects visible and find relation with the density.
  - ➔ Needed equipment: Resistograph

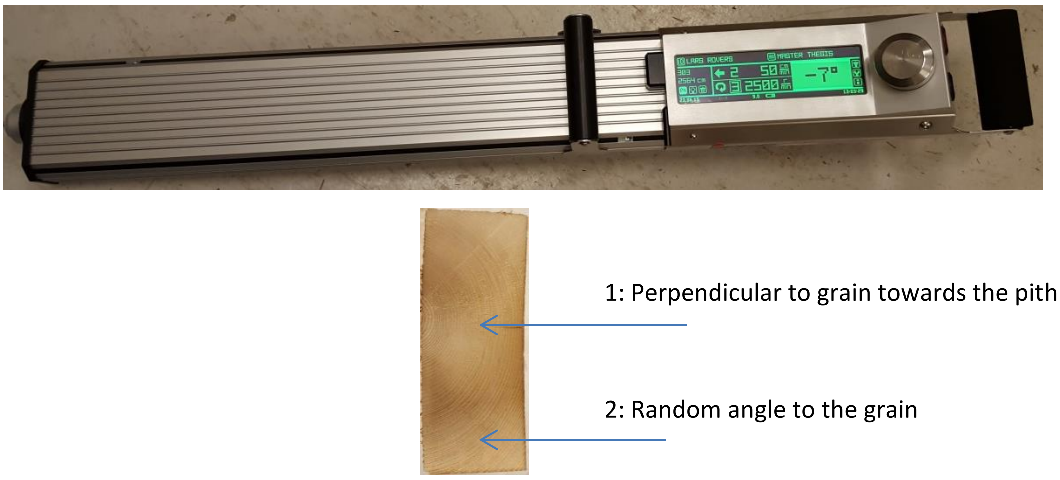


Figure G-11: Resistograph and the drilling direction

  - ➔ Procedure: A needle is driven at a constant feed and drill speed into the timber while the resistance is measured. In the first test the needle is driven perpendicular to the grain so that the ring thickness is measured. The second test drills to a random angle to the grain. Parameters to be measured are amplitude (%) vs depth (mm).

    The resistance measured is determined by:

$$RM = \frac{\sum Amplitude}{number\ of\ measured\ points}$$

    16 points are drilled along the longitudinal direction of the member. Hereafter samples are extracted to determine the associated density.

- <u>Dynamic stiffness measurement</u>
  - ➔ Goal: Measure free and restrained dynamic E-modulus
  - ➔ Needed equipment: Vibration meter and NEN EN-408

Figure G-12: MTG from Brookshuis Micro Electronics

The NEN-EN 408 notes that an alternative determination of the MOE is based on the dynamic MOE.

➔ Procedure: A hammer induces a wave while the meter measures the frequency. Parameter to be measured is longitudinal resonance frequency which is related to the dynamic stiffness modulus as follows:

Wave length:

$$\lambda = \frac{2 * L}{n} = \frac{C}{f}$$

The longitudinal motion of a rod has the wave speed:

$$C^2 = \frac{E_{dyn}}{\rho}$$

Dynamic elasticity modulus is then determined by:

$$E_{dyn} = C^2 * \rho = 4 * f^2 * l^2 * \rho$$

With:
n = Vibration mode, in this case the first mode is used
E_{dyn} = Dynamic modulus of elasticity [N/m²]
C = wave speed [m/s]
ρ = density [kg/m³]
f = frequency [Hz]
l = length of the member [m]

A formula can be derived to show the relationship of the dynamic E-modulus with the local and global E-modulus. For this the correlation is needed with the test results from static stiffness measurements. Free vibration is used for this correlation. To determine the influence of the surroundings various vibration tests will be performed to simulate in situ situation. In practice the vibration is damped by the wall, a decking and ceiling are attached and the beam ends are hidden.

<u>Alternative measurements</u>
The available vibration meter only measures the vibrations in the direction of the meter. Therefore in-situ only the transverse vibrations can be measured. Besides, the meter is hold by hand during measurements and thus the internal sensor is affected by human shakes. To get a better signal, the meter was adjusted so that an external sensor can be glued or mechanically connected on the side of the beam and can measure in three directions.

Figure G-13: Adjusted meter

Step 2: Destructive testing according to NEN-EN 408

According to the norms the moisture content should be

- Static stiffness measurement
  - ➔ Goal: Measure the static E-modulus global and local
  - ➔ Equipment needed: Pressure bench + measuring instruments according to NEN-EN 408 and NEN-EN 384

<u>Setup for determining the local modulus of elasticity:</u>

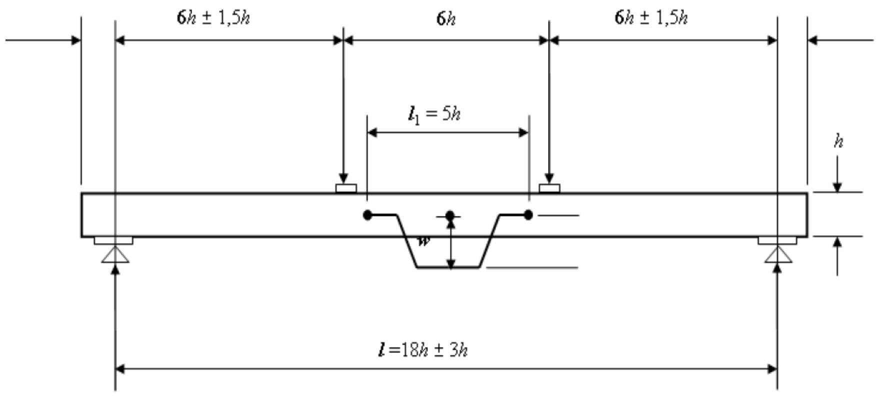


Figure G-14: Test setup for measuring E-local according to NEN-EN 408

The local E-modulus is determined by over a certain length in the neutral axis:

$$E_{m,l} = \frac{a * l_1^2 \ (F_2 - F_1)}{16 * I \ (w_2 - w_1)}$$

Where: a = Distance from support to outer loading point
l_1 = 5h
I = second moment of inertia
F_2 and w_2 = Total force and deflection at 0,4 * F_{max}
F_1 and w_1 = Total force and deflection at 0,1 * F_{max}

<u>Setup for determining the global modulus of elasticity:</u>

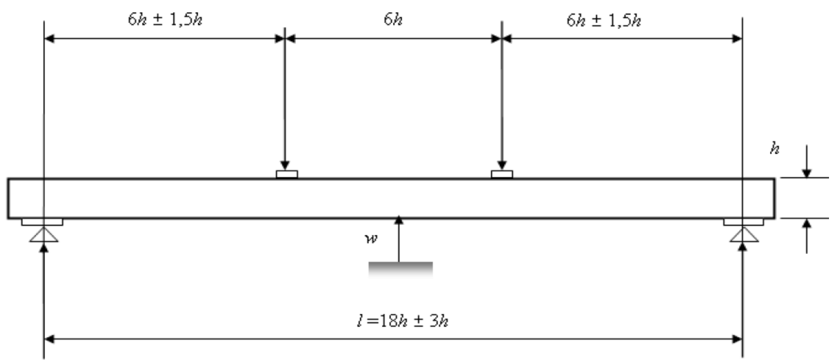


Figure G-15: Test setup for measuring E-global according to NEN-EN 408

The global E-modulus is determined at midspan by:

$$E_{m,g} = \frac{3 * a * l^2 - 4 * a^3}{2 * b * h^3 (2 * \frac{w_2 - w_1}{F_2 - F_1} - \frac{6 * a}{5 * G * b * h})}$$

Where: a = Distance from support to outer loading point
b and h = Width and height of member
F_2 and w_2 = Total force and deflection at 0,4 * F_{max}
F_1 and w_1 = Total force and deflection at 0,1 * F_{max}
G = The shear modulus is set to infinity because the procedure in NEN-EN 384:2010, 5.3.2 is followed which includes the shear influence.

- Four point bending test
  - ➔ Goal: Measure the ultimate failure load
  - ➔ Equipment needed: Pressure bench + measuring instruments according to NEN-EN 408 and NEN-EN 384

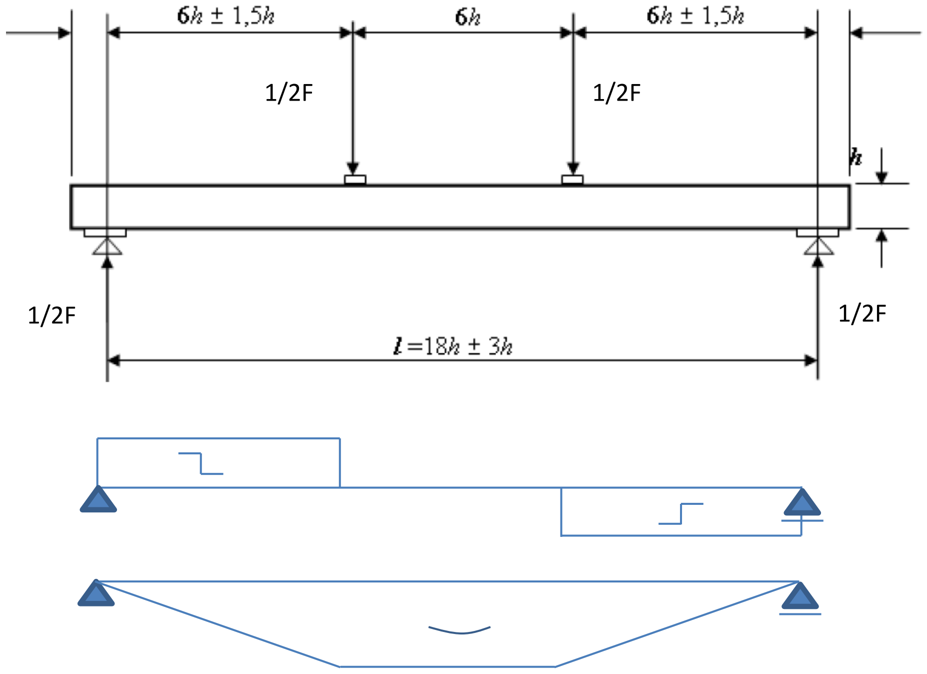


Figure G-16: Test setup for measuring the bending strength according to NEN-EN 408 along with the shear and moment distributions

The bending strength is determined with:

$$f_m = \frac{3 * F * a}{b * h^2}$$

Where: a = Distance from support to outer loading point
b and h = Width and height of member
F = Load at bending failure (total of both point loads)

G.5 THE EXPERIMENTS

G.5.1 VISUAL GRADING

NEN 5499 — **Member: S1**

Limitations of defects	Measured	Class assigned
individual knots on side	35 mm	T2 (38 mm)
Dimensions of individual knots on	70 mm	T1 (78 mm)
Splay knot	15 mm	T2
Knot cluster	104 mm	T2 (123 mm)
Knots in square sawn timber	NA	
Knot cluster in quare sawn timber	NA	
Slope of grain	1:50	T3 (1:10)
Growth rings average	3,7 mm	T3 (4 mm)
Ring shake	NA	T3
Not-transversing face shake	depth = max 25 mm; l = 1490 mm	T1 (1500 mm)
Transversing face shake	NA	T3
Top fracture	<1/4 b	T3 (1/4 b)
Curly grain	NA	T3

Limitations of geometric deviations	Measured	Class assigned
Wane	15 + 18 = 33 mm (11/65 b)	T3 (1/3 b)
Tolerance on length	NA	T3
Toleranceclass 1	Width = +5 mm; thickness = -2 mm	Should be rejected but it's not expected to cause problems
Toleranceclass 2	NA	
Bow	6 mm on 2 m	T3 (8 mm)
Spring	NA	T3
Twist	NA	T3
Cup	No demands	

Discoloration and fungi	Measured	Class assigned
Blue stain	Allowed	
Brown steak	Allowed	
Dote	NA	T3
Rot	NA	T3

Limitations in other defects	Measured	Class assigned
Bark	Not present	T3
Compression wood	<10%	T3 (10%)
Resin wood	Allowed	
Resin pocket	Allowed	
Insect damage	NA	T3
Mechanical damage	Past: none; Present: <5%	T2 (5%)
Overgrown defects	NA	T3
Ageing	NA	T3

NEN 3180:1970

Limitations of defects	Measured	Class assigned
Slope of grain	<1:10	CW (1:10)
Growth disturbance	NA	CW
Growth rings average	3,7 mm	CW (4 mm)
Resin pockets	NA	CW
Heart	Cleaved	CW
Bow	6 mm	CW
Spring	NA	CW
Knots side	35 mm	SB (57 mm)
Knot flat side	70 mm	Reject
Sum of knots	104 mm	SB (121 mm)
Square wood	NA	
Mechanical damage	Past: none; Present: 1/3 d	Past: CW; Present: Reject
Rot	NA	CW
Twist	NA	CW
Cracks Dry	<1/2 d	SB (1/2 d)
Ring shake	NA	CW
Inside crack	Some small cracks	CW
Discoloration	NA	CW
Volume weight	> 0,40	CW
Wane p	18 mm	SB (25 mm)
Wane p1+p2	33 mm	SB (38 mm)
Wane q	18 m	CW (39 mm)
wane q1+q2	33 mm	CW (65 mm)
Insect damage	NA	CW

NA = Not Applicable

NEN 3180:1958		**Member: S1**		NEN 5466:1983/1999		
Limitations of defects	Measured	Class assigned		Limitations of defects	Measured	Class assigned
Wane a/b	18 mm	CW (25 mm)		Borer hole	NA	B
Wane a+b	33 mm	CW (38 mm)		collaps	NA	B
Wane c/e	18 mm	CW (48 mm)		Slope of grain	1:50	B
Wane c+e	33 mm	CW (64 mm)		Compression fracture	NA	B
Knots side	35 mm	SB (56 mm)		Growth rings average	3,7 mm	B
Edge zone	7/39 h	SB (3/10 h)		Ressin pockets	NA	B
Mid zone	14/39 h	Reject		Heart	NA	B
Sum over length	104 mm	SB (121 mm)		Bark	NA	B
Spring	NA	CW		Hard/stuck knot	Present	B
Twist	NA	CW		Hard/loose knot	1	B
Growth rings average	3,7 mm	CW (4 mm)		soft knot	NA	B
Heart	Cleaved	CW		Knot portion	0,19	B
Slope of grain	<1:10	CW (1:10)		Knots, member width <190 mm	35 mm	C
Growth disturbance	NA	CW		Knots, member width >190 mm	70 mm	Reject
Cracks dry	<1/2 d	SB (1/2 d)		Reaction wood	<10%	B
Ring shake	NA	CW		Ring shake	NA	B
Inside crack	Some small cracks	CW		Hair shake	Allowed	B
Resin pocket	NA	CW		Length cracks max.	1 m	B
Mechanical damage	Past: none; Present: 1/3 d	Past: CW; Present: Reject		Sum of length cracks	1,89 m	B
Rot	NA	CW		Sum of width cracks	4 mm	C
Discoloration	NA	CW		Inside crack	Some small cracks	B
Insect holes	NA	CW		Split crack	NA	B
				End crack	b = 2 mm; l = 150 mm	C
				Fungi	NA	B
NA = Not Applicable				Sapwood hard	Allowed	B
				Discoloration	NA	B
				Bow	6 mm	B
				Spring	NA	B
				Twist	NA	B
				Cup	NA	B
				Wane	33 mm, 2 ribs	C
				Mechanical damage	Past: none; Present: 2 ribs	C

NEN 5499 **Member: S2**

Limitations of defects	Measured	Class assigned
Dimensions of individual knots on side	30 mm	T2 (38 mm)
Dimensions of individual knots on wide side	31 mm	T3 32 mm
Splay knot	5 mm	T2
Knot cluster	77 mm	T3 (83 mm)
Knots in square sawn timber	NA	
Knot cluster in quare sawn timber	NA	
Slope of grain	5:100	T3 (1:10)
Growth rings average	3,3 mm	T3 (4 mm)
Ring shake	NA	
Not-transversing face shake	Depth: 20 mm; length: 1500 mm; Near end a crack of 0,8 b is present;	Should be rejected but near end crack is not long. T1 (1500 mm)
Transversing face shake	NA	T3
Top fracture	<1/4 b	T3 (1/4 b)
Curly grain	NA	T3

Limitations of geometric deviations	Measured	Class assigned
Wane	NA	T3
Tolerance on length	NA	T3
Toleranceclass 1	Width = +4 mm; thickness = -3 mm	Should be rejected but it's not expected to cause problems
Toleranceclass 2	NA	
Bow	NA	T3
Spring	3 mm	T3
Twist	NA	T3
Cup		No demands

Discoloration and fungi	Measured	Class assigned
Blue stain	Allowed	
Brown steak	Allowed	
Dote	NA	T3
Rot	NA	T3

Limitations in other defects	Measured	Class assigned
Bark	Not present	T3
Compression wood	<10%	T3 (10%)
Resin wood	Allowed	
Resin pocket	Allowed	
Insect damage	NA	T3
Mechanical damage	Past: none; Present: 3% at end and <5% in middle	Past: T3; Present: T2
Overgrown defects	NA	T3
Ageing	grayer	T3

NEN 3180:1970

Limitations of defects	Measured	Class assigned
Slope of grain	<1:10	CW (1:10)
Growth disturbance	NA	CW
Growth rings average	3,3 mm	CW (4 mm)
Resin pockets	NA	CW
Heart	Cleaved	CW
Bow	NA	CW
Spring	3 mm	CW (5 mm)
Knots side	30 mm	CW (30 mm)
Knot flat side	31 mm	CW (39 mm)
Sum of knots	77 mm	SB (120 mm)
Square wood	NA	
Mechanical damage	Past: none; Present: 2/5 d	Past: CW; Present: Reject
Rot	NA	CW
Twist	NA	CW
Cracks Dry	<1/3 d	CW (at beam end reject)
Ring shake	NA	CW
Inside crack	Some small cracks	CW
Discoloration	Grayer	CW
Volume weight	> 0,40	CW
Wane p	NA	CW
Wane p1+p2	NA	CW
Wane q	NA	CW
wane q1+q2	NA	CW
Insect damage	NA	CW

NA = Not Applicable

NEN 3180:1958 **Member: S2**

Limitations of defects	Measured	Class assigned
Wane a/b	NA	CW
Wane a+b	NA	CW
Wane c/e	NA	CW
Wane c+e	NA	CW
Knots side	30 mm	CW (30mm)
Edge zone	30 mm	CW (32 mm)
Mid zone	31 mm	CW (39 mm)
Sum over length	77 mm	SB (120 mm)
Spring	3 mm	CW (5 mm)
Twist	NA	CW
Growth rings average	3,3 mm	CW (4 mm)
Heart	Cleaved	CW
Slope of grain	<1:10	CW (1:10)
Growth disturbance	NA	CW
Cracks dry	<1/3 d	CW (at beam end reject)
Ring shake	NA	CW
Inside crack	Some small cracks	CW
Resin pocket	NA	CW
Mechanical damage	Past: none; Present: 1/3 d	Past: CW; Present: Reject
Rot	NA	CW
Discoloration	Grayer	CW
Insect holes	NA	CW

NA = Not Applicable

NEN 5466:1983/1999

Limitations of defects	Measured	Class assigned
Borer hole	NA	B
collaps	NA	B
Slope of grain	5:100	B (1:10)
Compression fracture	NA	B
Growth rings average	3,3 mm	B (5 mm)
Ressin pockets	NA	B
Heart	Cleaved	B
Bark	NA	B
Hard/stuck knot	Present	B
Hard/loose knot	1	B (1)
soft knot	NA	B
Knot portion	0,14	B (0,20)
Knots, member width <190 mm	30 mm	B (30 mm)
Knots, member width >190 mm	31 mm	B (40 mm)
Reaction wood	<10%	B (10%)
Ring shake	NA	B
Hair shake	Allowed	B
Length cracks max.	200 mm	B (1191 mm)
Sum of length cracks	1500 mm	B (2382 mm)
Sum of width cracks	4 mm	C (Past might be B)
Inside crack	Small cracks	B
Split crack	NA	B
End crack	b = 1 mm; l = 110 mm	C (l=200 mm)
Fungi	NA	B
Sapwood hard	Allowed	B
Discoloration	Grayer	B
Bow	NA	B
Spring	3 mm	B (4 mm)
Twist	NA	B
Cup	2 mm	B (2 mm)
Wane	NA	B
Mechanical damage	Past: none; Present: 2 ribs	Past: B; Present: Reject

NEN 5499		**Member: S3**

Limitations of defects	Measured	Class assigned
Dimensions of individual knots on side	29 mm	T2 (38 mm)
Dimensions of individual knots on wide side	24 mm	T3 (32 mm)
Splay knot	30 mm	T2
Knot cluster	65 mm	T3 (82,5 mm)
Knots in square sawn timber	NA	
Knot cluster in quare sawn timber	NA	
Slope of grain	1:50	T3 (1:10)
Growth rings average	3,18 mm	T3 (4 mm)
Ring shake	NA	
Not-transversing face shake	600 mm	T3 (1000 mm)
Transversing face shake	NA	T3
Top fracture	< 1/4 b	T3 (1/4 b)
Curly grain	NA	T3

Limitations of geometric deviations	Measured	Class assigned
Wane	NA	T3
Tolerance on length	NA	T3
Toleranceclass 1	Width = +2 mm; thickness = -1 mm	Should be rejected but it's not expected to cause problems
Toleranceclass 2	NA	
Bow	1 mm on 2 m	T3 (8 mm)
Spring	2 mm on 2 m	T3 (5 mm)
Twist	NA	T3
Cup	No demands	

Discoloration and fungi	Measured	Class assigned
Blue stain	Allowed	
Brown steak	Allowed	
Dote	NA	T3
Rot	NA	T3

Limitations in other defects	Measured	Class assigned
Bark	Not present	T3
Compression wood	<10%	T3 (10%)
Resin wood	Allowed	
Resin pocket	Allowed	
Insect damage	NA	T3
Mechanical damage	Past: none; Present: <5%	Past: T3; Present: T2
Overgrown defects	NA	T3
Ageing	First meter is grayer	T3

NEN 3180:1970

	Measured	Class assigned
Slope of grain	<1:10	CW (1:10)
Growth disturbance	NA	CW
Growth rings average	3,18 mm	CW (4 mm)
Resin pockets	NA	
Heart	Cleaved	CW
Bow	1 mm	CW (19 mm)
Spring	2 mm	CW (5 mm)
Knots side	29 mm	CW (30 mm)
Knot flat side	24 mm	CW (39 mm)
Sum of knots	65 mm	CW (69 mm)
Square wood	NA	
Mechanical damage	Past: none; Present: 0,4 d	Past: CW; Present: Reject
Rot	NA	CW
Twist	NA	CW
Cracks Dry	22 mm	CW (25 mm)
Ring shake	NA	
Inside crack	Some small cracks	CW
Discoloration	Grayer	CW
Volume weight	>0,40	CW
Wane p	NA	CW
Wane p1+p2	NA	CW
Wane q	NA	CW
wane q1+q2	NA	CW
Insect damage	NA	CW

NA = Not Applicable

NEN 5499		**Member: S4**

Limitations of defects	Measured	Class assigned
Dimensions of individual knots on side	32 mm [ON TOP]	T2 (37 mm)
Dimensions of individual knots on wide side	39 mm	T2 (49 mm)
Splay knot	40 mm	T2
Knot cluster	80 mm	T3 (82,5 mm)
Knots in square sawn timber	NA	
Knot cluster in quare sawn timber	NA	
Slope of grain	1:50	T3 (1:10)
Growth rings average	3,57 mm	T3 (4 mm)
Ring shake	NA	
Not-transversing face shake	1310 mm	T1 (1500 mm)
Transversing face shake	NA	T3
Top fracture	< 1/4 b	T3 (1/4 b)
Curly grain	NA	T3

Limitations of geometric deviations	Measured	Class assigned
Wane	10 mm	T3 (24 mm)
Tolerance on length	NA	T3
Toleranceclass 1	Width = +5 mm; thickness = -2 mm	T3
Toleranceclass 2	NA	
Bow	1 mm on 2 m	T3 (8 mm)
Spring	2 mm on 2 m	T3 (5 mm)
Twist	NA	T3
Cup	No demands	

Discoloration and fungi	Measured	Class assigned
Blue stain	Allowed	
Brown steak	Allowed	
Dote	NA	T3
Rot	Fungi in knot	T3

Limitations in other defects	Measured	Class assigned
Bark	Not present	T3
Compression wood	<10%	T3 (10%)
Resin wood	Allowed	
Resin pocket	Allowed	
Insect damage	NA	T3
Mechanical damage	Past: none; Present: <5%	Past: T3; Present: T2
Overgrown defects	NA	T3
Ageing	First meter is grayer	T3

NEN 3180:1970

	Measured	Class assigned
Slope of grain	<1:10	CW (1:10)
Growth disturbance	NA	CW
Growth rings average	3,57 mm	CW (4 mm)
Resin pockets	NA	
Heart	Cleaved	CW
Bow	1 mm	CW (19 mm)
Spring	2 mm	CW (5 mm)
Knots side	32 mm	SB (56 mm)
Knot flat side	39 mm	CW (39 mm)
Sum of knots	80 mm	CW (120 mm)
Square wood	NA	
Mechanical damage	Past: none; Present: 0,4 d	Past: CW; Present: SB
Rot	Not allowed	Reject
Twist	NA	CW
Cracks Dry	23 mm	CW (25 mm)
Ring shake	NA	
Inside crack	Some small cracks	CW
Discoloration	NA	CW
Volume weight	>0,40	CW
Wane p	10 mm	CW (15 mm)
Wane p1+p2	10 mm	CW (25 mm)
Wane q	10 mm	CW (39 mm)
wane q1+q2	10 mm	CW (65 mm)
Insect damage	NA	CW

NA = Not Applicable

NEN 5499		**Member: S5**
Limitations of defects	Measured	Class assigned
Dimensions of individual knots on side	21 mm [on top]	T3 (25 mm)
Dimensions of individual knots on wide side	24 mm	T3 (32 mm)
Splay knot	Not present	T3
Knot cluster	49 mm	T3 (82,7 mm)
Knots in square sawn timber	NA	
Knot cluster in quare sawn timber	NA	
Slope of grain	1:50	T3 (1:10)
Growth rings average	2,8 mm	T3 (4 mm)
Ring shake	NA	
Not-transversing face shake	300 mm	T3 (1000 mm)
Transversing face shake	NA	T3
Top fracture	< 1/4 b	T3 (1/4 b)
Curly grain	NA	T3

Limitations of geometric deviations	Measured	Class assigned
Wane	Top: 20 mm; Side: 15 mm	T3 (25 mm and 64 mm)
Tolerance on length	NA	T3
Toleranceclass 1	Width = +5 mm; thickness = -3 mm	Should be rejected but it's not expected to cause problems
Toleranceclass 2	NA	
Bow	2 mm on 2 m	T3 (8 mm)
Spring	1 mm on 2 m	T3 (5 mm)
Twist	NA	T3
Cup	No demands	

Discoloration and fungi	Measured	Class assigned
Blue stain	Allowed	
Brown steak	Allowed	
Dote	NA	T3
Rot	NA	T3

Limitations in other defects	Measured	Class assigned
Bark	Not present	T3
Compression wood	<10%	T3 (10%)
Resin wood	Allowed	
Resin pocket	Allowed	
Insect damage	NA	T3
Mechanical damage	Past: none; Present: <5%	Past: T3; Present: T2
Overgrown defects	NA	T3
Ageing	NA	T3

NEN 3180:1970

	Measured	Class assigned
Slope of grain	<1:10	CW (1:10)
Growth disturbance	NA	CW
Growth rings average	2,8 mm	CW (4 mm)
Resin pockets	NA	
Heart	Cleaved	CW
Bow	2 mm	CW (19 mm)
Spring	1 mm	CW (5 mm)
Knots side	21 mm	CW (30 mm)
Knot flat side	24 mm	CW (38 mm)
Sum of knots	49 mm	CW (69 mm)
Square wood	NA	
Mechanical damage	Past: none; Present: 0,26 d	Past: CW; Present: Reject
Rot	NA	CW
Twist	NA	CW
Cracks Dry	17 mm	CW (25 mm)
Ring shake	NA	
Inside crack	Some small cracks	CW
Discoloration	NA	CW
Volume weight	>0,40	CW (0,40)
Wane p	20 mm	SB (25 mm)
Wane p1+p2	20 mm	CW (25 mm)
Wane q	15 mm	CW (38 mm)
wane q1+q2	15 mm	CW (64 mm)
Insect damage	NA	CW

NA = Not Applicable

NEN 5499		**Member: S6**

Limitations of defects	Measured	Class assigned
Dimensions of individual knots on side	36 mm	T2 (38 mm)
Dimensions of individual knots on wide side	30 mm	T3 (32 mm)
Splay knot	25 mm	T2
Knot cluster	129 mm	T1 (138 mm)
Knots in square sawn timber	NA	
Knot cluster in quare sawn timber	NA	
Slope of grain	1:50	T3 (1:10)
Growth rings average	3,28 mm	T3 (4 mm)
Ring shake	NA	
Not-transversing face shake	1450 mm	T1 (1500 mm)
Transversing face shake	NA	T3
Top fracture	< 1/4 b	T3 (1/4 b)
Curly grain	NA	T3

Limitations of geometric deviations	Measured	Class assigned
Wane	NA	T3
Tolerance on length	NA	T3
Toleranceclass 1	Width = +5 mm; thickness = -3 mm	Should be rejected but it's not expected to cause problems
Toleranceclass 2	NA	
Bow	2 mm on 2 m	T3 (8 mm)
Spring	2 mm on 2 m	T3 (5 mm)
Twist	NA	T3
Cup	No demands	

Discoloration and fungi	Measured	Class assigned
Blue stain	Allowed	
Brown steak	Allowed	
Dote	NA	T3
Rot	NA	T3

Limitations in other defects	Measured	Class assigned
Bark	Not present	T3
Compression wood	<10%	T3 (10%)
Resin wood	Allowed	
Resin pocket	Allowed	
Insect damage	NA	T3
Mechanical damage	Past: none; Present: <5%	Past: T3; Present: T2
Overgrown defects	NA	T3
Ageing	Graying	T3

NEN 3180:1970

	Measured	Class assigned
Slope of grain	<1:10	CW (1:10)
Growth disturbance	NA	CW
Growth rings average	3,27 mm	CW (4 mm)
Resin pockets	NA	
Heart	Cleaved	CW
Bow	2 mm	CW (19 mm)
Spring	2 mm	CW (5 mm)
Knots side	36 mm	SB (57 mm)
Knot flat side	30 mm	CW (38 mm)
Sum of knots	129 mm	Reject
Square wood	NA	
Mechanical damage	Past: none; Present: 0,39 d	Past: CW; Present: Reject
Rot	NA	CW
Twist	NA	CW
Cracks Dry	17 mm	CW (25 mm)
Ring shake	NA	
Inside crack	Some small cracks	CW
Discoloration	Graying	CW
Volume weight	>0,40	CW (0,40)
Wane p	NA	CW
Wane p1+p2	NA	CW
Wane q	NA	CW
wane q1+q2	NA	CW
Insect damage	NA	CW

NA = Not Applicable

NEN 5499		Member: S7
Limitations of defects	Measured	Class assigned
Dimensions of individual knots on side	30 mm	T2 (38 mm)
Dimensions of individual knots on wide side	35 mm	T2 (49 mm)
Splay knot	35 mm	T2
Knot cluster	65 mm	T3 (83 mm)
Knots in square sawn timber	NA	
Knot cluster in quare sawn timber	NA	
Slope of grain	3:100	T3 (1:10)
Growth rings average	3,39 mm	T3 (4 mm)
Ring shake	Small	T2
Not-transversing face shake	1120 mm	T1 (1500 mm)
Transversing face shake	NA	T3
Top fracture	< 1/4 b	T3 (1/4 b)
Curly grain	Some	T2

Limitations of geometric deviations	Measured	Class assigned
Wane	NA	T3
Tolerance on length	NA	T3
Toleranceclass 1	Width = +5 mm; thickness = -1 mm	Should be rejected but it's not expected to cause problems
Toleranceclass 2	NA	
Bow	2 mm on 2 m	T3 (8 mm)
Spring	1 mm on 2 m	T3 (5 mm)
Twist	0,4 mm on 25 mm	T3
Cup	No demands	

Discoloration and fungi	Measured	Class assigned
Blue stain	Allowed	
Brown steak	Allowed	
Dote	NA	T3
Rot	NA	T3

Limitations in other defects	Measured	Class assigned
Bark	Not present	T3
Compression wood	<10%	T3 (10%)
Resin wood	Allowed	
Resin pocket	Allowed	
Insect damage	NA	T3
Mechanical damage	Past: none; Present: <5%	Past: T3; Present: T2
Overgrown defects	NA	T3
Ageing	NA	T3

NEN 3180:1970		
Limitations of defects	Measured	Class assigned
Slope of grain	<1:10	CW (1:10)
Growth disturbance	NA	CW
Growth rings average	3,39 mm	CW (4 mm)
Resin pockets	NA	
Heart	Heart present	Reject
Bow	2 mm	CW (19 mm)
Spring	1 mm	CW (5 mm)
Knots side	30 mm	CW (31 mm)
Knot flat side	35 mm	CW (38 mm)
Sum of knots	65 mm	CW (69 mm)
Square wood	NA	
Mechanical damage	Past: none; Present: 0,39 d	Past: CW; Present: Reject
Rot	NA	CW
Twist	8 mm	CW (13 mm)
Cracks Dry	25+20=45 mm	Reject
Ring shake	Some	CW
Inside crack	Some small cracks	CW
Discoloration	NA	CW
Volume weight	>0,40	CW (0,40)
Wane p	NA	CW
Wane p1+p2	NA	CW
Wane q	NA	CW
wane q1+q2	NA	CW
Insect damage	NA	CW

NA = Not Applicable

NEN 5499 **Member: S8**

Limitations of defects	Measured	Class assigned
Dimensions of individual knots on side	28 mm	T2 (39 mm)
Dimensions of individual knots on wide side	40 mm	T2 (49 mm)
Splay knot	35 mm	T2
Knot cluster	108 mm	T2 (123 mm)
Knots in square sawn timber	NA	
Knot cluster in quare sawn timber	NA	
Slope of grain	1:50	T3 (1:10)
Growth rings average	3,39 mm	T3 (4 mm)
Ring shake	NA	T3
Not-transversing face shake	500 mm and crack on top	T1 (1000 mm)
Transversing face shake	NA	T3
Top fracture	< 1/4 b	T3 (1/4 b)
Curly grain	NA	T3

Limitations of geometric deviations	Measured	Class assigned
Wane	NA	T3
Tolerance on length	NA	T3
Toleranceclass 1	Width = +6 mm; thickness = 0 mm	Should be rejected but it's not expected to cause problems
Toleranceclass 2	NA	
Bow	1 mm on 2 m	T3 (8 mm)
Spring	1 mm on 2 m	T3 (5 mm)
Twist	NA	T3
Cup	No demands	

Discoloration and fungi	Measured	Class assigned
Blue stain	Allowed	
Brown steak	Allowed	
Dote	NA	T3
Rot	NA	T3

Limitations in other defects	Measured	Class assigned
Bark	Not present	T3
Compression wood	<10%	T3 (10%)
Resin wood	Allowed	
Resin pocket	Allowed	
Insect damage	NA	T3
Mechanical damage	Past: none; Present: <5%	Past: T3; Present: T2
Overgrown defects	NA	T3
Ageing	NA	T3

NEN 3180:1970

Limitations of defects	Measured	Class assigned
Slope of grain	<1:10	CW (1:10)
Growth disturbance	NA	CW
Growth rings average	3,39 mm	CW (4 mm)
Resin pockets	NA	
Heart	Heart present	Reject
Bow	1 mm	CW (19 mm)
Spring	1 mm	CW (5 mm)
Knots side	28 mm	CW (31 mm)
Knot flat side	40 mm	SB (65 mm)
Sum of knots	108 mm	SB (123 mm)
Square wood	NA	
Mechanical damage	Past: none; Present: 0,25 d	Past: CW; Present: Reject
Rot	NA	CW
Twist	NA	CW
Cracks Dry	40 mm on edge	Reject
Ring shake	NA	CW
Inside crack	Some small cracks	CW
Discoloration	NA	CW
Volume weight	>0,40	CW (0,40)
Wane p	NA	CW
Wane p1+p2	NA	CW
Wane q	NA	CW
wane q1+q2	NA	CW
Insect damage	NA	CW

NA = Not Applicable

NEN 5499		Member: S9
Limitations of defects	Measured	Class assigned
Dimensions of individual knots on side	50 mm [on top]	T1 (62 mm)
Dimensions of individual knots on wide side	43 mm	T2 (49 mm)
Splay knot	NA	T3
Knot cluster	124 mm	T2 (125 mm)
Knots in square sawn timber	NA	
Knot cluster in quare sawn timber	NA	
Slope of grain	1:50	T3 (1:10)
Growth rings average	3,10 mm	T3 (4 mm)
Ring shake	NA	T3
Not-transversing face shake	Over total length	T0
Transversing face shake	NA	T3
Top fracture	< 1/4 b	T3 (1/4 b)
Curly grain	NA	T3

Limitations of geometric deviations	Measured	Class assigned
Wane	8 mm	T3 (26 mm)
Tolerance on length	NA	T3
Toleranceclass 1	Width = +6 mm; thickness = 0 mm	Should be rejected but it's not expected to cause problems
Toleranceclass 2	NA	
Bow	1 mm on 2 m	T3 (8 mm)
Spring	1 mm on 2 m	T3 (5 mm)
Twist	NA	T3
Cup	No demands	

Discoloration and fungi	Measured	Class assigned
Blue stain	Allowed	
Brown steak	Allowed	
Dote	NA	T3
Rot	NA	T3

Limitations in other defects	Measured	Class assigned
Bark	Not present	T3
Compression wood	<10%	T3 (10%)
Resin wood	Allowed	
Resin pocket	Allowed	
Insect damage	NA	T3
Mechanical damage	Past: none; Present: <5%	Past: T3; Present: T2
Overgrown defects	NA	T3
Ageing	NA	T3

NEN 3180:1970		
Limitations of defects	Measured	Class assigned
Slope of grain	<1:10	CW (1:10)
Growth disturbance	NA	CW
Growth rings average	3,10 mm	CW (4 mm)
Resin pockets	NA	
Heart	Heart present	Reject
Bow	1 mm	CW (19 mm)
Spring	1 mm	CW (5 mm)
Knots side	50 mm	SB (58 mm)
Knot flat side	43 mm	SB (65 mm)
Sum of knots	124 mm	Reject
Square wood	NA	
Mechanical damage	Past: none; Present: 0,16 d	Past: CW; Present: SB
Rot	NA	CW
Twist	NA	CW
Cracks Dry	Average of 15 mm	CW
Ring shake	NA	CW
Inside crack	Some small cracks	CW
Discoloration	NA	CW
Volume weight	>0,40	CW (0,40)
Wane p	8 mm	CW
Wane p1+p2	8 mm	CW
Wane q	8 mm	CW
wane q1+q2	8 mm	CW
Insect damage	NA	CW

NA = Not Applicable

NEN 5499		**Member: S10**
Limitations of defects	Measured	Class assigned
Dimensions of individual knots on side	32 mm	T2 (38 mm)
Dimensions of individual knots on wide side	40 mm	T2 (49 mm)
Splay knot	NA	T3
Knot cluster	71 mm	T3 (83 mm)
Knots in square sawn timber	NA	
Knot cluster in quare sawn timber	NA	
Slope of grain	3:100	T3 (1:10)
Growth rings average	5,60 mm	T2 (6 mm)
Ring shake	NA	T3
Not-transversing face shake	Over total length	T0
Transversing face shake	NA	T3
Top fracture	< 1/4 b	T3 (1/4 b)
Curly grain	NA	T3

Limitations of geometric deviations	Measured	Class assigned
Wane	NA	T3
Tolerance on length	NA	T3
Toleranceclass 1	Width = +5 mm; thickness = +1 mm	Should be rejected but it's not expected to cause problems
Toleranceclass 2	NA	
Bow	1 mm on 2 m	T3 (8 mm)
Spring	1 mm on 2 m	T3 (5 mm)
Twist	1 mm per 2 m	T3
Cup	No demands	

Discoloration and fungi	Measured	Class assigned
Blue stain	Allowed	
Brown steak	Allowed	
Dote	NA	T3
Rot	NA	T3

Limitations in other defects	Measured	Class assigned
Bark	Not present	T3
Compression wood	<10%	T3 (10%)
Resin wood	Allowed	
Resin pocket	Allowed	
Insect damage	NA	T3
Mechanical damage	Past: none; Present: <5%	Past: T3; Present: T2
Overgrown defects	NA	T3
Ageing	NA	T3

NEN 3180:1970		
Limitations of defects	Measured	Class assigned
Slope of grain	<1:10	CW (1:10)
Growth disturbance	NA	CW
Growth rings average	5,6 mm	SB
Resin pockets	NA	
Heart	Heart present	Reject
Bow	1 mm	CW (19 mm)
Spring	1 mm	CW (5 mm)
Knots side	32 mm	SB (57 mm)
Knot flat side	40 mm	SB (65 mm)
Sum of knots	71 mm	SB (122 mm)
Square wood	NA	
Mechanical damage	Past: none; Present: 0,13 d	Past: CW; Present: SB
Rot	NA	CW
Twist	NA	CW
Cracks Dry	Average of 22 mm	CW
Ring shake	NA	CW
Inside crack	Some small cracks	CW
Discoloration	NA	CW
Volume weight	>0,40	CW (0,40)
Wane p	NA	CW
Wane p1+p2	NA	CW
Wane q	NA	CW
wane q1+q2	NA	CW
Insect damage	NA	CW

NA = Not Applicable

NEN 5499	Member: L1	
Limitations of defects	Measured	Class assigned
Dimensions of individual knots on side	32 mm	T3 (33 mm)
Dimensions of individual knots on wide side	35 mm	T3 (41 mm)
Splay knot	20 mm	T3
Knot cluster	48 mm	T3 (106 mm)
Knots in square sawn timber	NA	
Knot cluster in quare sawn timber	NA	
Slope of grain	1:50	T3 (1:10)
Growth rings average	<4 mm	T3 (4 mm)
Ring shake	NA	T3
Not-transversing face shake	Over total length	T0
Transversing face shake	NA	T3
Top fracture	< 1/4 b	T3 (1/4 b)
Curly grain	NA	T3

Limitations of geometric deviations	Measured	Class assigned
Wane	NA	T3
Tolerance on length	NA	T3
Toleranceclass 1	Unknown	
Toleranceclass 2	NA	
Bow	1 mm on 2 m	T3 (8 mm)
Spring	1 mm on 2 m	T3 (5 mm)
Twist	NA	T3
Cup	No demands	

Discoloration and fungi	Measured	Class assigned
Blue stain	Allowed	
Brown steak	Allowed	
Dote	NA	T3
Rot	NA	T3

Limitations in other defects	Measured	Class assigned
Bark	Not present	T3
Compression wood	<10%	T3 (10%)
Resin wood	Allowed	
Resin pocket	Allowed	
Insect damage	NA	T3
Mechanical damage	None	T3
Overgrown defects	NA	T3
Ageing	Gray	T3

NEN 5499	Member: L2	
Limitations of defects	Measured	Class assigned
Dimensions of individual knots on side	30 mm	T2 (40 mm)
Dimensions of individual knots on wide side	25 mm	T3 (40 mm)
Splay knot	Na	T3
Knot cluster	27 mm	T3
Knots in square sawn timber	NA	
Knot cluster in quare sawn timber	NA	
Slope of grain	1:50	T3 (1:10)
Growth rings average	<4 mm	T3 (4 mm)
Ring shake	NA	T3
Not-transversing face shake	Over total length	T0
Transversing face shake	NA	T3
Top fracture	< 1/4 b	T3 (1/4 b)
Curly grain	NA	T3

Limitations of geometric deviations	Measured	Class assigned
Wane	NA	T3
Tolerance on length	NA	T3
Toleranceclass 1	Unknown	
Toleranceclass 2	NA	
Bow	1 mm on 2 m	T3 (8 mm)
Spring	1 mm on 2 m	T3 (5 mm)
Twist	NA	T3
Cup	No demands	

Discoloration and fungi	Measured	Class assigned
Blue stain	Allowed	
Brown steak	Allowed	
Dote	NA	T3
Rot	NA	T3

Limitations in other defects	Measured	Class assigned
Bark	Not present	T3
Compression wood	<10%	T3 (10%)
Resin wood	Allowed	
Resin pocket	Allowed	
Insect damage	NA	T3
Mechanical damage	Past: none; Present: 11%	Past: T3; Present: T0
Overgrown defects	NA	T3
Ageing	Gray	T3

NEN 5499	Member: L3	
Limitations of defects	Measured	Class assigned
Dimensions of individual knots on side	NA	T3
Dimensions of individual knots on wide side	NA	T3
Splay knot	NA	T3
Knot cluster	NA	T3
Knots in square sawn timber	NA	
Knot cluster in quare sawn timber	NA	
Slope of grain	1:10	T3 (1:10)
Growth rings average	<4 mm	T3 (4 mm)
Ring shake	NA	T3
Not-transversing face shake	Over total length	T0
Transversing face shake	NA	T3
Top fracture	< 1/4 b	T3 (1/4 b)
Curly grain	NA	T3

Limitations of geometric deviations	Measured	Class assigned
Wane	NA	T3
Tolerance on length	NA	T3
Toleranceclass 1	Unknown	
Toleranceclass 2	NA	
Bow	2 mm on 2 m	T3 (8 mm)
Spring	3 mm on 2 m	T3 (5 mm)
Twist	NA	T3
Cup	No demands	

Discoloration and fungi	Measured	Class assigned
Blue stain	Allowed	
Brown steak	Allowed	
Dote	NA	T3
Rot	NA	T3

Limitations in other defects	Measured	Class assigned
Bark	Not present	T3
Compression wood	<10%	T3 (10%)
Resin wood	Allowed	
Resin pocket	Allowed	
Insect damage	NA	T3
Mechanical damage	Past: none; Present: 4%	Past: T3; Present: T2
Overgrown defects	NA	T3
Ageing	Gray	T3

G.5.2 DYNAMIC STIFNESS MEASUREMENTS

The meter measures the signal in the time domain after which it is transformed by a fourier transformation to the frequency domain. The quality of the signal in the frequency domain determines if the measure is reliable enough. A clear signal is gained when the meter is placed on the edge however when it is placed on the top or the side some extra interpretation is needed. For approval of the signal three criteria must be fulfilled:

A) The signal must show peaks that are associated with the eigenfrequencies. Figure G-16 shows a clear signal with the fundamental tone and the overtones.

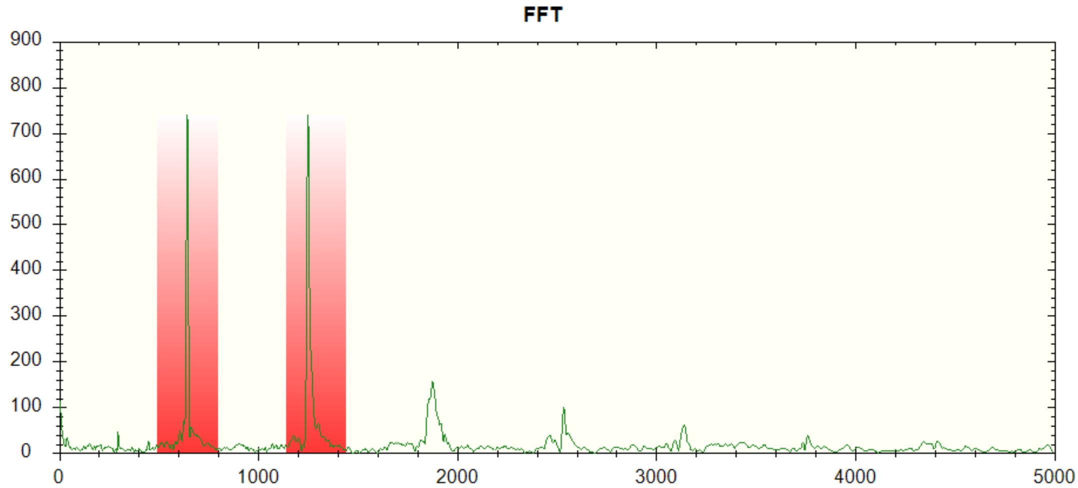


Figure G-17: Example of clear signal showing the fundamental tone and overtones

B) The distances between the peaks are the same because the overtones are a multiple of the first eigenfrequency. This is clearly visible in figure G-16.

C) The expected modulus of elasticity allows for predictions of the frequency which should be close to the measured value. Figure G-17 shows a graph that can be used for this prediction. The lower bound is defined by 0.9 x 10000 N/mm² and the upper bound by 1.1 x 12000 N/mm². This is based on the findings of (Govers, 1966) as described in chapter E.5 plus a margin of 10%. The margin is based on results of the tests. Proposed is:

$$f = \sqrt{\frac{E_{stat} * C1 * C2}{4 * l^2 * \rho * 10^{-12}}}$$

f = Expected frequency (Hz)
E_{stat} = Expected MOE (N/mm²)
C1 = Adjustment factor for E_{dyn}
C2 = Adjustment factor to take into account the surroundings
l = length of the member (mm)
ρ = Expected density (kg/m³)

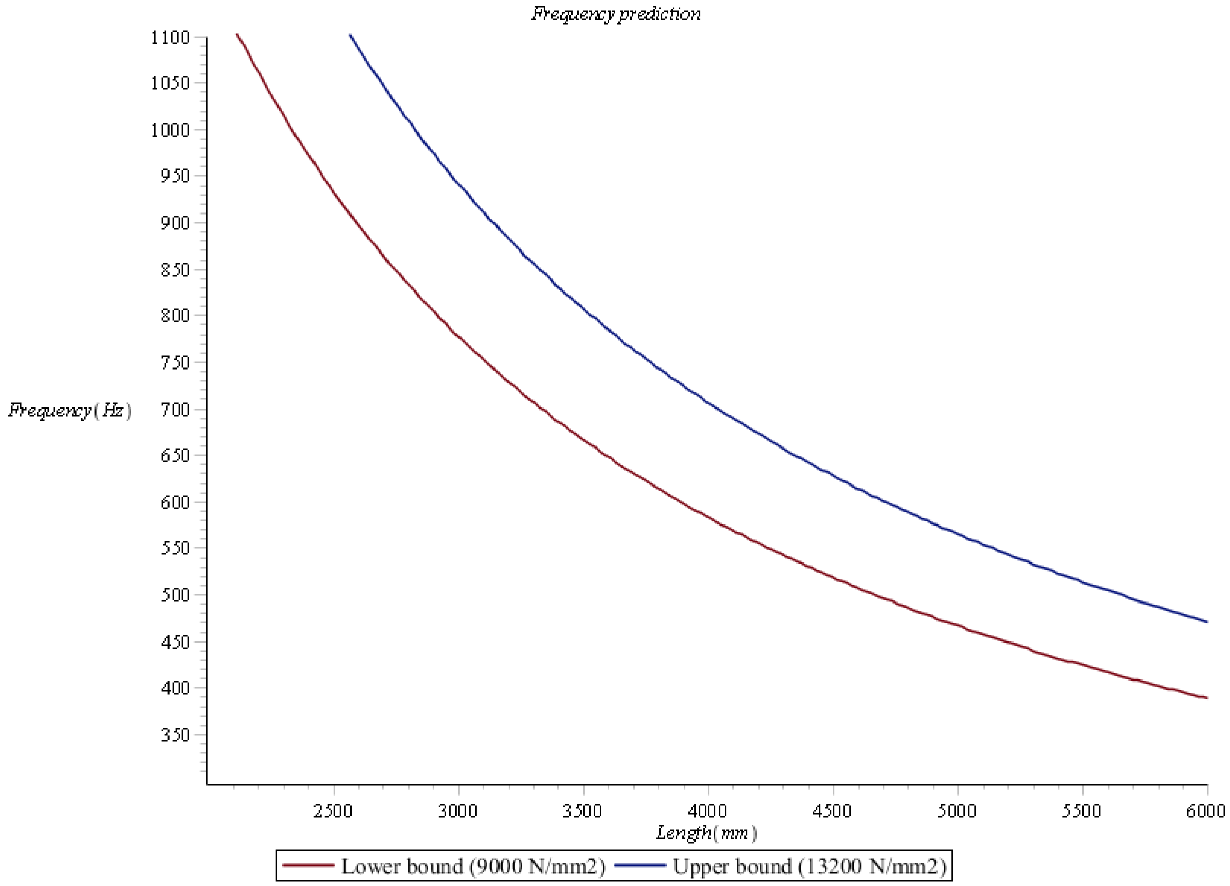


Figure G-18: Graphical representation for predicting the frequency. Here C1 = 0,94 and C2 = 1. Draw a vertical line from the beam length and on the intersection draw two horizontal lines, now an interval can be read on the vertical axis

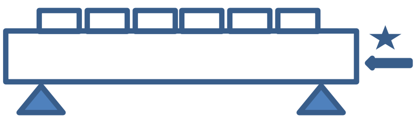
Test 1: Free vibration

<u>Test 1.1</u>
Hammer (★): Beam edge
Meter (⬅): Beam edge

<u>Test 1.2</u>
Hammer: Beam edge
Meter: Top

<u>Test 1.3</u>
Hammer: Beam edge
Meter: Side

<u>Test 1.4</u>
Hammer: Beam edge
Meter: Beam edge
Load: Two rows of bricks



Conclusion: Measurement can be taken with the meter on the side or top and load on top increases frequency

Test 2: Ways of introducing waves

<u>Test 2.1: Hitting block on side attached with clamp</u>
Hammer: Block
Meter: Beam edge / top / side
Result: Meter on beam edge works but when the meter is on top or the side than the signal varies a lot.

<u>Test 2.2: Hitting block on side attached with clamp</u>
Hammer: Beam edge
Meter: Block
Result: Signal gives lower frequency than free vibration.

<u>Test 2.3: Hitting block on side attached with one nail</u>
Hammer: Block
Meter: Beam edge / top / side
Result: Measurements on the edge comply with free vibration. When the meter is on top or the side the signal becomes harder to interpret and shows a higher frequency of 4%.

Figure G-19: One side block

<u>Test 2.4: Hitting block on side attached with one nail</u>
Hammer: Beam edge
Meter: Block
Result: Measurements on the edge comply with free vibration.

<u>Test 2.5: Two hitting blocks attached on opposite sides with one nail</u>
Hammer: Block left
Meter: Block right
Result: Meter gives error and signal interpretation becomes harder. However a frequency equal to the free vibration can be found.

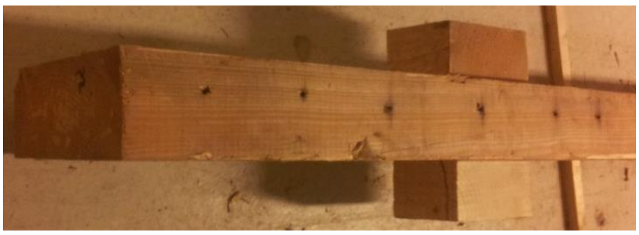


Figure G-20: One side block for placing the meter and one side block for hitting

Test 2.6: Screw on side

Hammer: Screw

Meter: Beam edge / top / side

Result: Measurements on the edge comply with free vibration. When the meter is on the side an error is given and the measurement deviates 10%. Placing the meter on top a frequency was found that equals the free vibration without errors.

Figure G-21: A screw under an angle for hitting

Meter on edge:

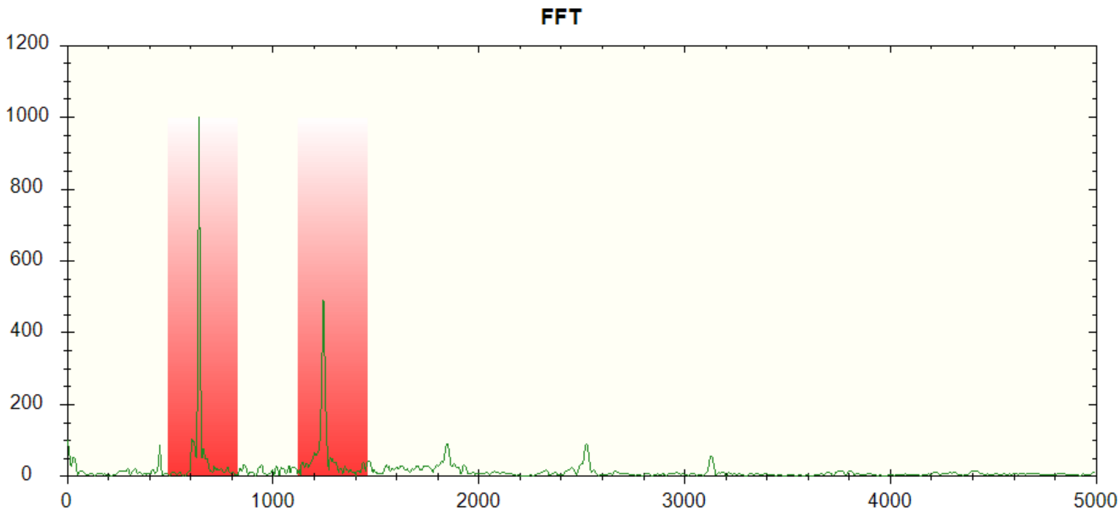


Figure G-22: Display of the base- and overtones, meter on edge and hitting the screw

Meter on top:

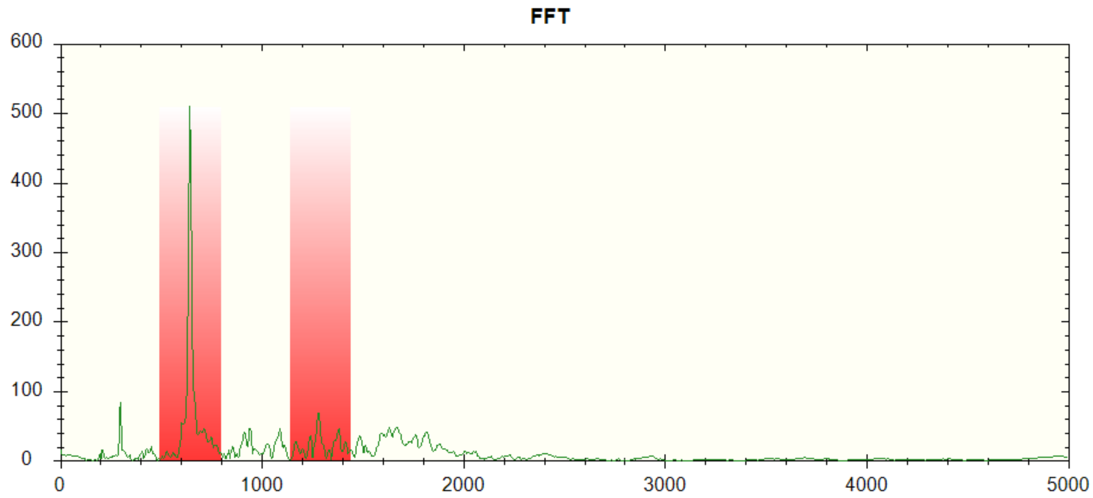


Figure G-23: Display of the base- and overtones, meter at top side and hitting the screw

Conclusion: A screw on the side works best for introducing a wave. The screw is easy to apply but a hole is left in the timber. It is possible to use regular screw sizes but after a few strikes the head starts to deform.

Test 3: Influence of the surroundings

<u>Test 3.1: Timber plates on top without connectors</u>
Hammer: Beam edge
Meter: Beam edge
Result: Frequency is 2 % higher than free vibration

Figure G-24: Timber plates placed on top of beam to simulate the decking

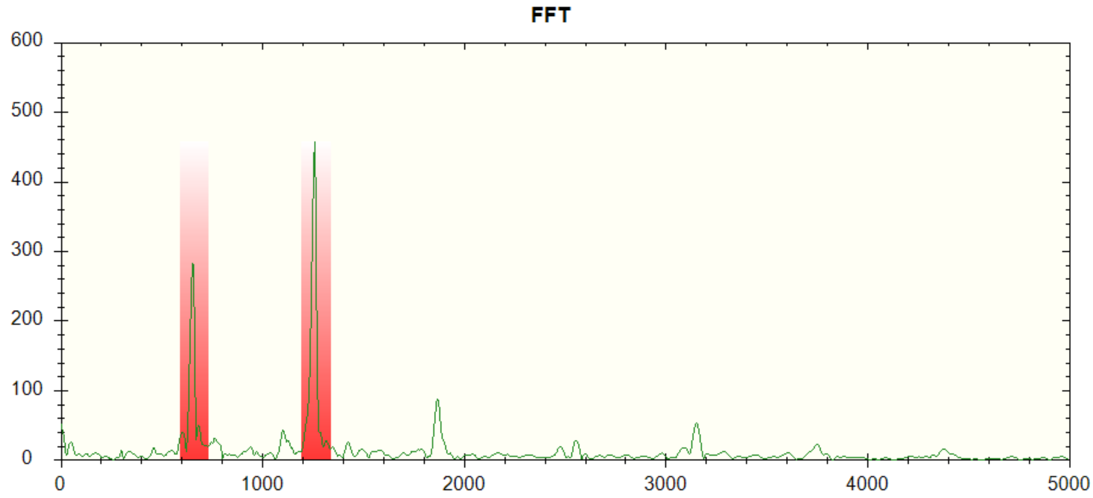


Figure G-25: Display of base- and overtones, meter on edge and hitting the edge (plates not connected)

Test 3.2: Timber plates on top connected with nails
Hammer: Beam edge
Meter: Beam edge
Result: Frequency is 7% higher than free vibration

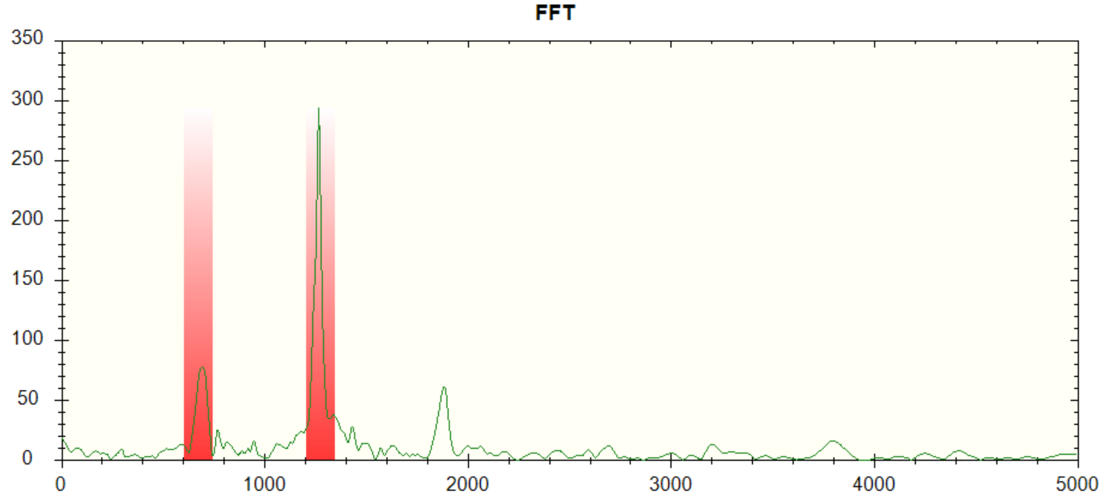


Figure G-26: Display of base- and overtones, meter on edge and hitting the edge (plates connected with nails)

Test 3.3: Member supported by two masonry walls of clay bricks
Hammer: Beam edge
Meter: Beam edge
Result: Frequency varies between 2% and 6% higher than free vibration

Figure G-27: Timber beam placed in two masonry walls of clay bricks

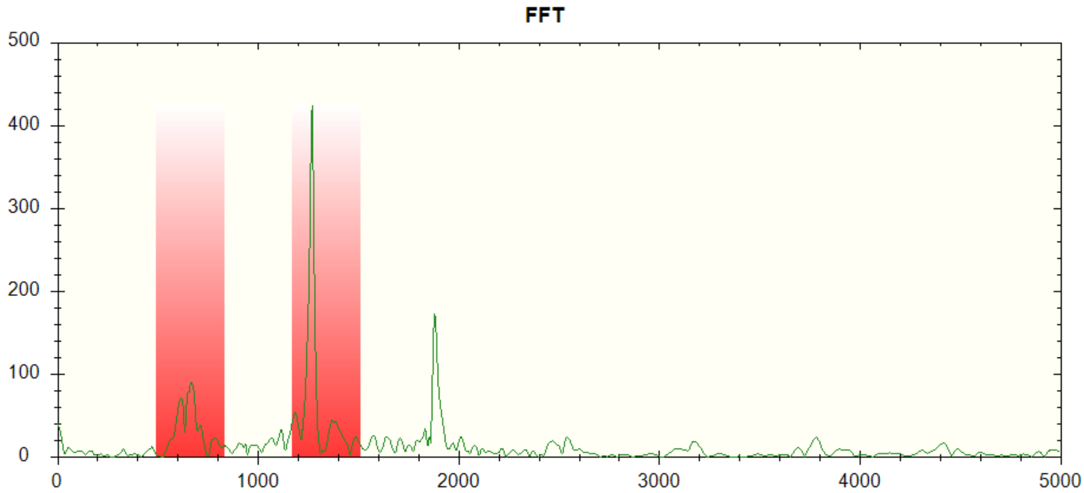


Figure G-28: Display of base- and overtones, meter on edge and hitting the edge (clay bricks)

Test 3.4: Member supported by two masonry walls of limestone
Hammer: Beam edge
Meter: Beam edge
Result: Frequency is 2% higher than free vibration

Figure G-29: Timber beam placed in two masonry walls of limestone

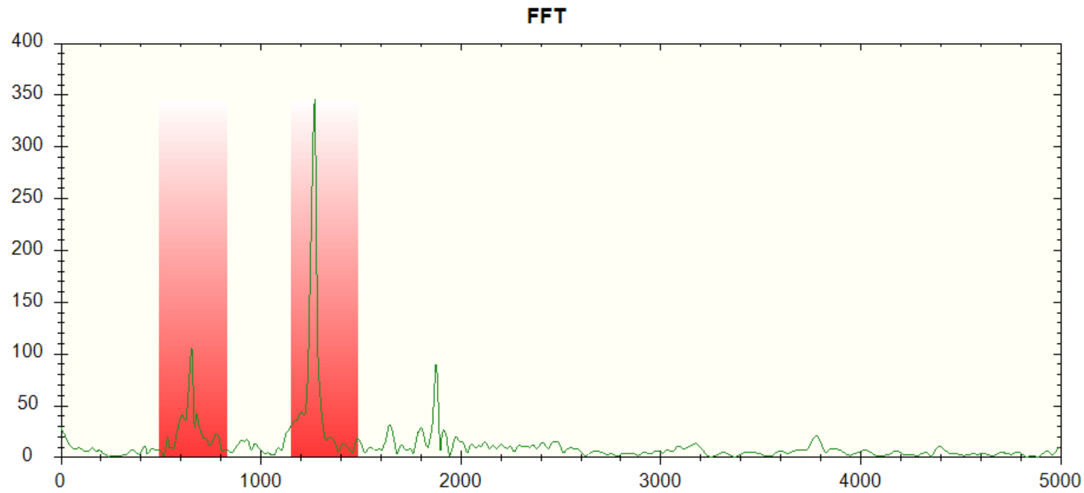


Figure G-30: Display of base- and overtones, meter on edge and hitting the edge (limestones)

Conclusion: Both the decking and the walls increase the frequency

Test 4: Simulation of reality

Figure G-31: Timber beam placed in limestone wall with timber plates on top

Test 4.1: Member supported by two masonry walls of limestone, plates connected with nails and screw on side

Hammer: Screw

Meter: Beam edge

Result: Frequency is 4-8% higher than free vibration

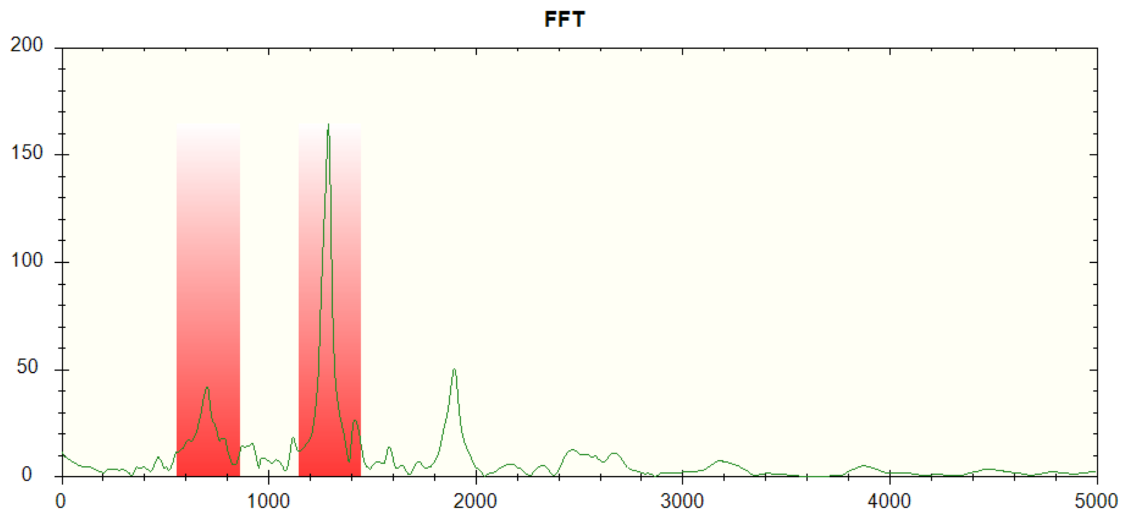


Figure G-32: Display of base- and overtones, meter on edge and hitting the screw (simulation)

<u>Test 4.2: Member supported by two masonry walls of limestone, plates connected with nails and screw on side</u>
Hammer: Screw
Meter: Bottom
Result: The meter gives an error and the signal needs to interpret manual:

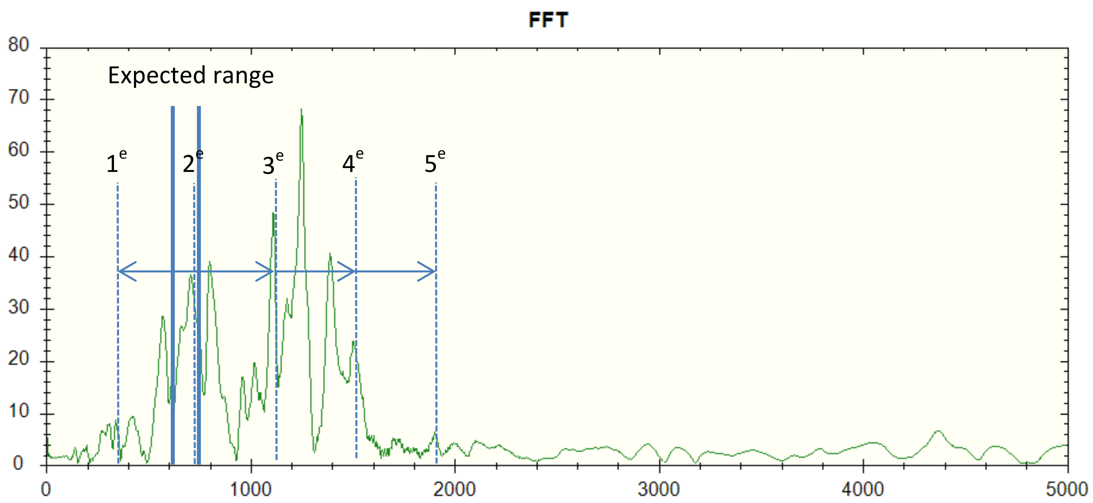


Figure G-33: Display of base- and overtones, meter on bottom side and hitting the screw (simulation)

First an estimation of the frequency is made. From the previous tests it is known that the frequency will be higher than for free vibration. Therefore the prediction model should be adjusted. Based on previous results the frequency is expected to be 6% higher, therefore C2=1,06. The expected frequency lies between 608 Hz and 736 Hz. One peak is found in this interval on 708 Hz. The last check is verify if the distances between the overtones are the same. The frequency of 708 Hz is the same value as was found in test 4.1.

Conclusion: The signals from measurement in situ need to be analyzed manual. However the measured frequencies will be higher than measurements from free vibration. Most of the frequencies lie close to each other from which the average can be taken.

Test 5: In situ

Two flat roofs from different garage were used for this test due to their accessibility. The waves were introduced by aid of a screw. Both the original and adjusted meter were used to measure the frequency.

Test 5.1: Garage with timber planks, mastic and gravel (1964)

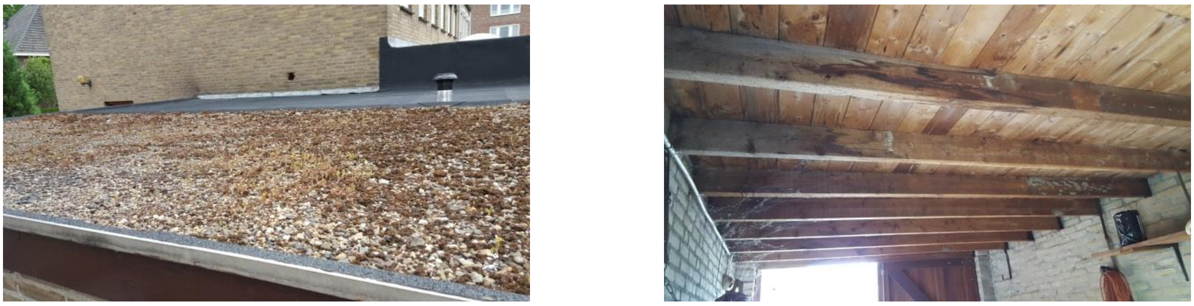


Figure G-34: Photographs of in-situ situation location 1

The length of the members is 3 meter and the expected strength class is standard building wood (MOE = 10000N/mm²) with an average density of 440 kg/m³. This would give a frequency of 855 Hz. It can be noted that the beams are discolored and moisture penetrated the beam near the support. This increases the change of biological attacks.

Original meter (internal sensor, measuring in transverse direction):

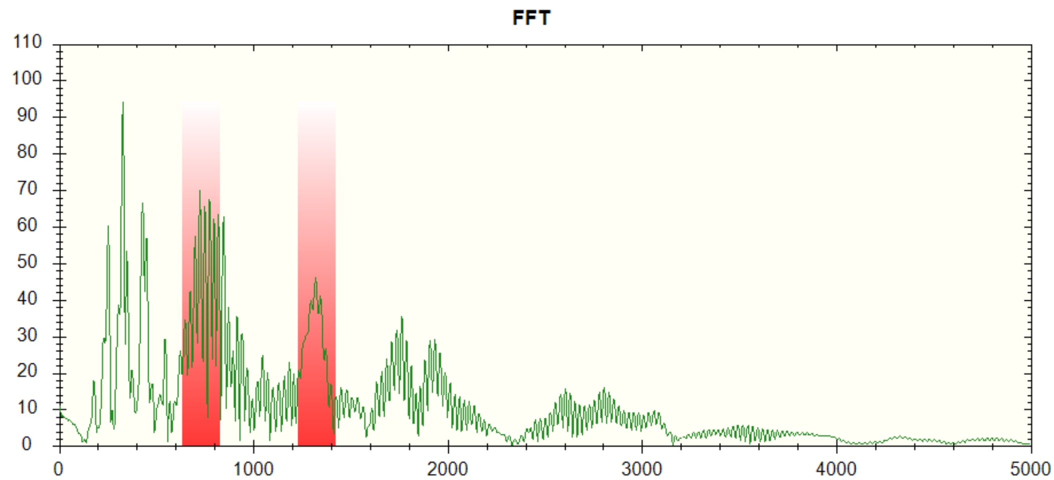


Figure G-35: Display of base- and overtones, meter on side and hitting the screw (In-situ 1 original meter)

40 measures were performed and 5 signals were chosen for their quality. This resulted in an average of 753 Hz. The overestimation might be due to a high moisture content which lowers the dynamic properties.

Adjusted meter (external sensor, measuring in longitudinal direction):

Two additional tests were needed to determine the best location and way of connecting between the sensor and the beam. Four connections were tested: hold the sensor on the surface by hand, use a glue clamp, connect with 1 screw and connect with 2 screws. The latter showed the best result. Here it is essential that a line between the 2 screws is perpendicular to the wave and thus the longitudinal direction. The locations are less sensible to disturbances, but a clear distinction can be made when the sensor is on the bottom or the side. Both locations showed acceptable results.

On the bottom near the introduction of the wave:

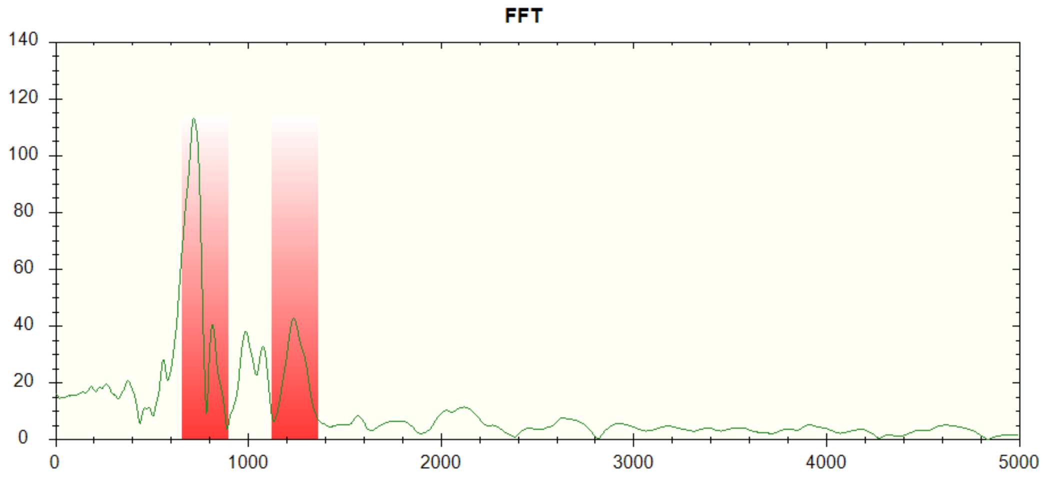


Figure G-36: Display of base- and overtones, meter on bottom and hitting the screw (In situ 1 adjusted meter)

The average value of 5 measurements is 749 Hz. This value is close to the measurements with the original meter.

On the side close to the support:

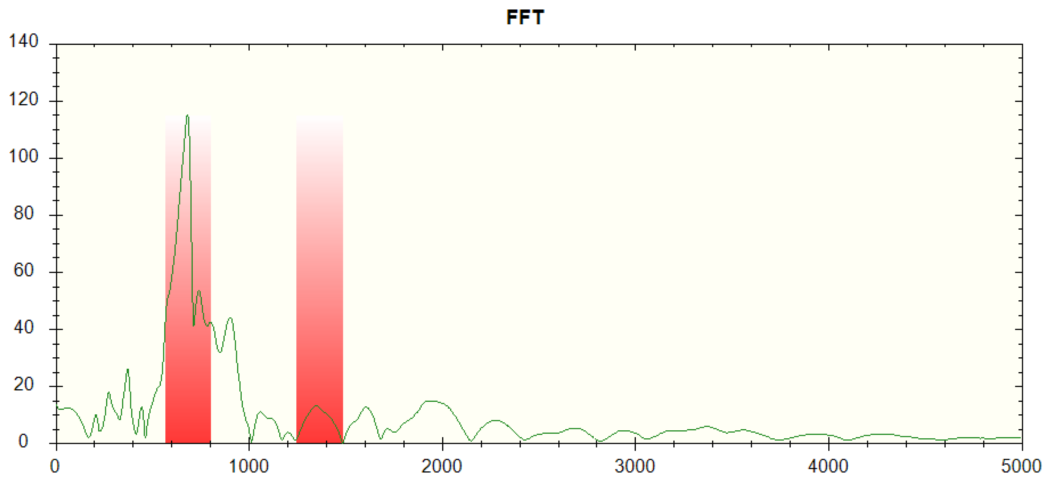


Figure G-37: Display of base- and overtones, meter on side and hitting the screw (In situ 1 adjusted meter)

The average value of 5 measurements is 676 Hz.

More research is required to determine if these results are reliable and which location must be used.

It is interesting to see the result of the external sensor measuring in transverse direction. Although the signal looks unreliable, the following frequencies can be found:

- On the bottom near the introduction of the wave: 766 Hz
- On the side close to the support: 566 Hz

Test 5.2: Garage with timber boards and mastic (1969)

Figure G-38: Photograph of in-situ situation location 2

The expected frequency is 815 Hz which is associated with a length of 3,2 meter and class standard building wood. No biological damage is observerd.

Original meter (internal sensor, measuring in transverse direction):

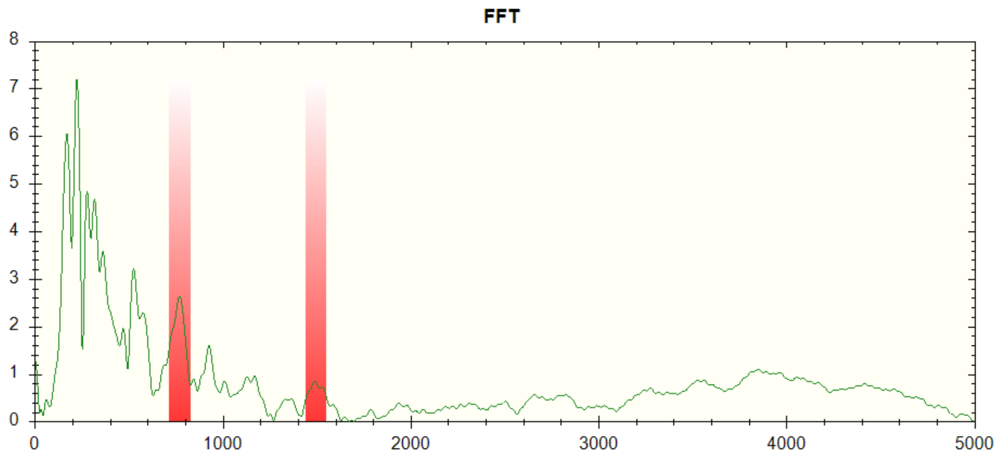


Figure G-39: Display of base- and overtones, meter on side and hitting the screw (in-situ 2 original meter)

37 measures were taken and 5 signals were chosen for their quality. The average result is 821 Hz, which is close to the predicted value.

Adjusted meter (external sensor, measuring in longitudinal direction):
Two locations were tested: close to the support and close to the impact.

On the side close to the impact:

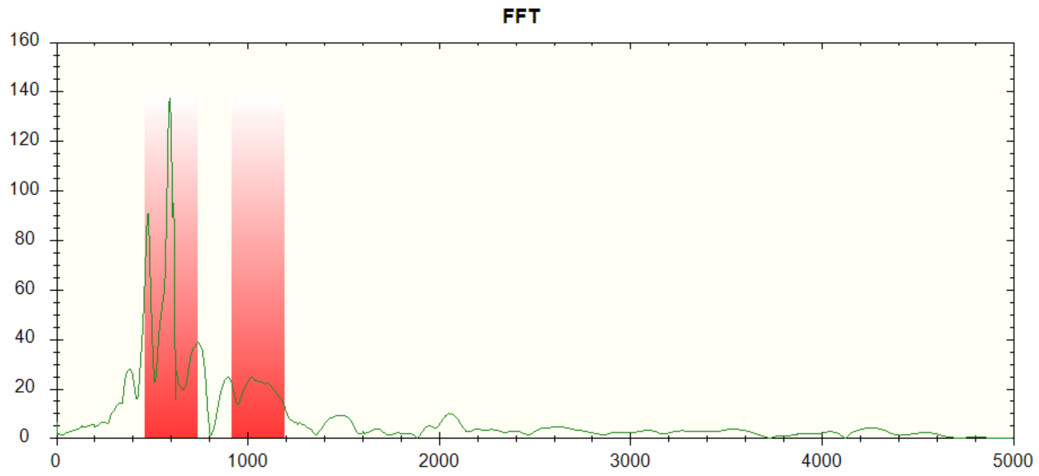


Figure G-40: Display of base- and overtones, meter on side near impact and hitting the screw (in-situ 2 adjusted meter)

The average of 5 measurements is 577 Hz.

On the side close to the support:

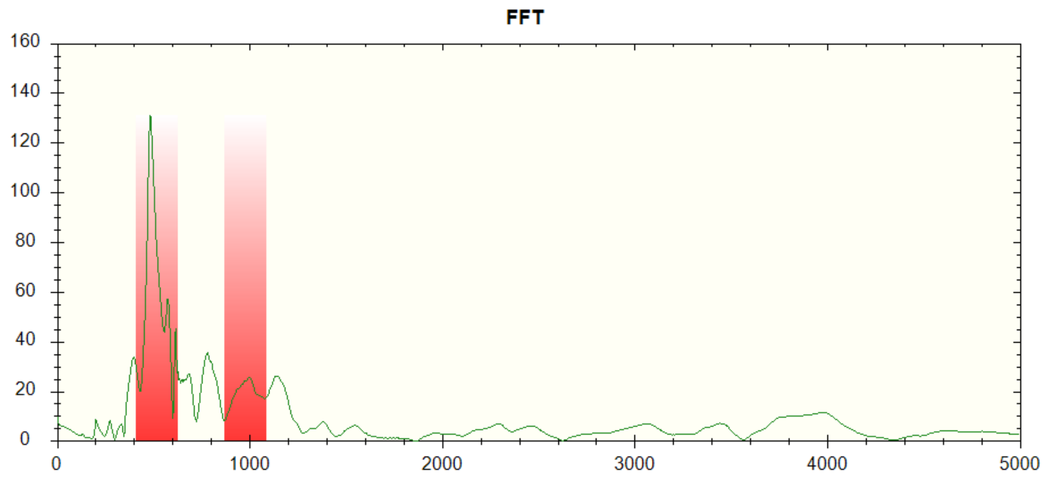


Figure G-41: Display of base- and overtones, meter on side near support and hitting the screw (in-situ 2 adjusted meter)

The average of 5 measurements is 486 Hz.

Conclusion: In situ measurements requires an experienced user to evaluate the signals because the signal quality is bad in the frequency domain. A prediction can help to find the right frequency, however more research is needed to the influence of the surroundings and the best location. Also the size of the impact on the screw matters. A small tap shows a better result than a hard smash.

It is clear that the adjusted meter has a better quality in the frequency domain because it measures in longitudinal direction.

A better conclusion can be made when an in-situ measured beam can be subjected to a bending test. A semi-destructive solution can be to perform a hardness test or a tension test on micro specimens. These tests also give an indication of the E-modulus.

Results of tests in Hz:

	Test															
Beam number	1.1	1.2	1.3	1.4	2.1	2.2	2.3	2.4	2.5	2.6	3.1	3.2	3.3	3.4	4.1	4.2
S1	566										576					
S2	634															
S3	649															
S4	610															
S5	615	615	615				615/630/630	615	(615)							
S6	630				650/NR/(654)	586	630/-/(634)	630			639	673				
S7	634															
S8	644	644	644	690						644/644/-	656	690	665	659	708	(708)
S9	581															
S10	610															
L1	576		576							576/576/(630)						
L2	590															
L3	561															

- = No results
NR = Not reliable
() = Meter gives error

Table G-3: Test results of frequency measurements

G.5.3 FOUR POINT BENDING TEST

The following results were gained:

Member ID	18h,used [mm]	6h,used [mm]	5h,used [mm]	0,1*F,max [kN]	0,4*F,max [kN]	W_{local} at 10% [mm]	W_{local} at 40% [mm]	W_{global} at 10% [mm]	W_{global} at 40% [mm]	W, ultimate [mm]	F,ultimate [kN]
S1	3300	900	750	3,243	13,008	0,361	1,518	6,151	27,366	52,48	23,57
S2	3300	900	750	3,879	15,501	0,256	1,067	5,399	22,550	80,94	37,07
S3	3300	900	750	4,311	14,236	0,321	1,101	5,604	19,366	50,27	34,09
S4	3300	900	750	2,262	9,049	0,182	0,759	3,424	14,807	48,63	22,61
S5	3300	900	750	3,859	15,447	0,276	1,216	5,961	24,371	48,93	28,78
S6	3300	900	750	2,778	11,114	0,176	0,740	3,685	16,023	41,80	27,78
S7											
S8	3300	900	750	3,853	15,408	0,280	1,175	5,236	22,057	70,65	35,50
S9	3300	900	750	3,377	13,506	0,266	1,102	5,951	23,946	67,66	30,55
S10	3300	900	750	3,877	15,520	0,277	1,200	5,955	23,376	51,17	32,50
L1	3300	900	750	4,589	15,329	0,134	0,448	3,190	10,897	47,94	58,24
L2	3300	900/295	750	6,044	21,173	0,211	0,763	5,060	17,492		52,99
L3	3300	600	500	6,223	21,939	0,102	0,358	4,479	16,237		

Table G-4: Test results of four point bending tests

During installing of the beam in the bending machine it was necessary to place the expected place of failure in the middle. In most cases this was due to a knot on the bottom side. Three failure mechanisms were observed:

1. Simple tension (S1,S3,S4,S6,S8,S9,L1,L2)

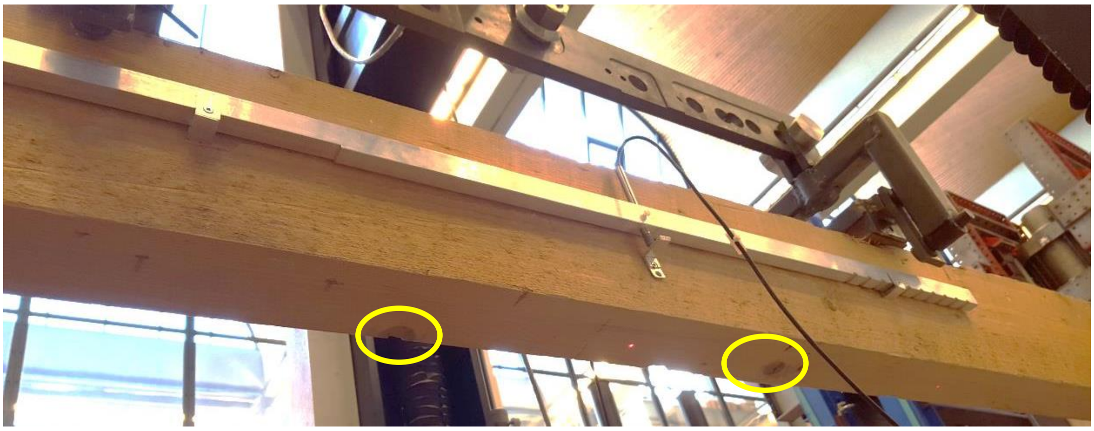


Figure G-42: Photographs before and after loading. Two knots are shown on the bottom side. The crack initiates from one of the knots

The crack initiates from the knot on the bottom side in the tension zone. Here the knot interrupts the grains and thus reduces the available surface. Besides the grain around the knot is curled. After a certain height is cracked the beam splits parallel to the grain along the longitudinal direction.

2. <u>Cross-grained tension (S5,S10)</u>

Figure G-43: Failure in cross grained tension

The failure mechanism occurs when the grain is under an angle. This causes the tension force to work oblique to the grain. The tension strength properties perpendicular to the grain are lower than parallel to the grain. Besides no large knots were present around the middle.

3. <u>Splintering tension (S2)</u>

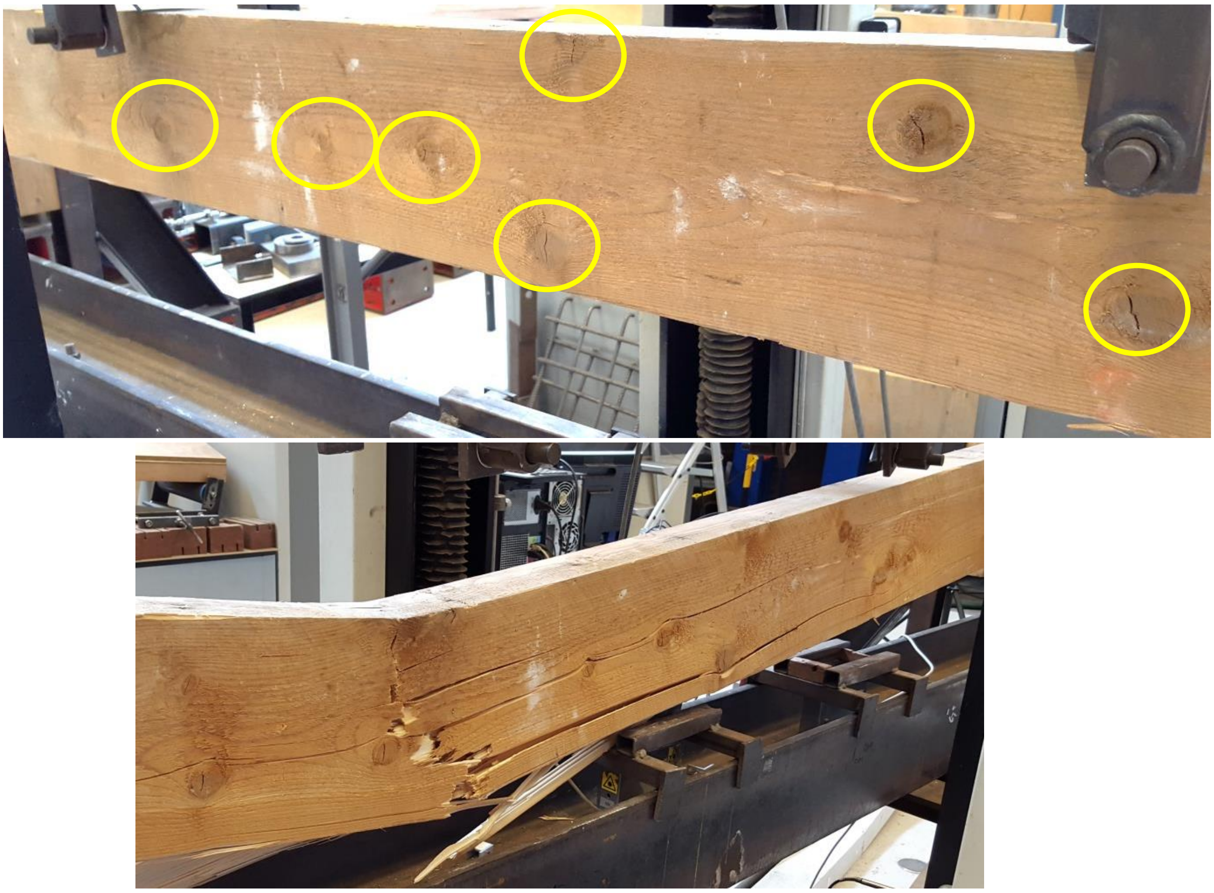


Figure G-44: Photograph before and after loading. The side is full of knots but failure occurred below the pressure point

Note that there are many knots present but failure occurred below the pressure point. The failure mechanism consist out of minor tension failures.

The stress-strain curve only shows a linear tension branch and a brittle failure:

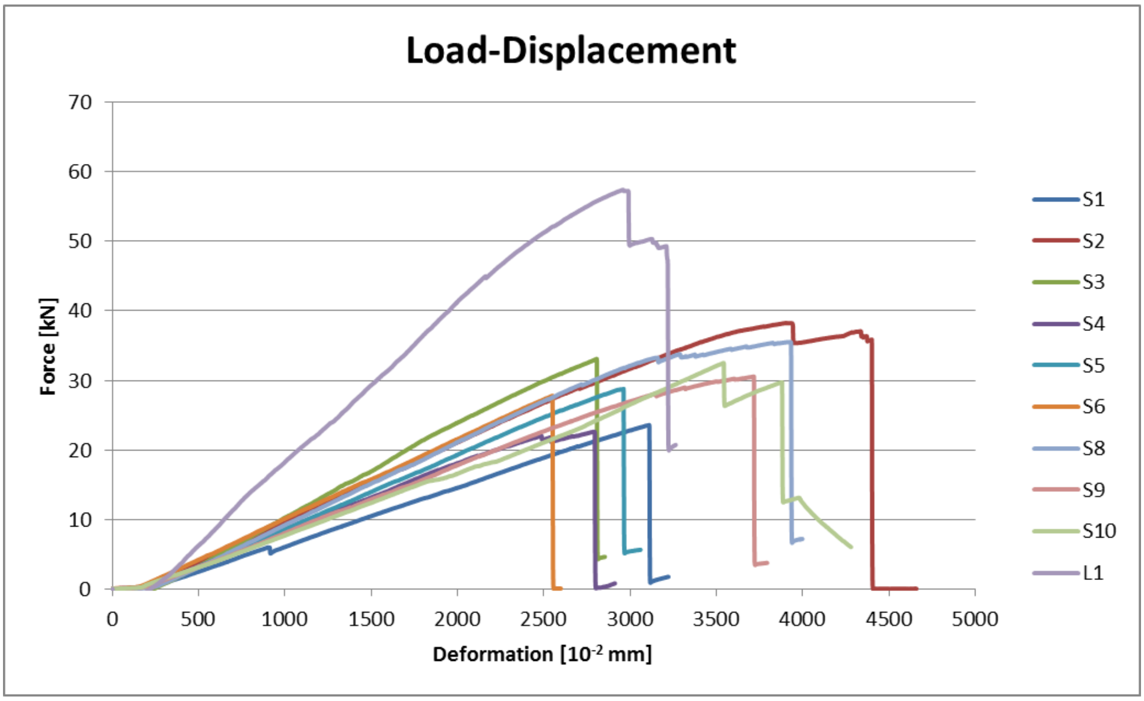


Figure G-45: Load – displacement curve of all beams

The relationship between the different material properties can be shown:

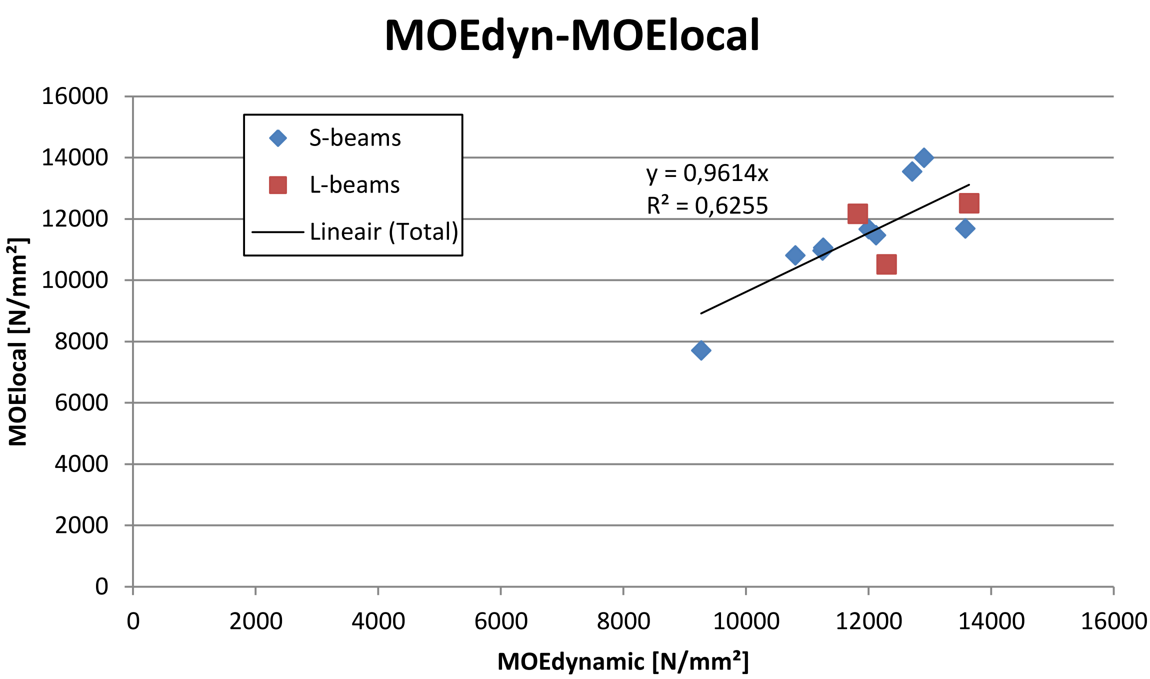


Figure G-46: Relationship between dynamic and local modulus of elasticity

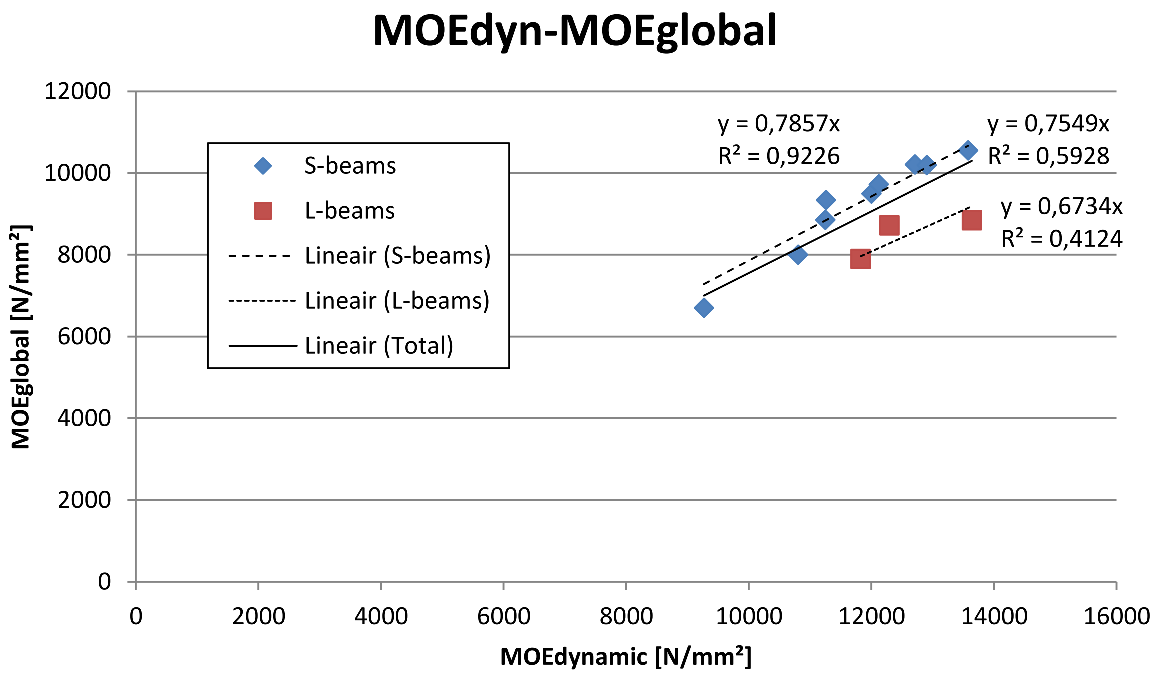


Figure G-47: Relationship between dynamic and global modulus of elasticity

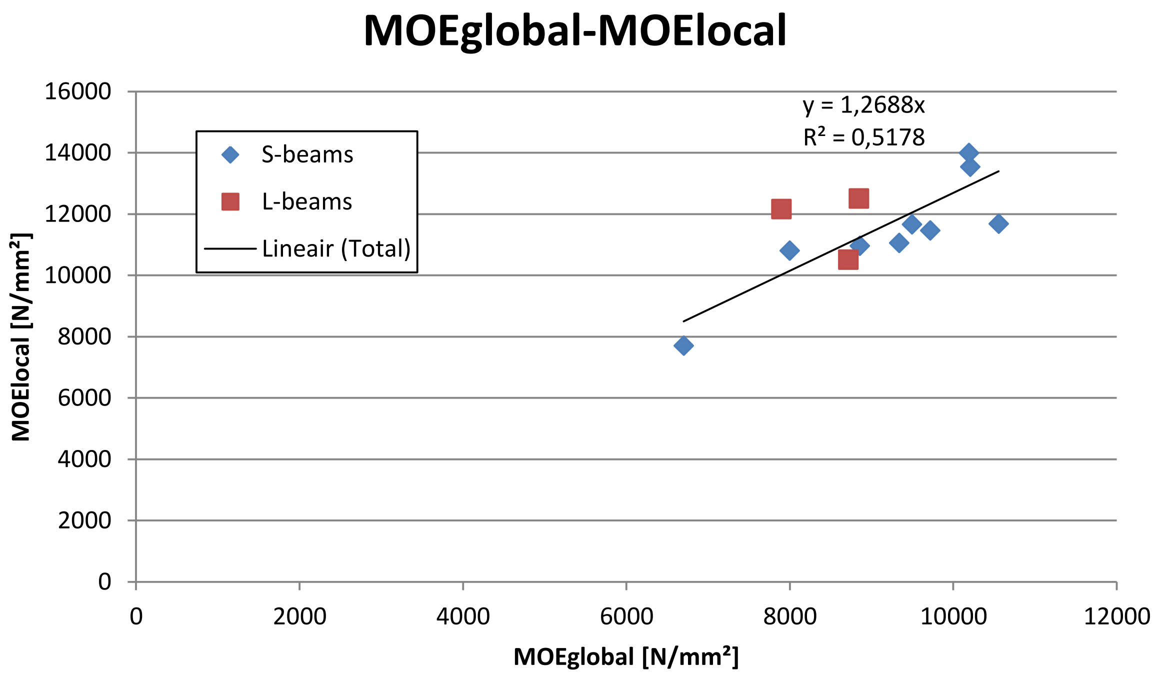


Figure G-48: Relationship between global and local modulus of elasticity

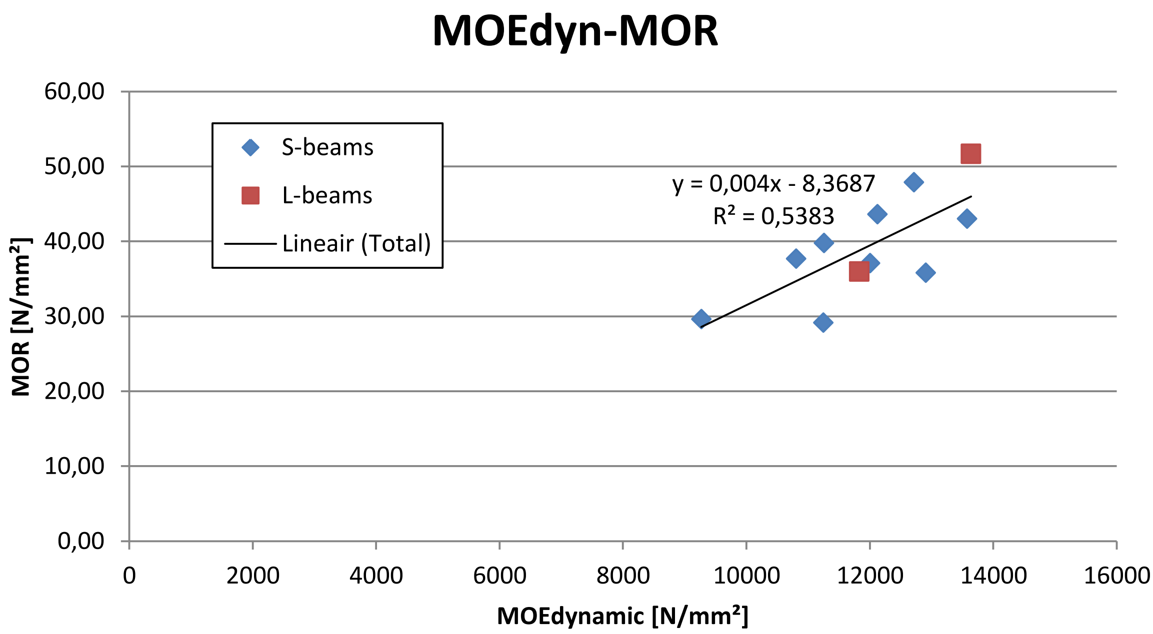


Figure G-49: Relationship between dynamic modulus of elasticity and the modulus of rupture

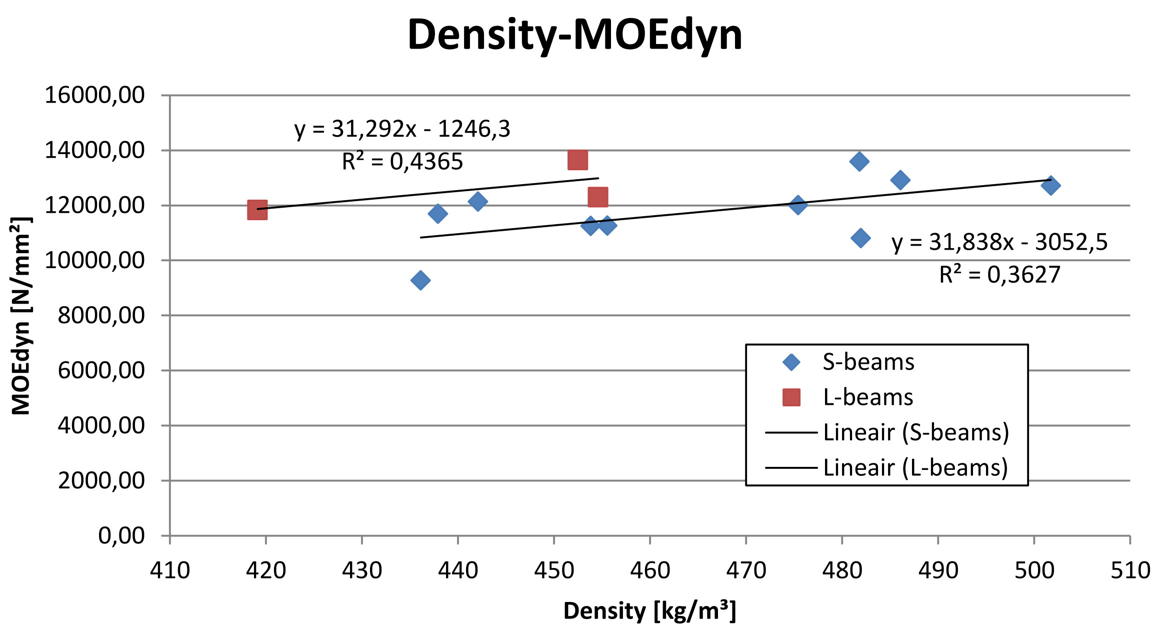


Figure G-50: Relationship between the density and dynamic modulus of elasticity

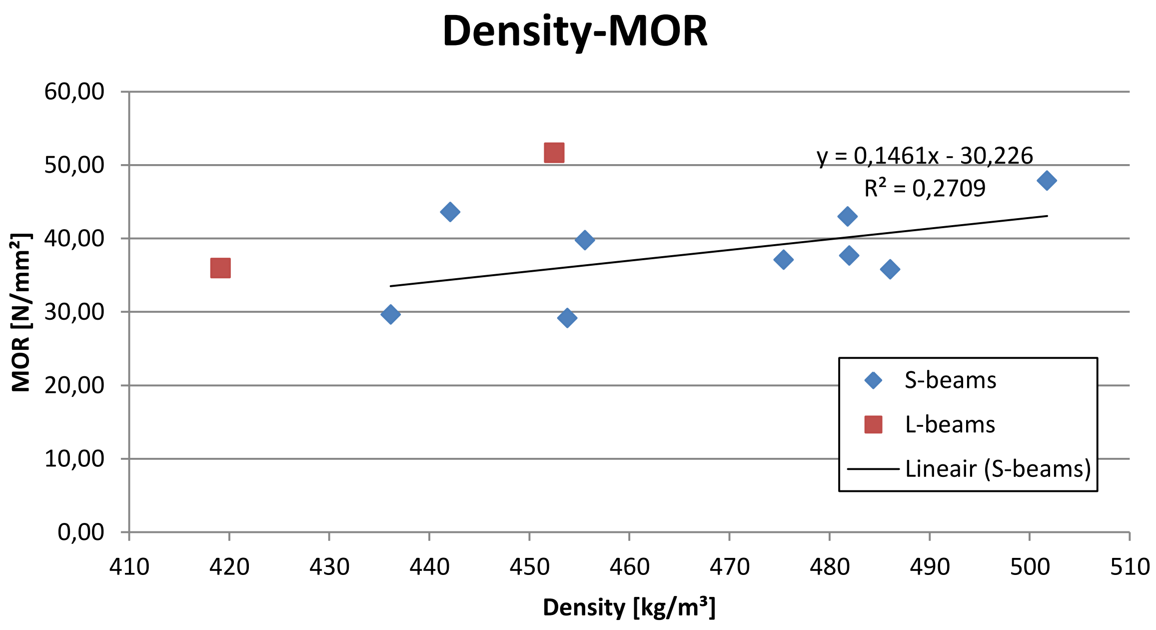


Figure G-51: Relationship between the density and the modulus of rupture

G.6 DERIVATION DEFLECTION INHOMOGENEOUS BEAM

Euler bernoulli beam theory:

$> w1 := \frac{q \cdot x^4}{24 \cdot EI1} + \frac{C1 \cdot x^3}{6} + \frac{C2}{2} \cdot x^2 + C3 \cdot x + C4; w2 := \frac{q \cdot x^4}{24 \cdot EI2} + \frac{C5 \cdot x^3}{6} + \frac{C6}{2} \cdot x^2 + C7 \cdot x + C8;$

$$w1 := \frac{1}{24} \frac{q\, x^4}{EI1} + \frac{1}{6} C1\, x^3 + \frac{1}{2} C2\, x^2 + C3\, x + C4$$

$$w2 := \frac{1}{24} \frac{q\, x^4}{EI2} + \frac{1}{6} C5\, x^3 + \frac{1}{2} C6\, x^2 + C7\, x + C8 \tag{1}$$

$> Phi1 := diff(w1, x) : Phi2 := diff(w2, x) :$

$> M1 := -1 \cdot EI1 \cdot diff(Phi1, x) : M2 := -1 \cdot EI2 \cdot diff(Phi2, x) :$

$> V1 := diff(M1, x) : V2 := diff(M2, x) :$

$> x := 0 :$

$> eq1 := w1 = 0 :$

$> eq2 := M1 = 0 :$

$> x := \frac{L}{3} :$

$> eq3 := w1 = w2 :$

$> eq4 := Phi1 = Phi2 :$

$> eq5 := M1 = M2 :$

$> eq6 := V1 = V2 :$

$> x := \frac{L}{2} :$

$> eq7 := Phi2 = 0 :$

$> eq8 := V2 = 0 :$

$> solution := solve(\{eq1, eq2, eq3, eq4, eq5, eq6, eq7, eq8\}, \{C1, C2, C3, C4, C5, C6, C7, C8\});$

$$solution := \left\{C1 = -\frac{1}{2} \frac{q\, L}{EI1}, C2 = 0, C3 = \frac{1}{648} \frac{q\, L^3\, (13\, EI1 + 14\, EI2)}{EI1\, EI2}, C4 = 0, C5 = -\frac{1}{2} \frac{q\, L}{EI2}, C6 = 0, C7 = \frac{1}{24} \frac{q\, L^3}{EI2}, C8 = -\frac{1}{216} \frac{q\, L^4\, (EI1 - EI2)}{EI1\, EI2}\right\} \tag{2}$$

$> assign(solution);$

$> x := 'x';$

$$x := x \tag{3}$$

$> print(w1); print(w2); whomogeneous := \frac{5}{384} \cdot \frac{q \cdot L^4}{EI1};$

$$\frac{1}{24} \frac{q\, x^4}{EI1} - \frac{1}{12} \frac{q\, L\, x^3}{EI1} + \frac{1}{648} \frac{q\, L^3\, (13\, EI1 + 14\, EI2)\, x}{EI1\, EI2}$$

$$\frac{1}{24} \frac{q\, x^4}{EI2} - \frac{1}{12} \frac{q\, L\, x^3}{EI2} + \frac{1}{24} \frac{q\, L^3\, x}{EI2} - \frac{1}{216} \frac{q\, L^4\, (EI1 - EI2)}{EI1\, EI2}$$

$$\tag{4}$$

$$whomogeneous := \frac{5}{384}\,\frac{q\,L^4}{EI1} \tag{4}$$

> $x := \frac{L}{2}$: $\varepsilon := whomogeneous - w2;$

$$\varepsilon := \frac{5}{384}\,\frac{q\,L^4}{EI1} - \frac{5}{384}\,\frac{q\,L^4}{EI2} + \frac{1}{216}\,\frac{q\,L^4\,(EI1 - EI2)}{EI1\,EI2} \tag{5}$$

> $simplify(\ (5), 'size'\ )$

$$-\frac{29}{3456}\,\frac{q\,L^4\,(EI1 - EI2)}{EI1\,EI2} \tag{6}$$

G.7 STRATEGIES APPLIED ON A CASE

The case of Kerkhofstraat is used for testing the strategies. The assumptions are as follow:

Strength class: C18
Material: Sawn timber
Consequence class: CC2
Building category: H – Roofs
Climate class: 2
Duration of load class: permanent (permanent load) and short (variable load)

The following abbreviations are used:

Geometrie

hoh = distance between beams [m]
l = length of one beam [m]
b = width of one beam [m]
h = height of one beam [m]
W = cross section modulus [m^3]
I = moment of inertia [m^4]
t = height of decking

Factors

gammag1 ($\gamma_{g,1}$) = Load factor for only permanent load (ULS)
gammag2 ($\gamma_{g,2}$) = Load factor for permanent load with variable load (ULS)
gammaq (γ_q) = Load factor for variable load (ULS)
Kmod1 ($K_{mod,1}$) = Strength modification factor for only permanent load, duration class permanent
Kmod2 ($K_{mod,2}$) = Strength modification factor for variable load, duration class short
gammasls (γ_{sls}) = Load factor for SLS
gammam (γ_m) = Material factor
kdef (k_{def}) = Deformation modification factor

Material properties

fmk ($f_{m,o,k}$) = Characteristic bending strength [N/mm²]
E0mean ($E_{o,mean}$) = Characteristic modulus of elasticity [N/mm²]

Load

Permanent (G_1) = Permanent load [kN/m]
udl (Q_1) = Uniformly distributed load (maintenance) [kN/m]
GRext (G_2) = Extensive green roof load [kN/m]
GRint1 (G_3) = Intensive green roof load (low weight) [kN/m]
GRint2 (G_4) = Intensive green roof load (high weight) [kN/m]
GRuse1 (Q_2) = Variable load associated with G_3 [kN/m]
GRuse2 (Q_3) = Variable load associated with G_4 [kN/m]

Geometrie Kerkhofstraat

> $hoh := 0.600 : l := 4.100 : b := 0.075 : h := 0.195 : W := \left(\frac{1}{6}\right) \cdot b \cdot h^2 : Iy := \left(\frac{1}{12}\right) \cdot b \cdot h^3 :$

Eurocode

Factors Eurocode

> $gammag1 := 1.35 : gammag2 := 1.2 : gammaq := 1.5 : kmod1 := 0.60 : kmod2 := 0.90 : gammasls := 1 : gammam := 1.3 : kdef := 0.80 :$

Material properties C18

> $fmk := 18 : E0mean := 9000 :$

Load

> permanent := 0.6·hoh; udl := 1·hoh; GRext := 1·hoh; GRint1 := 1·hoh; GRint2 := 3.4·hoh; GRuse1 := 0.60·1.75·hoh; GRuse2 := 0.80·1.75·hoh;

$$permanent := 0.3600$$
$$udl := 0.600$$
$$GRext := 0.600$$
$$GRint1 := 0.600$$
$$GRint2 := 2.0400$$
$$GRuse1 := 0.6300000$$
$$GRuse2 := 0.8400000 \qquad (1)$$

Load combination

> $LC1 := gammag1 \cdot (permanent + GRext); LC2 := gammag1 \cdot (permanent + GRint2); LC3 := gammag2 \cdot (permanent + GRext) + gammaq \cdot udl; LC4 := gammag2 \cdot (permanent + GRint1) + gammaq \cdot GRuse1; LC5 := gammag2 \cdot (permanent + GRint2) + gammaq \cdot GRuse2; LC6 := gammasls \cdot (permanent + GRext); LC7 := gammasls \cdot (permanent + GRint2); LC8 := gammasls \cdot (permanent + GRext) + gammasls \cdot udl; LC9 := gammasls \cdot (permanent + GRint1) + gammasls \cdot GRuse1; LC10 := gammasls \cdot (permanent + GRint2) + gammasls \cdot GRuse2;$

$$LC1 := 1.296000$$
$$LC2 := 3.240000$$
$$LC3 := 2.05200$$
$$LC4 := 2.09700000$$
$$LC5 := 4.14000000$$
$$LC6 := 0.9600$$
$$LC7 := 2.4000$$
$$LC8 := 1.5600$$
$$LC9 := 1.5900000$$
$$LC10 := 3.2400000 \qquad (2)$$

Moment

> $M1 := \left(\frac{1}{8}\right) \cdot LC1 \cdot l^2; M2 := \left(\frac{1}{8}\right) \cdot LC2 \cdot l^2; M3 := \left(\frac{1}{8}\right) \cdot LC3 \cdot l^2; M4 := \left(\frac{1}{8}\right) \cdot LC4 \cdot l^2; M5 := \left(\frac{1}{8}\right) \cdot LC5 \cdot l^2;$

$$M1 := 2.723220000$$

$$M2 := 6.808050000$$
$$M3 := 4.311765000$$
$$M4 := 4.406321250$$
$$M5 := 8.699175000 \quad \textbf{(3)}$$

Ultimate limit state according to EC5 art. 6.1.6

> $ULS1 := \frac{kmod1 \cdot fmk}{gammam}$; $ULS2 := \frac{kmod2 \cdot fmk}{gammam}$;

$$ULS1 := 8.307692308$$
$$ULS2 := 12.46153846 \quad \textbf{(4)}$$

> $Check1 := evalf\left(\frac{M1}{W} \cdot 10^{-3} \leq ULS1, 3\right)$; $Check2 := evalf\left(\frac{M2}{W} \cdot 10^{-3} \leq ULS1, 3\right)$; $Check3 := evalf\left(\frac{M3}{W} \cdot 10^{-3} \leq ULS2, 3\right)$; $Check4 := evalf\left(\frac{M4}{W} \cdot 10^{-3} \leq ULS2, 3\right)$; $Check5 := evalf\left(\frac{M5}{W} \cdot 10^{-3} \leq ULS2, 3\right)$;

$$Check1 := 5.73 \leq 8.31$$
$$Check2 := 14.3 \leq 8.31$$
$$Check3 := 9.07 \leq 12.5$$
$$Check4 := 9.28 \leq 12.5$$
$$Check5 := 18.3 \leq 12.5 \quad \textbf{(5)}$$

Serviceability lime state according to EC5 art. 7.2

> $Ecreep := \frac{E0mean}{(1 + kdef)}$;

$$Ecreep := 5000.000000 \quad \textbf{(6)}$$

> $Winst1 := evalf\left(\frac{5}{384} \cdot \frac{LC8 \cdot l^4}{E0mean \cdot Iy}, 3\right)$; $Winst2 := evalf\left(\frac{5}{384} \cdot \frac{LC9 \cdot l^4}{E0mean \cdot Iy}, 3\right)$; $Winst3 := evalf\left(\frac{5}{384} \cdot \frac{LC10 \cdot l^4}{E0mean \cdot Iy}, 3\right)$; $Wfin1 := evalf\left(\frac{5}{384} \cdot \frac{LC6 \cdot l^4}{Ecreep \cdot Iy} + \frac{5}{384} \cdot \frac{(LC8 - LC6) \cdot l^4}{E0mean \cdot Iy}, 3\right)$; $Wfin2 := evalf\left(\frac{5}{384} \cdot \frac{LC6 \cdot l^4}{Ecreep \cdot Iy} + \frac{5}{384} \cdot \frac{(LC9 - LC6) \cdot l^4}{E0mean \cdot Iy}, 3\right)$; $Wfin3 := evalf\left(\frac{5}{384} \cdot \frac{LC7 \cdot l^4}{Ecreep \cdot Iy} + \frac{5}{384} \cdot \frac{(LC10 - LC7) \cdot l^4}{E0mean \cdot Iy}, 3\right)$;

$$Winst1 := 13.8$$
$$Winst2 := 14.1$$
$$Winst3 := 28.6$$
$$Wfin1 := 20.5$$
$$Wfin2 := 20.8$$
$$Wfin3 := 45.7 \quad \textbf{(7)}$$

> $Wallowed := \frac{l \cdot 10^3}{250}$;

$$Wallowed := 16.400 \quad \textbf{(8)}$$

Decking

> $t := 0.021 : density := 410 : fmk := 14.8 : E0mean := 7800 : Wdeck := \left(\frac{1}{6}\right) \cdot 1 \cdot t^2; Iydeck := \left(\frac{1}{12}\right) \cdot 1 \cdot t^3; gammamdeck := 1.2 : kdefdeck := 1.0 :$

$$Wdeck := 0.00007350000000$$

$$Iydeck := 7.717500000\ 10^{-7} \qquad \textbf{(9)}$$

Load

> $permanent := \frac{density}{100} \cdot \mathrm{t} + 0.15 : \mathrm{udl} := 1 : \mathrm{GRext} := 1 : \mathrm{GRint1} := 1 : \mathrm{GRint2} := 3.4 : \mathrm{GRuse1} := 0.60 \cdot 1.75 : \mathrm{GRuse2} := 0.80 \cdot 1.75 :$

Load combination

> $LC1 := gammag1 \cdot (permanent + GRext); LC2 := gammag1 \cdot (permanent + GRint2); LC3 := gammag2 \cdot (permanent + GRext) + gammaq \cdot udl; LC4 := gammag2 \cdot (permanent + GRint1) + gammaq \cdot GRuse1; LC5 := gammag2 \cdot (permanent + GRint2) + gammaq \cdot GRuse2; LC6 := gammasls \cdot (permanent + GRext); LC7 := gammasls \cdot (permanent + GRint2); LC8 := gammasls \cdot (permanent + GRext) + gammasls \cdot udl; LC9 := gammasls \cdot (permanent + GRint1) + gammasls \cdot GRuse1; LC10 := gammasls \cdot (permanent + GRint2) + gammasls \cdot GRuse2;$

$$LC1 := 1.668735000$$
$$LC2 := 4.908735000$$
$$LC3 := 2.983320000$$
$$LC4 := 3.058320000$$
$$LC5 := 6.463320000$$
$$LC6 := 1.236100000$$
$$LC7 := 3.636100000$$
$$LC8 := 2.236100000$$
$$LC9 := 2.286100000$$
$$LC10 := 5.036100000 \qquad \textbf{(10)}$$

Moment

> $M1 := \left(\frac{1}{8}\right) \cdot LC1 \cdot hoh^2; M2 := \left(\frac{1}{8}\right) \cdot LC2 \cdot hoh^2; M3 := \left(\frac{1}{8}\right) \cdot LC3 \cdot hoh^2; M4 := \left(\frac{1}{8}\right) \cdot LC4 \cdot hoh^2; M5 := \left(\frac{1}{8}\right) \cdot LC5 \cdot hoh^2; M6 := \left(\frac{1}{8}\right) \cdot LC6 \cdot hoh^2; M7 := \left(\frac{1}{8}\right) \cdot LC7 \cdot hoh^2; M8 := \left(\frac{1}{8}\right) \cdot LC8 \cdot hoh^2; M9 := \left(\frac{1}{8}\right) \cdot LC9 \cdot hoh^2; M10 := \left(\frac{1}{8}\right) \cdot LC10 \cdot hoh^2;$

$$M1 := 0.07509307500$$
$$M2 := 0.2208930750$$
$$M3 := 0.1342494000$$
$$M4 := 0.1376244000$$
$$M5 := 0.2908494000$$
$$M6 := 0.05562450000$$
$$M7 := 0.1636245000$$
$$M8 := 0.1006245000$$
$$M9 := 0.1028745000$$
$$M10 := 0.2266245000 \qquad \textbf{(11)}$$

Ultimate limit state according to EC5 art. 6.1.6

> $ULS1 := \frac{kmod1 \cdot fmk}{gammamdeck}$; $ULS2 := \frac{kmod2 \cdot fmk}{gammamdeck}$;

$$ULS1 := 7.400000000$$
$$ULS2 := 11.10000000 \tag{12}$$

> $Check1 := evalf\left(\frac{M1}{Wdeck} \cdot 10^{-3} \le ULS1, 3\right)$; $Check2 := evalf\left(\frac{M2}{Wdeck} \cdot 10^{-3} \le ULS1, 3\right)$;
$Check3 := evalf\left(\frac{M3}{Wdeck} \cdot 10^{-3} \le ULS2, 3\right)$; $Check4 := evalf\left(\frac{M4}{Wdeck} \cdot 10^{-3} \le ULS2, 3\right)$;
$Check5 := evalf\left(\frac{M5}{Wdeck} \cdot 10^{-3} \le ULS2, 3\right)$;

$$Check1 := 1.02 \le 7.40$$
$$Check2 := 3.01 \le 7.40$$
$$Check3 := 1.82 \le 11.1$$
$$Check4 := 1.88 \le 11.1$$
$$Check5 := 3.96 \le 11.1 \tag{13}$$

Serviceability lime state according to EC5 art. 7.2

> $Ecreep := \frac{E0mean}{(1 + kdefdeck)}$;

$$Ecreep := 3900.000000 \tag{14}$$

> $Winst1 := evalf\left(\frac{5}{384} \cdot \frac{LC8 \cdot hoh^4}{E0mean \cdot Iydeck} - \frac{M8 \cdot hoh^2}{\sqrt{243} \cdot E0mean \cdot Iydeck}, 3\right)$; $Winst2 := evalf\left(\frac{5}{384} \cdot \frac{LC9 \cdot hoh^4}{E0mean \cdot Iydeck} - \frac{M9 \cdot hoh^2}{\sqrt{243} \cdot E0mean \cdot Iydeck}, 3\right)$; $Winst3 := evalf\left(\frac{5}{384} \cdot \frac{LC10 \cdot hoh^4}{E0mean \cdot Iydeck} - \frac{M10 \cdot hoh^2}{\sqrt{243} \cdot E0mean \cdot Iydeck}, 3\right)$; $Wfin1 := evalf\left(\frac{5}{384} \cdot \frac{LC6 \cdot hoh^4}{Ecreep \cdot Iydeck} + \frac{5}{384} \cdot \frac{(LC8 - LC6) \cdot hoh^4}{E0mean \cdot Iydeck}, 3\right)$; $Wfin2 := evalf\left(\frac{5}{384} \cdot \frac{LC6 \cdot hoh^4}{Ecreep \cdot Iydeck} + \frac{5}{384} \cdot \frac{(LC9 - LC6) \cdot hoh^4}{E0mean \cdot Iydeck}, 3\right)$; $Wfin3 := evalf\left(\frac{5}{384} \cdot \frac{LC7 \cdot hoh^4}{Ecreep \cdot Iydeck} + \frac{5}{384} \cdot \frac{(LC10 - LC7) \cdot hoh^4}{E0mean \cdot Iydeck}, 3\right)$;

$$Winst1 := 0.241$$
$$Winst2 := 0.250$$
$$Winst3 := 0.550$$
$$Wfin1 := 0.978$$
$$Wfin2 := 0.990$$
$$Wfin3 := 2.43 \tag{15}$$

> $Wallowed := \frac{hoh \cdot 10^3}{250}$;

$$Wallowed := 2.400 \tag{16}$$

NEN8700 - Reduction of reference period

Factors NEN8700

> $gammag1 := 1.2 : gammag2 := 1.15 : gammaq := 1.3 :$

Material properties C18

> $fmk := 18 : E0mean := 9000 :$

Load

> $\mathrm{permanent} := 0.6 \cdot \mathrm{hoh} : \mathrm{udl} := 1 \cdot \mathrm{hoh} : \mathrm{GRext} := 1 \cdot \mathrm{hoh} : \mathrm{GRint1} := 1 \cdot \mathrm{hoh} : \mathrm{GRint2} := 3.4 \cdot \mathrm{hoh} : \mathrm{GRuse1} := 0.60 \cdot 1.75 \cdot \mathrm{hoh} : \mathrm{GRuse2} := 0.80 \cdot 1.75 \cdot \mathrm{hoh} :$

Load combination

> $LC1 := gammag1 \cdot (permanent + GRext); LC2 := gammag1 \cdot (permanent + GRint2); LC3 := gammag2 \cdot (permanent + GRext) + gammaq \cdot udl; LC4 := gammag2 \cdot (permanent + GRint1) + gammaq \cdot GRuse1; LC5 := gammag2 \cdot (permanent + GRint2) + gammaq \cdot GRuse2;$

$$LC1 := 1.15200$$
$$LC2 := 2.88000$$
$$LC3 := 1.884000$$
$$LC4 := 1.92300000$$
$$LC5 := 3.85200000 \tag{17}$$

Moment

> $M1 := \left(\frac{1}{8}\right) \cdot LC1 \cdot l^2; M2 := \left(\frac{1}{8}\right) \cdot LC2 \cdot l^2; M3 := \left(\frac{1}{8}\right) \cdot LC3 \cdot l^2; M4 := \left(\frac{1}{8}\right) \cdot LC4 \cdot l^2; M5 := \left(\frac{1}{8}\right) \cdot LC5 \cdot l^2;$

$$M1 := 2.420640000$$
$$M2 := 6.051600000$$
$$M3 := 3.958755000$$
$$M4 := 4.040703750$$
$$M5 := 8.094015000 \tag{18}$$

Ultimate limit state according to EC5 art. 6.1.6

> $ULS1 := \frac{kmod1 \cdot fmk}{gammam}; ULS2 := \frac{kmod2 \cdot fmk}{gammam};$

$$ULS1 := 8.307692308$$
$$ULS2 := 12.46153846 \tag{19}$$

> $Check1 := evalf\left(\frac{M1}{W} \cdot 10^{-3} \leq ULS1, 3\right); Check2 := evalf\left(\frac{M2}{W} \cdot 10^{-3} \leq ULS1, 3\right); Check3 := evalf\left(\frac{M3}{W} \cdot 10^{-3} \leq ULS2, 3\right); Check4 := evalf\left(\frac{M4}{W} \cdot 10^{-3} \leq ULS2, 3\right); Check5 := evalf\left(\frac{M5}{W} \cdot 10^{-3} \leq ULS2, 3\right);$

$$Check1 := 5.09 \leq 8.31$$
$$Check2 := 12.7 \leq 8.31$$
$$Check3 := 8.34 \leq 12.5$$
$$Check4 := 8.51 \leq 12.5$$
$$Check5 := 17.0 \leq 12.5 \tag{20}$$

NEN8700 - Adjust characteristic values 1

Factors Eurocode

> $gammag1 := 1.35 : gammag2 := 1.2 : gammaq := 1.5 : kmod1 := 0.60 : kmod2 := 0.90 : gammasls := 1 : gammam := 1.3 : kdef := 0.80 :$

Updated material properties through visual grading

> $fmk1 := 18 : E0mean1 := 9000 : fmk2 := 24 : E0mean2 := 11000 :$

Load

> $\mathrm{permanent} := 0.6 \cdot \mathrm{hoh} : \mathrm{udl} := 1 \cdot \mathrm{hoh} : \mathrm{GRext} := 1 \cdot \mathrm{hoh} : \mathrm{GRint1} := 1 \cdot \mathrm{hoh} : \mathrm{GRint2} := 3.4 \cdot \mathrm{hoh} : \mathrm{GRuse1} := 0.60 \cdot 1.75 \cdot \mathrm{hoh} : \mathrm{GRuse2} := 0.80 \cdot 1.75 \cdot \mathrm{hoh} :$

Load combination

> $LC1 := gammag1 \cdot (permanent + GRext); LC2 := gammag1 \cdot (permanent + GRint2); LC3 := gammag2 \cdot (permanent + GRext) + gammaq \cdot udl; LC4 := gammag2 \cdot (permanent + GRint1) + gammaq \cdot GRuse1; LC5 := gammag2 \cdot (permanent + GRint2) + gammaq \cdot GRuse2; LC6 := gammasls \cdot (permanent + GRext); LC7 := gammasls \cdot (permanent + GRint2); LC8 := gammasls \cdot (permanent + GRext) + gammasls \cdot udl; LC9 := gammasls \cdot (permanent + GRint1) + gammasls \cdot GRuse1; LC10 := gammasls \cdot (permanent + GRint2) + gammasls \cdot GRuse2;$

$$LC1 := 1.296000$$
$$LC2 := 3.240000$$
$$LC3 := 2.05200$$
$$LC4 := 2.09700000$$
$$LC5 := 4.14000000$$
$$LC6 := 0.9600$$
$$LC7 := 2.4000$$
$$LC8 := 1.5600$$
$$LC9 := 1.5900000$$
$$LC10 := 3.2400000 \tag{21}$$

Moment

> $M1 := \left(\frac{1}{8}\right) \cdot LC1 \cdot l^2; M2 := \left(\frac{1}{8}\right) \cdot LC2 \cdot l^2; M3 := \left(\frac{1}{8}\right) \cdot LC3 \cdot l^2; M4 := \left(\frac{1}{8}\right) \cdot LC4 \cdot l^2; M5 := \left(\frac{1}{8}\right) \cdot LC5 \cdot l^2;$

$$M1 := 2.723220000$$
$$M2 := 6.808050000$$
$$M3 := 4.311765000$$
$$M4 := 4.406321250$$
$$M5 := 8.699175000 \tag{22}$$

Ultimate limit state according to EC5 art. 6.1.6

> $ULS1 := \frac{kmod1 \cdot fmk2}{gammam}; ULS2 := \frac{kmod2 \cdot fmk2}{gammam};$

$$ULS1 := 11.07692308$$
$$ULS2 := 16.61538462 \tag{23}$$

> $Check1 := evalf\left(\frac{M1}{W} \cdot 10^{-3} \leq ULS1, 3\right); Check2 := evalf\left(\frac{M2}{W} \cdot 10^{-3} \leq ULS1, 3\right); Check3 := evalf\left(\frac{M3}{W} \cdot 10^{-3} \leq ULS2, 3\right); Check4 := evalf\left(\frac{M4}{W} \cdot 10^{-3} \leq ULS2, 3\right); Check5$

$$:= evalf\left(\frac{M5}{W}\cdot 10^{-3} \le ULS2, 3\right);$$

$$Check1 := 5.73 \le 11.1$$
$$Check2 := 14.3 \le 11.1$$
$$Check3 := 9.07 \le 16.6$$
$$Check4 := 9.28 \le 16.6$$
$$Check5 := 18.3 \le 16.6 \qquad \textbf{(24)}$$

Serviceability lime state according to EC5 art. 7.2

$$> Ecreep1 := \frac{E0mean1}{(1 + kdef)}; Ecreep2 := \frac{E0mean2}{(1 + kdef)};$$

$$Ecreep1 := 5000.000000$$
$$Ecreep2 := 6111.111111 \qquad \textbf{(25)}$$

$$> Winst1inhom := evalf\left(\frac{5}{384}\,\frac{LC8\cdot l^4}{E0mean2\cdot Iy} - \frac{1}{216}\,\frac{LC8\cdot l^4\cdot Iy\cdot (E0mean1 - E0mean2)}{Iy\cdot (E0mean1\cdot E0mean2)}, 3\right);$$

$$Winst2inhom := evalf\left(\frac{5}{384}\,\frac{LC9\cdot l^4}{E0mean2\cdot Iy} - \frac{1}{216}\,\frac{LC9\cdot l^4\cdot Iy\cdot (E0mean1 - E0mean2)}{Iy\cdot (E0mean1\cdot E0mean2)}, 3\right); Winst1inhom := evalf\left(\frac{5}{384}\,\frac{LC10\cdot l^4}{E0mean2\cdot Iy} - \frac{1}{216}\,\frac{LC10\cdot l^4\cdot Iy\cdot (E0mean1 - E0mean2)}{Iy\cdot (E0mean1\cdot E0mean2)}, 3\right);$$

$$Wfin1 := evalf\left(\frac{5}{384}\cdot\frac{LC6\cdot l^4}{Ecreep2\cdot Iy} - \frac{1}{216}\,\frac{LC6\cdot l^4\cdot Iy\cdot (Ecreep1 - Ecreep2)}{Iy\cdot (Ecreep1\cdot Ecreep2)} + \frac{5}{384}\cdot\frac{(LC8 - LC6)\cdot l^4}{E0mean2\cdot Iy} - \frac{1}{216}\,\frac{(LC8 - LC6)\cdot l^4\cdot Iy\cdot (E0mean1 - E0mean2)}{Iy\cdot (E0mean1\cdot E0mean2)}, 3\right);$$

$$Wfin2 := evalf\left(\frac{5}{384}\cdot\frac{LC6\cdot l^4}{Ecreep2\cdot Iy} - \frac{1}{216}\,\frac{LC6\cdot l^4\cdot Iy\cdot (Ecreep1 - Ecreep2)}{Iy\cdot (Ecreep1\cdot Ecreep2)} + \frac{5}{384}\cdot\frac{(LC9 - LC6)\cdot l^4}{E0mean2\cdot Iy} - \frac{1}{216}\,\frac{(LC9 - LC6)\cdot l^4\cdot Iy\cdot (E0mean1 - E0mean2)}{Iy\cdot (E0mean1\cdot E0mean2)}, 3\right);$$

$$Wfin3 := evalf\left(\frac{5}{384}\cdot\frac{LC6\cdot l^4}{Ecreep2\cdot Iy} - \frac{1}{216}\,\frac{LC6\cdot l^4\cdot Iy\cdot (Ecreep1 - Ecreep2)}{Iy\cdot (Ecreep1\cdot Ecreep2)} + \frac{5}{384}\cdot\frac{(LC10 - LC6)\cdot l^4}{E0mean2\cdot Iy} - \frac{1}{216}\,\frac{(LC10 - LC6)\cdot l^4\cdot Iy\cdot (E0mean1 - E0mean2)}{Iy\cdot (E0mean1\cdot E0mean2)}, 3\right);$$

$$Winst1inhom := 11.3$$
$$Winst2inhom := 11.5$$
$$Winst1inhom := 23.4$$
$$Wfin1 := 16.9$$
$$Wfin2 := 17.1$$
$$Wfin3 := 29.0 \qquad \textbf{(26)}$$

$$> Wallowed := \frac{l\cdot 10^3}{250};$$

$$Wallowed := 16.400 \qquad \textbf{(27)}$$

NEN8700 - Adjust characteristic values 2

Factors Eurocode

> $gammag1 := 1.35 : gammag2 := 1.2 : gammaq := 1.5 : kmod1 := 0.60 : kmod2 := 0.90 : gammasls := 1 : gammam := 1.3 : kdef := 0.80 :$

Updated material properties through non-destructive testing

> $fmk := 27 : E0mean := 11500 :$

Load

> permanent := 0.6·hoh : udl := 1·hoh : GRext := 1·hoh : GRint1 := 1·hoh : GRint2 := 3.4·hoh : GRuse1 := 0.60·1.75·hoh : GRuse2 := 0.80·1.75·hoh :

Load combination

> $LC1 := gammag1\cdot(permanent + GRext);\ LC2 := gammag1\cdot(permanent + GRint2);\ LC3 := gammag2\cdot(permanent + GRext) + gammaq\cdot udl;\ LC4 := gammag2\cdot(permanent + GRint1) + gammaq\cdot GRuse1;\ LC5 := gammag2\cdot(permanent + GRint2) + gammaq\cdot GRuse2;\ LC6 := gammasls\cdot(permanent + GRext);\ LC7 := gammasls\cdot(permanent + GRint2);\ LC8 := gammasls\cdot(permanent + GRext) + gammasls\cdot udl;\ LC9 := gammasls\cdot(permanent + GRint1) + gammasls\cdot GRuse1;\ LC10 := gammasls\cdot(permanent + GRint2) + gammasls\cdot GRuse2;$

$$LC1 := 1.296000$$
$$LC2 := 3.240000$$
$$LC3 := 2.05200$$
$$LC4 := 2.09700000$$
$$LC5 := 4.14000000$$
$$LC6 := 0.9600$$
$$LC7 := 2.4000$$
$$LC8 := 1.5600$$
$$LC9 := 1.5900000$$
$$LC10 := 3.2400000 \tag{28}$$

Moment

> $M1 := \left(\frac{1}{8}\right)\cdot LC1\cdot l^2;\ M2 := \left(\frac{1}{8}\right)\cdot LC2\cdot l^2;\ M3 := \left(\frac{1}{8}\right)\cdot LC3\cdot l^2;\ M4 := \left(\frac{1}{8}\right)\cdot LC4\cdot l^2;\ M5 := \left(\frac{1}{8}\right)\cdot LC5\cdot l^2;$

$$M1 := 2.723220000$$
$$M2 := 6.808050000$$
$$M3 := 4.311765000$$
$$M4 := 4.406321250$$
$$M5 := 8.699175000 \tag{29}$$

Ultimate limit state according to EC5 art. 6.1.6

> $ULS1 := \frac{kmod1\cdot fmk}{gammam};\ ULS2 := \frac{kmod2\cdot fmk}{gammam};$

$$ULS1 := 12.46153846$$
$$ULS2 := 18.69230769 \tag{30}$$

> $Check1 := evalf\left(\frac{M1}{W}\cdot 10^{-3} \le ULS1, 3\right);\ Check2 := evalf\left(\frac{M2}{W}\cdot 10^{-3} \le ULS1, 3\right);\ Check3 := evalf\left(\frac{M3}{W}\cdot 10^{-3} \le ULS2, 3\right);\ Check4 := evalf\left(\frac{M4}{W}\cdot 10^{-3} \le ULS2, 3\right);\ Check5 := evalf\left(\frac{M5}{W}\cdot 10^{-3} \le ULS2, 3\right);$

$$Check1 := 5.73 \leq 12.5$$
$$Check2 := 14.3 \leq 12.5$$
$$Check3 := 9.07 \leq 18.7$$
$$Check4 := 9.28 \leq 18.7$$
$$Check5 := 18.3 \leq 18.7 \quad (31)$$

Serviceability lime state according to EC5 art. 7.2

$$> Ecreep := \frac{E0mean}{(1 + kdef)};$$

$$Ecreep := 6388.888889 \quad (32)$$

$$> Winst1 := evalf\left(\frac{5}{384} \cdot \frac{LC8 \cdot l^4}{E0mean \cdot Iy}, 3\right); Winst2 := evalf\left(\frac{5}{384} \cdot \frac{LC9 \cdot l^4}{E0mean \cdot Iy}, 3\right); Winst3 := evalf\left(\frac{5}{384} \cdot \frac{LC10 \cdot l^4}{E0mean \cdot Iy}, 3\right); Wfin1 := evalf\left(\frac{5}{384} \cdot \frac{LC6 \cdot l^4}{Ecreep \cdot Iy} + \frac{5}{384} \cdot \frac{(LC8 - LC6) \cdot l^4}{E0mean \cdot Iy}, 3\right); Wfin2 := evalf\left(\frac{5}{384} \cdot \frac{LC6 \cdot l^4}{Ecreep \cdot Iy} + \frac{5}{384} \cdot \frac{(LC9 - LC6) \cdot l^4}{E0mean \cdot Iy}, 3\right); Wfin3 := evalf\left(\frac{5}{384} \cdot \frac{LC7 \cdot l^4}{Ecreep \cdot Iy} + \frac{5}{384} \cdot \frac{(LC10 - LC7) \cdot l^4}{E0mean \cdot Iy}, 3\right);$$

$$Winst1 := 10.8$$
$$Winst2 := 11.0$$
$$Winst3 := 22.4$$
$$Wfin1 := 16.2$$
$$Wfin2 := 16.4$$
$$Wfin3 := 35.6 \quad (33)$$

$$> Wallowed := \frac{l \cdot 10^3}{250};$$

$$Wallowed := 16.400 \quad (34)$$

Geometrie Kerkhoflaan

> $hoh := 0.500 : l := 4.600 : b := 0.090 : h := 0.240 : W := \left(\frac{1}{6}\right) \cdot b \cdot h^2 : Iy := \left(\frac{1}{12}\right) \cdot b \cdot h^3 :$

Eurocode

Factors Eurocode

> $gammag1 := 1.35 : gammag2 := 1.2 : gammaq := 1.5 : kmod1 := 0.60 : kmod2 := 0.90 : gammasls := 1 : gammam := 1.3 : kdef := 0.80 :$

Material properties C18

> $fmk := 24 : E0mean := 11000 :$

Load

> $\mathrm{permanent} := 1.40 \cdot \mathrm{hoh} : \mathrm{udl} := 1 \cdot \mathrm{hoh} : \mathrm{GRext} := 1 \cdot \mathrm{hoh} : \mathrm{GRint1} := 1 \cdot \mathrm{hoh} : \mathrm{GRint2} := 3.4 \cdot \mathrm{hoh} : \mathrm{GRuse1} := 0.60 \cdot 1.75 \cdot \mathrm{hoh} : \mathrm{GRuse2} := 0.80 \cdot 1.75 \cdot \mathrm{hoh} :$

Load combination

> $LC1 := gammag1 \cdot (permanent + GRext); LC2 := gammag1 \cdot (permanent + GRint2); LC3 := gammag2 \cdot (permanent + GRext) + gammaq \cdot udl; LC4 := gammag2 \cdot (permanent + GRint1) + gammaq \cdot GRuse1; LC5 := gammag2 \cdot (permanent + GRint2) + gammaq \cdot GRuse2; LC6 := gammasls \cdot (permanent + GRext); LC7 := gammasls \cdot (permanent + GRint2); LC8 := gammasls \cdot (permanent + GRext) + gammasls \cdot udl; LC9 := gammasls \cdot (permanent + GRint1) + gammasls \cdot GRuse1; LC10 := gammasls \cdot (permanent + GRint2) + gammasls \cdot GRuse2;$

$$LC1 := 1.6200000$$
$$LC2 := 3.2400000$$
$$LC3 := 2.190000$$
$$LC4 := 2.22750000$$
$$LC5 := 3.93000000$$
$$LC6 := 1.20000$$
$$LC7 := 2.40000$$
$$LC8 := 1.70000$$
$$LC9 := 1.7250000$$
$$LC10 := 3.1000000 \qquad (1)$$

Moment

> $M1 := \left(\frac{1}{8}\right) \cdot LC1 \cdot l^2; M2 := \left(\frac{1}{8}\right) \cdot LC2 \cdot l^2; M3 := \left(\frac{1}{8}\right) \cdot LC3 \cdot l^2; M4 := \left(\frac{1}{8}\right) \cdot LC4 \cdot l^2; M5 := \left(\frac{1}{8}\right) \cdot LC5 \cdot l^2;$

$$M1 := 4.284900000$$
$$M2 := 8.569800000$$
$$M3 := 5.792550000$$
$$M4 := 5.891737500$$
$$M5 := 10.39485000 \qquad (2)$$

Ultimate limit state according to EC5 art. 6.1.6

> $ULS1 := \frac{kmod1 \cdot fmk}{gammam}; ULS2 := \frac{kmod2 \cdot fmk}{gammam};$

$$ULS1 := 11.07692308$$
$$ULS2 := 16.61538462 \qquad (3)$$

$> Check1 := evalf\left(\frac{M1}{W}\cdot 10^{-3} \le ULS1, 3\right); Check2 := evalf\left(\frac{M2}{W}\cdot 10^{-3} \le ULS1, 3\right); Check3 := evalf\left(\frac{M3}{W}\cdot 10^{-3} \le ULS2, 3\right); Check4 := evalf\left(\frac{M4}{W}\cdot 10^{-3} \le ULS2, 3\right); Check5 := evalf\left(\frac{M5}{W}\cdot 10^{-3} \le ULS2, 3\right);$

$$Check1 := 4.95 \le 11.1$$
$$Check2 := 9.92 \le 11.1$$
$$Check3 := 6.70 \le 16.6$$
$$Check4 := 6.82 \le 16.6$$
$$Check5 := 12.0 \le 16.6 \qquad (4)$$

Serviceability lime state according to EC5 art. 7.2

$> Ecreep := \frac{E0mean}{(1+kdef)};$

$$Ecreep := 6111.111111 \qquad (5)$$

$> Winst1 := evalf\left(\frac{5}{384}\cdot\frac{LC8\cdot l^4}{E0mean\cdot Iy}, 3\right); Winst2 := evalf\left(\frac{5}{384}\cdot\frac{LC9\cdot l^4}{E0mean\cdot Iy}, 3\right); Winst3 := evalf\left(\frac{5}{384}\cdot\frac{LC10\cdot l^4}{E0mean\cdot Iy}, 3\right); Wfin1 := evalf\left(\frac{5}{384}\cdot\frac{LC6\cdot l^4}{Ecreep\cdot Iy} + \frac{5}{384}\cdot\frac{(LC8-LC6)\cdot l^4}{E0mean\cdot Iy}, 3\right); Wfin2 := evalf\left(\frac{5}{384}\cdot\frac{LC6\cdot l^4}{Ecreep\cdot Iy} + \frac{5}{384}\cdot\frac{(LC9-LC6)\cdot l^4}{E0mean\cdot Iy}, 3\right); Wfin3 := evalf\left(\frac{5}{384}\cdot\frac{LC7\cdot l^4}{Ecreep\cdot Iy} + \frac{5}{384}\cdot\frac{(LC10-LC7)\cdot l^4}{E0mean\cdot Iy}, 3\right);$

$$Winst1 := 8.67$$
$$Winst2 := 8.78$$
$$Winst3 := 15.8$$
$$Wfin1 := 13.6$$
$$Wfin2 := 13.7$$
$$Wfin3 := 25.7 \qquad (6)$$

$> Wallowed := \frac{l\cdot 10^3}{250};$

$$Wallowed := 18.400 \qquad (7)$$

NEN8700 - Reduction of reference period

Factors NEN8700

$> gammag1 := 1.2 : gammag2 := 1.15 : gammaq := 1.3 :$

Material properties C18

$> fmk := 24 : E0mean := 11000 :$

Load

$>$ permanent := 1.4·hoh : udl := 1·hoh : GRext := 1·hoh : GRint1 := 1·hoh : GRint2 := 3.4·hoh : GRuse1 := 0.60·1.75·hoh : GRuse2 := 0.80·1.75·hoh :

Load combination

> $LC1 := gammag1\cdot(permanent + GRext);\ LC2 := gammag1\cdot(permanent + GRint2);\ LC3 := gammag2\cdot(permanent + GRext) + gammaq\cdot udl;\ LC4 := gammag2\cdot(permanent + GRint1) + gammaq\cdot GRuse1;\ LC5 := gammag2\cdot(permanent + GRint2) + gammaq\cdot GRuse2;$

$$LC1 := 1.44000$$
$$LC2 := 2.88000$$
$$LC3 := 2.030000$$
$$LC4 := 2.06250000$$
$$LC5 := 3.67000000 \quad \textbf{(8)}$$

Moment

> $M1 := \left(\frac{1}{8}\right)\cdot LC1\cdot l^2;\ M2 := \left(\frac{1}{8}\right)\cdot LC2\cdot l^2;\ M3 := \left(\frac{1}{8}\right)\cdot LC3\cdot l^2;\ M4 := \left(\frac{1}{8}\right)\cdot LC4\cdot l^2;\ M5 := \left(\frac{1}{8}\right)\cdot LC5\cdot l^2;$

$$M1 := 3.808800000$$
$$M2 := 7.617600000$$
$$M3 := 5.369350000$$
$$M4 := 5.455312500$$
$$M5 := 9.707150000 \quad \textbf{(9)}$$

Ultimate limit state according to EC5 art. 6.1.6

> $ULS1 := \frac{kmod1\cdot fmk}{gammam};\ ULS2 := \frac{kmod2\cdot fmk}{gammam};$

$$ULS1 := 11.07692308$$
$$ULS2 := 16.61538462 \quad \textbf{(10)}$$

> $Check1 := evalf\left(\frac{M1}{W}\cdot 10^{-3} \le ULS1, 3\right);\ Check2 := evalf\left(\frac{M2}{W}\cdot 10^{-3} \le ULS1, 3\right);\ Check3 := evalf\left(\frac{M3}{W}\cdot 10^{-3} \le ULS2, 3\right);\ Check4 := evalf\left(\frac{M4}{W}\cdot 10^{-3} \le ULS2, 3\right);\ Check5 := evalf\left(\frac{M5}{W}\cdot 10^{-3} \le ULS2, 3\right);$

$$Check1 := 4.41 \le 11.1$$
$$Check2 := 8.82 \le 11.1$$
$$Check3 := 6.22 \le 16.6$$
$$Check4 := 6.32 \le 16.6$$
$$Check5 := 11.2 \le 16.6 \quad \textbf{(11)}$$

NEN8700 - Adjust characteristic values 1

Factors Eurocode

> $gammag1 := 1.35 : gammag2 := 1.2 : gammaq := 1.5 : kmod1 := 0.60 : kmod2 := 0.90 : gammasls := 1 : gammam := 1.3 : kdef := 0.80 :$

Updated material properties through visual grading

> $fmk1 := 24 : E0mean1 := 11000 : fmk2 := 30 : E0mean2 := 12000 :$

Load

> $\text{permanent} := 1.4\cdot\text{hoh} : \text{udl} := 1\cdot\text{hoh} : \text{GRext} := 1\cdot\text{hoh} : \text{GRint1} := 1\cdot\text{hoh} : \text{GRint2} := 3.4\cdot\text{hoh} : \text{GRuse1} := 0.60\cdot 1.75\cdot\text{hoh} : \text{GRuse2} := 0.80\cdot 1.75\cdot\text{hoh} :$

Load combination

> $LC1 := gammag1 \cdot (permanent + GRext); LC2 := gammag1 \cdot (permanent + GRint2); LC3 := gammag2 \cdot (permanent + GRext) + gammaq \cdot udl; LC4 := gammag2 \cdot (permanent + GRint1) + gammaq \cdot GRuse1; LC5 := gammag2 \cdot (permanent + GRint2) + gammaq \cdot GRuse2; LC6 := gammasls \cdot (permanent + GRext); LC7 := gammasls \cdot (permanent + GRint2); LC8 := gammasls \cdot (permanent + GRext) + gammasls \cdot udl; LC9 := gammasls \cdot (permanent + GRint1) + gammasls \cdot GRuse1; LC10 := gammasls \cdot (permanent + GRint2) + gammasls \cdot GRuse2;$

$$LC1 := 1.620000$$
$$LC2 := 3.240000$$
$$LC3 := 2.19000$$
$$LC4 := 2.22750000$$
$$LC5 := 3.93000000$$
$$LC6 := 1.2000$$
$$LC7 := 2.4000$$
$$LC8 := 1.7000$$
$$LC9 := 1.7250000$$
$$LC10 := 3.1000000 \quad (12)$$

Moment

> $M1 := \left(\frac{1}{8}\right) \cdot LC1 \cdot l^2; M2 := \left(\frac{1}{8}\right) \cdot LC2 \cdot l^2; M3 := \left(\frac{1}{8}\right) \cdot LC3 \cdot l^2; M4 := \left(\frac{1}{8}\right) \cdot LC4 \cdot l^2; M5 := \left(\frac{1}{8}\right) \cdot LC5 \cdot l^2;$

$$M1 := 4.284900000$$
$$M2 := 8.569800000$$
$$M3 := 5.792550000$$
$$M4 := 5.891737500$$
$$M5 := 10.39485000 \quad (13)$$

Ultimate limit state according to EC5 art. 6.1.6

> $ULS1 := \frac{kmod1 \cdot fmk2}{gammam}; ULS2 := \frac{kmod2 \cdot fmk2}{gammam};$

$$ULS1 := 13.84615385$$
$$ULS2 := 20.76923077 \quad (14)$$

> $Check1 := evalf\left(\frac{M1}{W} \cdot 10^{-3} \le ULS1, 3\right); Check2 := evalf\left(\frac{M2}{W} \cdot 10^{-3} \le ULS1, 3\right); Check3 := evalf\left(\frac{M3}{W} \cdot 10^{-3} \le ULS2, 3\right); Check4 := evalf\left(\frac{M4}{W} \cdot 10^{-3} \le ULS2, 3\right); Check5 := evalf\left(\frac{M5}{W} \cdot 10^{-3} \le ULS2, 3\right);$

$$Check1 := 4.95 \le 13.8$$
$$Check2 := 9.92 \le 13.8$$
$$Check3 := 6.70 \le 20.8$$
$$Check4 := 6.82 \le 20.8$$
$$Check5 := 12.0 \le 20.8 \quad (15)$$

Serviceability lime state according to EC5 art. 7.2

$$> \; Ecreep1 := \frac{E0mean1}{(1+kdef)};\; Ecreep2 := \frac{E0mean2}{(1+kdef)};$$

$$Ecreep1 := 6111.111111$$
$$Ecreep2 := 6666.666667 \tag{16}$$

$$> \; Winst1inhom := evalf\left(\frac{5}{384}\,\frac{LC8\cdot l^4}{E0mean2\cdot Iy} - \frac{1}{216}\,\frac{LC8\cdot l^4\cdot Iy\cdot(E0mean1-E0mean2)}{Iy\cdot(E0mean1\cdot E0mean2)}, 3\right);$$
$$Winst2inhom := evalf\left(\frac{5}{384}\,\frac{LC9\cdot l^4}{E0mean2\cdot Iy} - \frac{1}{216}\,\frac{LC9\cdot l^4\cdot Iy\cdot(E0mean1-E0mean2)}{Iy\cdot(E0mean1\cdot E0mean2)}, 3\right);$$
$$Winst1inhom := evalf\left(\frac{5}{384}\,\frac{LC10\cdot l^4}{E0mean2\cdot Iy} - \frac{1}{216}\,\frac{LC10\cdot l^4\cdot Iy\cdot(E0mean1-E0mean2)}{Iy\cdot(E0mean1\cdot E0mean2)}, 3\right);$$
$$Wfin1 := evalf\left(\frac{5}{384}\cdot\frac{LC6\cdot l^4}{Ecreep2\cdot Iy} - \frac{1}{216}\,\frac{LC6\cdot l^4\cdot Iy\cdot(Ecreep1-Ecreep2)}{Iy\cdot(Ecreep1\cdot Ecreep2)} + \frac{5}{384}\cdot\frac{(LC8-LC6)\cdot l^4}{E0mean2\cdot Iy} - \frac{1}{216}\,\frac{(LC8-LC6)\cdot l^4\cdot Iy\cdot(E0mean1-E0mean2)}{Iy\cdot(E0mean1\cdot E0mean2)}, 3\right);$$
$$Wfin2 := evalf\left(\frac{5}{384}\cdot\frac{LC6\cdot l^4}{Ecreep2\cdot Iy} - \frac{1}{216}\,\frac{LC6\cdot l^4\cdot Iy\cdot(Ecreep1-Ecreep2)}{Iy\cdot(Ecreep1\cdot Ecreep2)} + \frac{5}{384}\cdot\frac{(LC9-LC6)\cdot l^4}{E0mean2\cdot Iy} - \frac{1}{216}\,\frac{(LC9-LC6)\cdot l^4\cdot Iy\cdot(E0mean1-E0mean2)}{Iy\cdot(E0mean1\cdot E0mean2)}, 3\right);$$
$$Wfin3 := evalf\left(\frac{5}{384}\cdot\frac{LC6\cdot l^4}{Ecreep2\cdot Iy} - \frac{1}{216}\,\frac{LC6\cdot l^4\cdot Iy\cdot(Ecreep1-Ecreep2)}{Iy\cdot(Ecreep1\cdot Ecreep2)} + \frac{5}{384}\cdot\frac{(LC10-LC6)\cdot l^4}{E0mean2\cdot Iy} - \frac{1}{216}\,\frac{(LC10-LC6)\cdot l^4\cdot Iy\cdot(E0mean1-E0mean2)}{Iy\cdot(E0mean1\cdot E0mean2)}, 3\right);$$

$$Winst1inhom := 7.96$$
$$Winst2inhom := 8.03$$
$$Winst1inhom := 14.6$$
$$Wfin1 := 12.4$$
$$Wfin2 := 12.5$$
$$Wfin3 := 19.0 \tag{17}$$

$$> \; Wallowed := \frac{l\cdot 10^3}{250};$$

$$Wallowed := 18.400 \tag{18}$$

NEN8700 - Adjust characteristic values 2

Factors Eurocode

$> \; gammag1 := 1.35 : gammag2 := 1.2 : gammaq := 1.5 : kmod1 := 0.60 : kmod2 := 0.90 : gammasls := 1 : gammam := 1.3 : kdef := 0.80 :$

Updated material properties through non-destructive testing

$> \; fmk := 30 : E0mean := 12000 :$

Load

$> \; \mathrm{permanent} := 1.4\cdot\mathrm{hoh} : \mathrm{udl} := 1\cdot\mathrm{hoh} : \mathrm{GRext} := 1\cdot\mathrm{hoh} : \mathrm{GRint1} := 1\cdot\mathrm{hoh} : \mathrm{GRint2} := 3.4\cdot\mathrm{hoh} : \mathrm{GRuse1} := 0.60\cdot 1.75\cdot\mathrm{hoh} : \mathrm{GRuse2} := 0.80\cdot 1.75\cdot\mathrm{hoh} :$

Load combination

$> \; LC1 := gammag1\cdot(permanent + GRext);\; LC2 := gammag1\cdot(permanent + GRint2);\; LC3$

$:= gammag2 \cdot (permanent + GRext) + gammaq \cdot udl, LC4 := gammag2 \cdot (permanent + GRint1) + gammaq \cdot GRuse1; LC5 := gammag2 \cdot (permanent + GRint2) + gammaq \cdot GRuse2; LC6 := gammasls \cdot (permanent + GRext); LC7 := gammasls \cdot (permanent + GRint2); LC8 := gammasls \cdot (permanent + GRext) + gammasls \cdot udl, LC9 := gammasls \cdot (permanent + GRint1) + gammasls \cdot GRuse1; LC10 := gammasls \cdot (permanent + GRint2) + gammasls \cdot GRuse2;$

$$LC1 := 1.620000$$
$$LC2 := 3.240000$$
$$LC3 := 2.19000$$
$$LC4 := 2.22750000$$
$$LC5 := 3.93000000$$
$$LC6 := 1.2000$$
$$LC7 := 2.4000$$
$$LC8 := 1.7000$$
$$LC9 := 1.7250000$$
$$LC10 := 3.1000000 \quad \textbf{(19)}$$

Moment

$> M1 := \left(\frac{1}{8}\right) \cdot LC1 \cdot l^2; M2 := \left(\frac{1}{8}\right) \cdot LC2 \cdot l^2; M3 := \left(\frac{1}{8}\right) \cdot LC3 \cdot l^2; M4 := \left(\frac{1}{8}\right) \cdot LC4 \cdot l^2; M5 := \left(\frac{1}{8}\right) \cdot LC5 \cdot l^2;$

$$M1 := 4.284900000$$
$$M2 := 8.569800000$$
$$M3 := 5.792550000$$
$$M4 := 5.891737500$$
$$M5 := 10.39485000 \quad \textbf{(20)}$$

Ultimate limit state according to EC5 art. 6.1.6

$> ULS1 := \frac{kmod1 \cdot fmk}{gammam}; ULS2 := \frac{kmod2 \cdot fmk}{gammam};$

$$ULS1 := 12.46153846$$
$$ULS2 := 18.69230769 \quad \textbf{(21)}$$

$> Check1 := evalf\left(\frac{M1}{W} \cdot 10^{-3} \leq ULS1, 3\right); Check2 := evalf\left(\frac{M2}{W} \cdot 10^{-3} \leq ULS1, 3\right); Check3 := evalf\left(\frac{M3}{W} \cdot 10^{-3} \leq ULS2, 3\right); Check4 := evalf\left(\frac{M4}{W} \cdot 10^{-3} \leq ULS2, 3\right); Check5 := evalf\left(\frac{M5}{W} \cdot 10^{-3} \leq ULS2, 3\right);$

$$Check1 := 4.95 \leq 12.5$$
$$Check2 := 9.92 \leq 12.5$$
$$Check3 := 6.70 \leq 18.7$$
$$Check4 := 6.82 \leq 18.7$$
$$Check5 := 12.0 \leq 18.7 \quad \textbf{(22)}$$

Serviceability lime state according to EC5 art. 7.2

$> Ecreep := \frac{E0mean}{(1 + kdef)};$

$$Ecreep := 6388.888889 \qquad (23)$$

> $Winst1 := evalf\left(\frac{5}{384}\cdot\frac{LC8\cdot l^4}{E0mean\cdot Iy}, 3\right);$ $Winst2 := evalf\left(\frac{5}{384}\cdot\frac{LC9\cdot l^4}{E0mean\cdot Iy}, 3\right);$ $Winst3 := evalf\left(\frac{5}{384}\cdot\frac{LC10\cdot l^4}{E0mean\cdot Iy}, 3\right);$ $Wfin1 := evalf\left(\frac{5}{384}\cdot\frac{LC6\cdot l^4}{Ecreep\cdot Iy} + \frac{5}{384}\cdot\frac{(LC8 - LC6)\cdot l^4}{E0mean\cdot Iy}, 3\right);$ $Wfin2 := evalf\left(\frac{5}{384}\cdot\frac{LC6\cdot l^4}{Ecreep\cdot Iy} + \frac{5}{384}\cdot\frac{(LC9 - LC6)\cdot l^4}{E0mean\cdot Iy}, 3\right);$ $Wfin3 := evalf\left(\frac{5}{384}\cdot\frac{LC7\cdot l^4}{Ecreep\cdot Iy} + \frac{5}{384}\cdot\frac{(LC10 - LC7)\cdot l^4}{E0mean\cdot Iy}, 3\right);$

$$Winst1 := 8.31$$
$$Winst2 := 8.38$$
$$Winst3 := 15.1$$
$$Wfin1 := 13.0$$
$$Wfin2 := 13.0$$
$$Wfin3 := 24.5 \qquad (24)$$

> $Wallowed := \frac{l\cdot 10^3}{250};$

$$Wallowed := 18.400 \qquad (25)$$

H. REINFORCING TIMBER BEAMS

This appendix belongs to chapter 4 and shows different options of strengthening and the calculations of the case study.

Replacing (parts)			
Method	**Description**	**Suitable**	**Notes**
Full or partial replacing[1,5]	A new roof structure that is design on the extra load	x	The roof structure needs to be demolished

Additional structure (parts)			
Method	**Description**	**Suitable**	**Notes**
Adding extra beams[1,5]	The distance between the beams is reduced and thus the bearing capacity of one beam is increased	x	Damage to the wall but existing structure is untouched
Adding extra support[1]	The span will decrease	(x)	The extra support will rest on the floor beneath which doesn't anticipate the extra weight
Increasing support[1,5]	Additional construction to aid support		Doesn't increase bending resistant
Increasing cross section[1,5]	Adding timber parts to the surfaces of the existing member. Cooperation of old and new parts must be ensured	x	Easy method but aesthetics are lost.
Transverse reinforcement[1]	Local strength is increased due to cooperation between members. Examples: Andrew's cross or transverse brace		Only local reinforcement
Tie rods[1,5]	Steel cables or rods to contribute to the tension. Both the strength and stiffness can be increased.		Braced system requires much room below the beam

Composite systems			
Method	**Description**	**Suitable**	**Notes**
Timber-concrete[1,5]	A T-shaped beam/floor is formed where the concrete takes the compression force over an effective width by means of shear connectors	x	Stiffness and strength greatly increases but beams must be in good condition
Timber-timber[3,5]	Timber flanges take the compression or tension force over an effective width	x	Lighter than concrete and smaller structural improvement.
Timber-steel[3,5]	A strip on the bottom takes the tension force	x	Makes use of the plastic behaviour of wood
Bonding fibres[2]	Various methods using Fibre reinforced polymers (FRP) or natural fibres are possible. The most applicable is a (pre-stressed) FRP sheet bonded to the bottom	x	Control of the humidity is important

Inserting reinforcing elements			
Method	**Description**	**Suitable**	**Notes**
Glued bars[1,2,5]	Steel or fibreglass rods are glued horizontally in the tension zone		Damages the beam and increases possibility for crack initiation
Glued plates[1,2,5]	Steel plates are glued into vertical grooves along the beam that take up most of the load		Damage to the beam but steel is protected from fire and corrosion
Self-tapping screws[4]	Screws perpendicular to the grain take up tension forces so that splitting of the fibres is prevented		Is best suitable for resisting shear forces

1 Beoordeling en restauratie van historische (eiken) houten balklagen; van Reenen, M.; Master thesis TU Delft; 2003
2 Reinforcement of timber elements in existing structures; Tannert, T. & Branco, J.M. & Riggio, M.; RILEM; 2011
3 Flexural strengthening of timber beams by traditional and innovative techniques; Valuzzi, M.R. & Garbin, E. & Modena, C.; Journal of Building Appraisal vol.3 no.2 pp 125-143; 2007
4 Self-tapping screws as reinforcement for timber structures; Trautz, M. & KOJ, C.; Proceedings of the International Association for Shell and Spatial Structures (IASS) Symposium 2009, Valencia; 2009
5 Restoring timber structures - Repair and strengthening; Uzielli, L.; STEP 2 Timber Engineering, lecture D4, Centrum Hout, The Netherlands; 1995

Table G-1: Reinforcing methods

H.1 STRENGTHENING OPTIONS APPLIED ON CASE

Two strips

$$> E1 := 9000 : E2 := 11000 : b1 := 75 : b2 := 35 : h1 := 195 : h2 := 80 : d := 10 : I1 := \frac{1}{12} \cdot b1 \cdot h1^3 : W1 := \frac{1}{6} \cdot b1 \cdot h1^2 : I2 := \frac{1}{12} \cdot b2 \cdot h2^3 : M27 := 27 \cdot W1;$$

$$M27 := \frac{25666875}{2} \quad (1)$$

$$> yt := evalf\left(\frac{\left(\frac{b1 \cdot h1 \cdot h1}{2} + 2 \cdot b2 \cdot h2 \cdot \left(h1 - d - \frac{h2}{2} \right) \right)}{(b1 \cdot h1 + 2 \cdot b2 \cdot h2)}, 5 \right);$$

$$yt := 110.65 \quad (2)$$

$$> EIeff := E1 \cdot I1 + E1 \cdot b1 \cdot h1 \cdot \left(yt - \frac{h1}{2} \right)^2 + 2 \cdot \left(E2 \cdot I2 + E2 \cdot b2 \cdot h2 \cdot \left(h1 - yt - d - \frac{h2}{2} \right)^2 \right)$$

$$EIeff := 5.453842022\ 10^{11} \quad (3)$$

$$> Oldpart := \frac{M27 \cdot (h1 - yt) \cdot E1}{EIeff} < 18; Newparts := \frac{M27 \cdot (h1 - yt - d) \cdot E2}{EIeff} < 24;$$

$$Oldpart := 17.86356121 < 18$$

$$Newparts := 19.24483111 < 24 \quad (4)$$

Two triplex plates

$$> t := 19 : E := 9000 : W := \frac{1}{6} \cdot (b1 + 2 \cdot t) \cdot h1^2;$$

$$W := \frac{1432275}{2} \quad (5)$$

$$> evalf\left(\frac{M27}{W} \le 18, 3 \right)$$

$$17.9 \le 18. \quad (6)$$