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Bracing, modelling, full-scale testing and safety

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Slender roof structures
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by Anders Klasson

LUND UNIVERSITY

DOCTORAL DISSERTATION
Thesis advisors: Roberto Crocetti, Eva Frühwald Hansson, Ívar Björnsson and Oskar Larsson Ivanov
Faculty opponent: Associate Professor Ghasan Doudak
University of Ottawa

To be presented, with the permission of the Faculty of Engineering of Lund University, for public criticism in the MA3 lecture hall (Mathematics Annex building) at the Department of Building and Environmental Technology on Monday, the 10th of December 2018 at 9:00.

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Slender roof structures: Bracing, modelling, full-scale testing and safety

Abstract

The work presented in the thesis stems from the many roof failures that occurred due to heavy snow loading during the winters of 2009/10 and 2010/11 in Sweden. Similar events have occurred during other winters in other countries as well. Failure investigations indicate that many collapses are due to major design errors and not to the loading being extreme, i.e. exceeding the design snow loads.

The overall objective of the thesis was to increase awareness of the need to ensure adequate safety of slender roof structures and also to contribute to certain technical knowledge concerning the design of slender structural members.

The designing of slender structures typically involves uncertain parameters; engineers thus usually need to make subjective choices in modelling slender structures. In order to learn more regarding the effects of imperfections and of slip and slack, which are examples of uncertain parameters, on the bracing performance of slender structural members, numerical analyses involving such members were carried out. For example, it was shown that the performance of slender structural members is highly sensitive to structural imperfections. The imperfect shape of a structural member that is critical for the load-bearing capacity of the member itself is generally not the same imperfect shape that is critical for the forces that would be involved in the bracings of the member. Slip in bracing systems can reduce the load-bearing capacity of the braced member and also increase the bracing forces.

Full-scale laboratory testing was conducted in order to learn more regarding the effect on the bracing stiffness of a timber roof structure's different members; a special test rig that could be used to determine the point-wise bracing stiffness of the roof structure was developed. Both stabilization by means of diaphragm action and wind trusses in the plane of the roof were considered. Through use of finite element model updating approaches and the results of laboratory tests that were carried out, the stiffness values of connections, for example, could be estimated. It was found that the stiffness of bracing systems can be markedly overestimated if the connections are not accurately accounted for in the models employed. Also, the methods used in the laboratory testing can be used for the field-testing of roof structures, so as to verify that the structure is adequately stiff (i.e. that it meets design assumptions).

In order to learn more concerning important aspects of the design of slender structures, and to identify potential sources of errors in designing such structures, a survey of experienced structural engineers was conducted. The results of the survey indicate that many structural engineers believe that structural failures are commonly due to erroneous calculations, and they also indicate that improved communication between different partners in a building project would be useful for improving the overall safety of structures. In addition, the survey revealed that the designing work of experienced engineers varies significantly, in ways that have a potential for reducing structural safety. Thorough and independent review is seen as important for ensuring the adequate design of structures.

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by Anders Klasson

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A doctoral thesis at a university in Sweden takes either the form of a single, cohesive research study (monograph) or a summary of research papers (compilation thesis), which the doctoral student has written alone or together with one or more other author(s).

In the latter case the thesis consists of two parts. An introductory text puts the research work into context and summarizes the main points of the papers. Then, the research publications themselves are reproduced, together with a description of the individual contributions of the authors. The research papers may either have been already published or are manuscripts at various stages (in press, submitted, or in draft).

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Dedicated to Nina

Preface

The work presented in the thesis was conducted between 2013 and 2018 at the Division of Structural Engineering, Lund University.

I would like to express my gratitude to my supervisors, Prof. Roberto Crocetti, Docent Eva Frühwald Hansson, Dr. Ívar Björnsson and Dr. Oskar Larsson Ivanov for their guidance throughout the project. I would also like to thank Skanska Teknik and all the staff of the Division of Structural Engineering (especially Niklewski for all the interesting MatLab-exercises) for the joyful time I have had.

The work presented in the thesis would not have been possible to carry out without the financial support of both SBUF and Skanska, for which I am exceedingly grateful.

Finally, I would like to thank my friends and family for their endless support.

Anders Klasson
Lund 2018-10-15

Contents

List of publications

The thesis is based on the following publications, which will be referred to in text by their Roman numerals (the papers are appended at the end of the printed version of the thesis):

I **Slender steel columns: how they are affected by imperfections and bracing stiffness**
A. Klasson, R. Crocetti, E. Frühwald Hansson
Structures, 2016

II **Design for lateral stability of slender timber beams considering slip in the lateral bracing system**
A. Klasson, R. Crocetti, I. Björnsson, E. Frühwald Hansson
Structures, 2018

III **Slender roof structures: Failure reviews and a qualitative survey of experienced structural engineers**
A. Klasson, I. Björnsson, R. Crocetti, E. Frühwald Hansson
Structures, 2018

IV **The effects on the bracing stiffness of timber structures of the stiffness of its members**
A. Klasson, R. Crocetti
Submitted for publication in Structures, 2018

All the papers are reproduced with permission of their respective publishers.

Other publications not included in the thesis:

Bracing of slender steel and timber structures
A. Klasson
Report TVBK:1051. Division of Structural Engineering, 2015 (licentiate thesis)

Discrete bracing of timber beams subjected to gravity loads
A. Klasson, R. Crocetti, E. Frühwald Hansson
Presented at the World Conference on Timber Engineering 2014, August 10-14, Quebec City, Canada

Is advanced structural modeling always synonymous with increased accuracy?
I. Björnsson, R. Crocetti, M. Fröderberg, A. Klasson
Structural 208, 1-18 (2016)

Acknowledgements

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Popular summary in English

The work presented in the thesis stems from the many roof failures that occurred due to heavy snow loading during the winters of 2009/10 and 2010/11 in Sweden. Similar events have occurred during other winters in other countries as well. Failure investigations indicate that many collapses are due to major design errors and not to the loading being extreme, i.e. exceeding the design snow loads.

The overall objective of the thesis was to increase awareness of the need to ensure adequate safety of slender roof structures and also to contribute to certain technical knowledge concerning the design of slender structural members.

The designing of slender structures typically involves uncertain parameters; engineers thus usually need to make subjective choices in modelling slender structures. In order to learn more regarding the effects of imperfections and of slip and slack, which are examples of uncertain parameters, on the bracing performance of slender structural members, numerical analyses involving such members were carried out. For example, it was shown that the performance of slender structural members is highly sensitive to structural imperfections. The imperfect shape of a structural member that is critical for the load-bearing capacity of the member itself is generally not the same imperfect shape that is critical for the forces that would be involved in the bracings of the member. Slip in bracing systems can reduce the load-bearing capacity of the braced member and also increase the bracing forces.

Full-scale laboratory testing was conducted in order to learn more regarding the effect on the bracing stiffness of a timber roof structure's different members; a special test rig that could be used to determine the point-wise bracing stiffness of the roof structure was developed. Both stabilization by means of diaphragm action and wind trusses in the plane of the roof were considered. Through use of finite element model updating approaches and the results of laboratory tests that were carried out, the stiffness values of connections, for example, could be estimated. It was found that the stiffness of bracing systems can be markedly overestimated if the connections are not accurately accounted for in the models employed. Also, the methods used in the laboratory testing can be used for the field-testing of roof structures, so as to verify that the structure is adequately stiff (i.e. that it meets design assumptions).

In order to learn more concerning important aspects of the design of slender structures, and to identify potential sources of errors in designing such structures, a survey of experienced structural engineers was conducted. The results of the survey indicate that many structural engineers believe that structural failures are commonly due to erroneous calculations, and they also indicate that improved communication between different partners in a building project would be useful for improving the overall safety of structures. In addition, the survey revealed that the designing work of experienced engineers varies significantly, in ways that have a potential for reducing structural safety. Thorough and independent review is seen as important for ensuring the adequate design of structures.

Populärvetenskaplig sammanfattning på svenska

Arbetet med denna avhandling tar avstamp i de många takras som skedde under vintrarna 2009/10 och 2010/11 i Sverige. Liknande ras har dock skett både under andra vintrar i Sverige och i andra länder. Rasutredningar har fastslagit att många ras skett med anledning av grova konstruktionsfel, inte på grund av att snölasterna översteg de normföreskrivna lasterna som konstruktionerna borde ha varit dimensionerade för.

Det övergripande målet med avhandlingen var att medvetandegöra risken för brister hos slanka takkonstruktioner samt att även bidra med viss teknisk kunskap avseende dimensionering av slanka konstruktionselement.

Det finns ett antal osäkra parametetrar som måste beaktas vid dimensionering av slanka konstruktioner. Detta medför att konstruktören ofta måste göra subjektiva val för att kunna modellera en slank konstruktion. Med målet att bidra till ökad kunskap avseende imperfektioner hos konstruktionselement och glidning i stagningssystem, som är exempel på osäkra parameterar, har ett antal numeriska analyser av stagade konstruktionselement med olika imperfektionsmodeller och glidning i stagningssystemet utförts. Till exempel, så har det visats att bärförmågan hos slanka konstruktionselement är mycket känslig för valet av dess initiella imperfektion. Den imperfektion som är kritisk för den framräknade bärförmågan av konstruktionselementet är generellt inte samma imperfektion som är kritisk för kraften som skulle uppkomma i stagningssystemet. Glidning i stagningssystemet kan reducera bärförmågan hos stagade kontruktionselement och ge upphov till större stagningskrafter.

Med syfte att utreda hur stagningsstyvheten i slanka takkontruktioner påverkas av olika tekniska lösningar har ett fullskaleförsök utförts i laboratoriemiljö. Skivverkan och stabilisering med vindkryss beaktades i försöket. För att utvärdera takets styvhet utvecklades en provningsrigg som kan påföra en horisontell punktlast i takets plan. Med hjälp av numeriska modeller och kalibrering av dessa avseende resultaten från laboratorieförsöken kunde till exempel styvheten hos olika förband i takkonstruktionen fastställas. Studien visar att styvheten hos stagningssytem kan kraftigt överskattas om förband inte beaktas på ett adekvat sätt vid modellering. Testmetoden kan också användas för mätningar på riktiga takkonstruktioner med syfte att säkerställa tillräcklig stagningsstyvhet (det vill säga att säkerställa att dimensioneringsantagandena stämmer).

Med syfte att utöka kunskapen om hur slanka tak hanteras i praktiken och identifiera potentiella felkällor relaterade till dimensioneringsprocessen, har en enkätstudie riktad till erfarna konstruktörer utförts. Resultaten indikerar att många konstruktörer tror att takras beror på felaktiga konstruktionsberäkningar. De tror också att förbättrad samordning mellan olika aktörer i ett byggprojekt skulle förbättra säkerheten generellt. Studien visade också, emellertid, att konstruktörernas antaganden rörande slanka konstruktioner skiljer sig kraftigt åt i vissa avseenden; vissa konstruktörer gör till exempel väldigt osäkra val som skulle medföra reducerad säkerhet. Grundlig och oberoende granskning av konstruktionsdokumentation ses som viktiga åtgärder för att höja säkerheten avseende slanka tak.

Slender roof structures: Bracing, modelling, full-scale testing and safety

1 Introduction

1.1 Background

The work presented in the thesis stems from the many (>180) failures of slender roof structures that occurred during the winters of 2009/10 and 2010/11 in Sweden. These failures, however, are merely examples of comparable happenings. During the winter of 1976/77 similar failures occurred in the same regions in Sweden, during the winter of 2005/06 more than 50 collapses occurred in Germany, Poland and Austria, and during the winter of 2008/09 more than 100 collapses took place in the US (in inland regions in the northwest). Most of the failures occurred at snow loads that were less than the design loads involved. This clearly indicates there to have been errors in the design of these structures. Common failure modes of slender roof structures include connection failures and instability (or inadequate bracing); a selection of failure cases is described further in Section 2.

Moreover, structural failures during the construction process also occur quite frequently (Frühwald *et al.*, 2007; Scheer, 2010). In fact, many failures of both buildings and bridges have occurred during this phase, commonly due to either a lack of adequate temporary bracing or an under-engineered formwork. Altogether, this reflects the fact that the need for temporary bracing during construction appears to easily be overlooked.

Since building collapses can lead to the loss of human life, to serious injuries, as well as to economic losses, they cannot simply be tolerated by society. Therefore, building codes are written in order to control how buildings are designed and how sufficient safety can be ensured. Nevertheless, despite such code regulations, buildings do fail.

The present thesis is the second and final report (PhD thesis) of the project described further in Section 1.2. An interim report, in the form of a licentiate thesis (Klasson, 2015), was presented earlier. An overview of the entire project and of the different activities involved is shown in Figure 1.1 that follows.

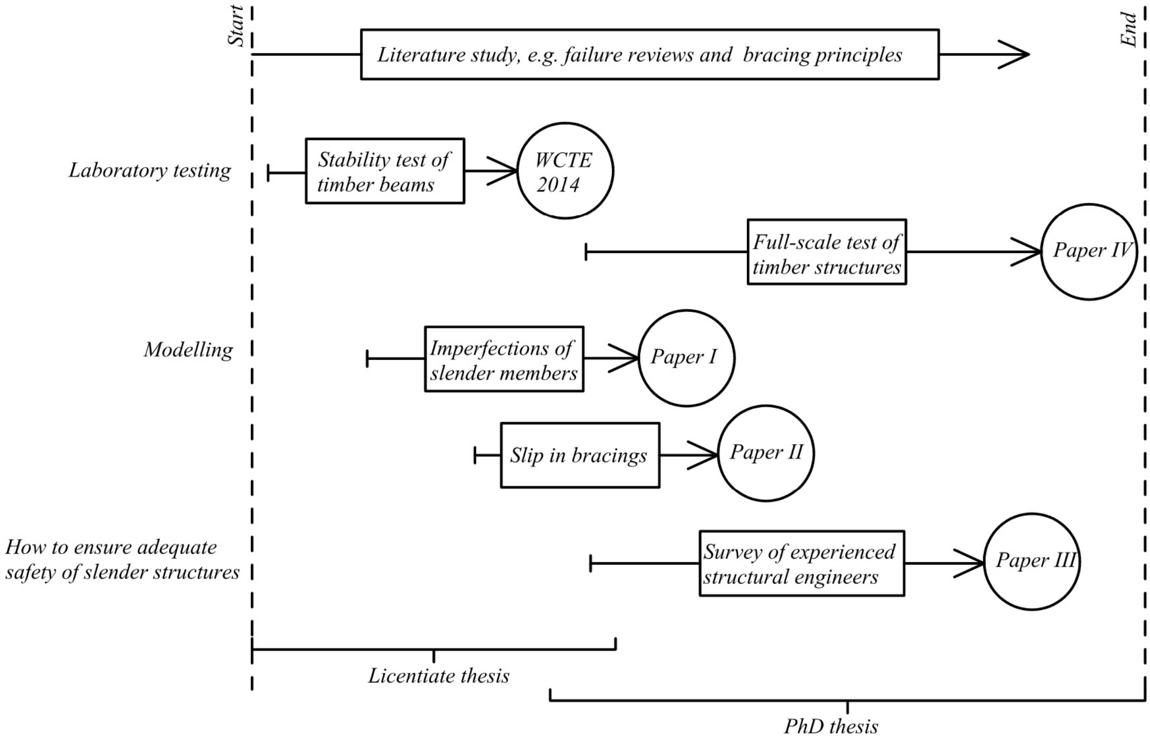


Figure 1.1: An overview of the project and of the different main activities involved.

1.2 Objectives

The overall objective of the thesis is to increase awareness of the need of ensuring the adequate safety of slender roof structures, specifically of designing them so as to ensure their stability.

Also, the thesis aims at contributing to the knowledge of certain specific technical aspects of the design of slender structures.

Within the framework of these overall objectives, the more specific aims of the thesis are as follows:

- To provide an overview of different structural collapses that have occurred, emphasis being placed on failures of roof structures that occurred due to heavy snow loading, and to identify the major causes of the failures that occurred (Section 2).
- To investigate the effects of different design assumptions that are of importance in the design of slender structures, in particular for cases in which advanced mod-

elling is employed (Section 3.4, Papers I and II).

- To explore how experienced structural engineers approach the designing of slender roof structures (Paper III).
- To provide a basis for discussion of the possible implications of subjective choices made by structural engineers in designing slender roof structures (Section 5 and Paper III).
- To develop a methodology concerning how to ensure adequate bracing stiffness in real structures, e.g. during their construction (Sections 4 and 5).
- To conduct laboratory tests of common bracing systems in order to learn more of the effect on the bracing stiffness of their different structural members (Section 4 and Paper IV).

1.3 Limitations

The investigation of engineering assumptions important to advanced modelling is limited here to the study of slender steel columns and slender timber beams, respectively. Specifically, the effects of imperfections, bracing stiffness and potential slip on such bracing systems are analysed.

The laboratory testing is limited to a specific timber structure, one consisting essentially of columns, beams and purlins. Two different bracing approaches are analysed, those of using 1) diaphragm action and 2) wind trusses.

The failure considerations and the research carried out here are focused primarily on cases that involve inadequate bracing and instability.

1.4 New findings

New findings that the research project provided include the following:

- In view of the fact that the imperfection modes that generate the largest bracing forces for slender structural members are usually not the same modes as those that are critical for the load-bearing capacity of these members, more than one imperfection shape needs to be considered in the structural design.

- Slip that may occur in bracing systems lead both to greater stresses and to greater lateral deformations of the braced structural members involved. Thus, slip in bracing systems can significantly reduce the load-bearing capacity of the braced members.

- Slip in bracing systems also increases the bracing forces.

The laboratory tests of a specific slender roof structure that were carried out indicate the following:

- In timber structures, connections between bracing members and primary load-bearing members can have a significant effect on the lateral stiffness.

- Bracing systems using wind trusses and those using roof decking tend to be about equally stiff (for the specific conditions employed in the study).

- It is crucial for the adequate designing of slender roof structures that one seeks safe assumptions regarding bracing systems, in view of the fact that having nominal material parameters and "rigid" connections between different members (an approach that is commonly a standard one in FE-programs)results in marked overestimates of the stiffness of the structure in question.

Conclusions that could be drawn from the survey of experienced structural engineers include the following:

- Many experienced structural engineers believe that roof failures originate from faulty design work. This indicates a need for improved third party review of design documentation.

- Discrepancies between the design assumptions made by different experienced structural engineers concerning slender roof structures could be noted. Some of these assumptions made, such as regarding the buckling length of a beam, were not safe.

1.5 Outline of the thesis

The overall structure of the thesis expresses the aims of both reflecting the objectives provided in Section 1.2 and introducing the research reported in the papers that are appended.

Section 2 provides an extended background to the many structural failures that have occurred since the mid-1970s due to heavy snow loading. Some major building failures that have taken place during construction are discussed as well. In addition, an historical reflection on the importance of learning from failures is provided.

Section 3 of the thesis is used to introduce the concept of bracing; the different criteria applied to bracing systems for slender structures are discussed. Also, the modelling of slender structures and the different uncertainties involved in such modelling are taken up.

Section 4 provides a more thorough presentation of a method that was developed for determining the bracing stiffness of full-scale roof structures. The laboratory tests that were conducted are described here as well. Also, a brief introduction to important matters to consider when conducting laboratory tests, on both a full- and a reduced scale, as well as field testing of real structures, is provided.

Section 5 serves to introduce ideas regarding how to ensure the adequate safety of slender structures during the design and construction phases.

In section 6 the conclusions drawn in the thesis and in the papers that are appended are presented.

In section 7, suggestions for further research are provided.

Section 8 provides a summary of the appended papers I-IV.

2 Structural failures

In the thesis, a structural failure is defined, if nothing else is stated, as a total loss of the load-bearing capacity (sometimes with a residual capacity) of a component or a member of a structure, or of the structure as a whole. In other literature that is cited here, a temporary loss of the functionality and serviceability of a building, for example, may also be referred to as a structural failure.

A general discussion of the role of structural failures is provided in Section 2.1. A brief discussion of some spectacular failures of finished structures and of failures that occurred during construction is provided in Section 2.2. Section 2.3 contains a more thorough discussion of failures that took place during three different winters due to heavy snow loading, these being the main focus of the thesis.

2.1 The importance of learning from failures

Historically, structural failures have played an important role in the progress of engineering knowledge and design (Blockley, 1980; Petroski, 1985; Björnsson, 2015). For example, one can note that principles of trial and error decided the angle of ancient Egyptian pyramids and the design of medieval domes and similar buildings in Europe; see Figure 2.1. Analogously, more recent failures of hundreds of railway bridges during the 19th century (Feld & Carper, 1997), have been a very important source of information for the development of modern engineering approaches. Following the collapse of the Murrah Federal Building in Oklahoma City in 1995 and of the World Trade Center in New York City in 2001, for example, research within the areas of structural robustness and of the principles of progressive collapse has been intensified (Björnsson, 2015).

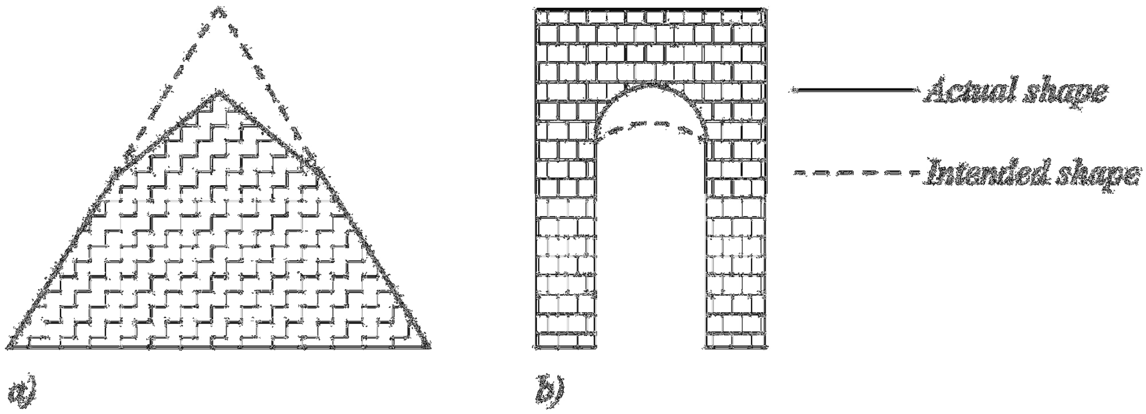


Figure 2.1: Examples of how principles of trial and error may have decided the design of ancient buildings: a) the angle of a pyramid, and b) the radius of an archway.

It should be emphasized that many structural failures that have occurred in modern times have been due mainly to gross human errors rather than to intended experimenting (trial and error), which may have been the case in the past. According to Kaminetzky (1991), human errors can be divided into three different categories, namely (1) errors of knowledge (ignorance), (2) errors of performance (carelessness) and (3) errors of intent (greed). According to many recent surveys of roof failures (e.g. Klasson *et al.* (2018a) and Frühwald *et al.* (2007)), most failures are due to carelessness or to a lack of proper review of the design documentation that is available.

Historically, failures may well have played an important role in making it possible to learn more regarding structural behaviour. Today, however, both laboratory testing of various models as well as advanced computer modelling can be employed to ensure the adequate safety of structures before constructing them. It should be emphasized, however, that both the adequate designing and the laboratory testing of a structure can only be carried out for a *given* set of circumstances, such as for very specific load cases and boundary conditions that are assumed to be adequate representations of the real structure. The actual loading to which a structure is subjected, and the boundary conditions and material properties of the structure itself are generally all quite uncertain (Nowak & Collins, 2012), their thus being either of a random nature or simply not being completely known. Also, the effects of long-term loading and the potential deterioration of materials can be cumbersome to interpret fully and adequately by means of modelling and laboratory testing alone. Altogether, this means that absolute safety can never be fully ensured.

Many failure investigations highlight both the need and the importance of thorough reporting of cases of failure in order to improve building practices (Wardhana & Hadipriono, 2003; Breysse, 2012; Klasson *et al.*, 2018a). The failure studies described in Section 2.3 span a period of over 30 years yet concern failure modes and structures that are all of basically the same type. This indicates a poor dissemination of failure lessons in general. Thus, the use of improved knowledge and experience is crucial to avoiding potential failures of the same type in the future. The in-depth analysis of cases of failure provides very useful information to practising engineers and should thus be shared, such as through public databases. Unfortunately, the sharing of failure experience is commonly very limited, largely as a result of a reluctance to share information regarding failures, partly for legal reasons and partly for a fear of a ruining one's reputation by reporting such information (Breysse, 2012).

The total number of buildings in the world obviously outnumbers by far the number of buildings that have collapsed during any given period of time. For instance, according to *Statistiska Centralbyrån* (the agency responsible for public statistics in Sweden), the total floor area of buildings in Sweden during the year of 2010 was one of at least 931,239,034m^2, and during the winters of 2009/10 and 2010/11 about 180 buildings in the country collapsed. If one assumes, just to get an idea of the proportions involved, that these buildings had an average floor area of $500m^2$ each, the total area of the collapsed buildings was approximately 90,000m^2, i.e. merely 0.01% of the total building area in Sweden that year. Thus, the problem of collapsing buildings might seem insignificant, yet for obvious reasons the acceptance of building failures tends to be very low, for such reasons, for example, as a fact that a single collapse might readily kill hundreds of people.

2.2 Collapses occurring during the construction of a building or just after the finalization of it

Although the thesis focuses on failures of slender roof structures that occurred during snowy winters, it should be emphasized that other types of building failures likewise occur. Various examples are provided below.

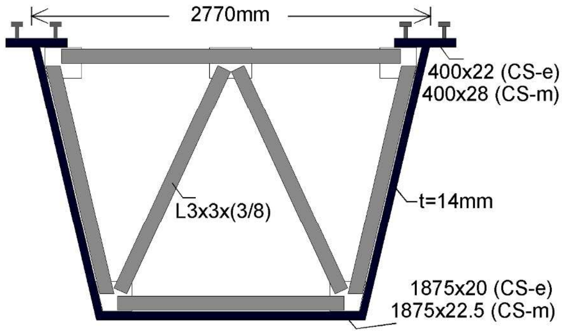
A number of spectacular collapses of long-span timber structures due to inadequate connections have occurred (Thorup & Larsen, 2003; Frühwald *et al.*, 2007; Bell, 2016); e.g. the collapses of 1) the Ballerup arena (a fish-belly-shaped timber truss having a span of 72m) in Denmark in 2003, 2) the Jyväskylä Arena (having glulam trusses with a span of 55m) in Finland in 2003, and 3) the Perkolo bridge (having glulam trusses with a span of 47.5m) in Norway in 2016. The main error of the connections present in the Ballerup arena was that the net timber section in a connection consisting of slotted-in steel plates and dowels was inadequate (a design mistake). In the case of the Jyväskylä Arena, the number of dowels of one of the slotted-in steel plate connections was inadequate. In fact, at the time of the collapse, only 7 of 33 dowels were in place (a production mistake). The Perkolo bridge failed due to a conceptual mistake concerning a butt joint in the bottom chord of the truss. The fasteners in the joint had merely been designed for the difference in horizontal axial force between the left- and the right-hand side of the node point (a design mistake), whereas the principle that applies requires that the member be continuous throughout the joint. When the continuity is interrupted, as it was in the case of the Perkolo bridge, all of the forces in the connection are transferred through its fasteners (Björnsson *et al.*, 2016). At the time of the collapse of each of the the three above-mentioned structures, the structure was loaded by merely a fraction of the design load. This emphasizes the importance of the principles referred to above being adhered to in the designing of a given structure.

According to Frühwald *et al.* (2007) and to Wardhana & Hadipriono (2003), many structural failures take place during construction. The predominant failure mode of buildings during construction is instability of some sort. Recent failures of importance, such as cases that have generated a substantial amount of attention in the media in Sweden, include the failures that occurred in Kista in 2008 (involving the local buckling of the web of a slender steel girder) and in Ystad in 2012 (involving the lateral buckling of a slender steel column)(Fröderberg, 2014), both failures occurring during the construction phase.

According to Scheer (2010), out of 440 bridge collapses that have been documented, 110 occurred during construction. A number of bridge collapses were brought about by inadequate bracing being present during construction (Mehri, 2015), such as in the collapse of the Marcy bridge in New York City in 2002 (due to inadequate bracing of the steel section), see Figure 2.2, and of the Älandsfjärden bridge in Sweden in 2008 (due to the buckling of a number of struts of the form-work).

Altogether, slender structures are very vulnerable to collapse during the construction phase. An obvious reason for this is that the bracing system of the final structure is normally not fully effective during this phase, temporary bracing thus being required to ensure adequate stability.

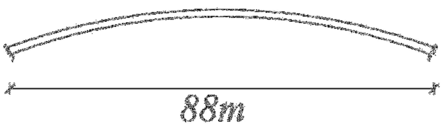
(**a**) A picture of the collapsed bridge.



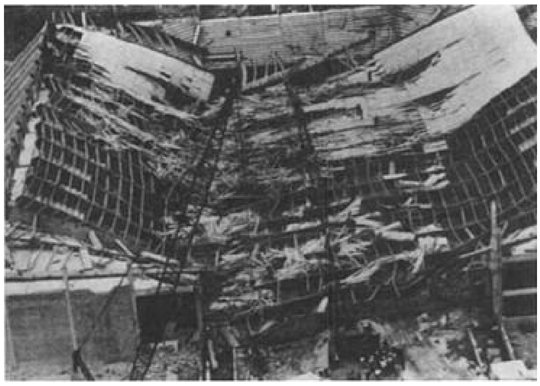
(**b**) The cross section of the bridge.

Figure 2.2: The Marcy Bridge in New York City that collapsed due to insufficient bracing (lateral torsional buckling) when concreting of the bridge deck reached mid-span (Mehri & Crocetti, 2012).

A spectacular example of a building that failed during construction due to inadequate bracing is the Rosemont Horizon Arena (Chicago, USA) that collapsed during its erection in 1979; see Figure 2.3. The roof was built of slender timber arches (span $> 88m$) that were restrained laterally by means of purlins alone. However, all of the bays were unbraced during the construction phase. In its finalized state, there would have been a decking on top of the structure that would have contributed to stabilizing the roof through diaphragm action. During the construction phase, a temporary bracing system would have been required in order to prevent the collapse that occurred. The collapse was initiated by a small wind load in the direction perpendicular to the arches.



(**a**) A sketch of the structure.



(**b**) A picture taken after the collapse. Note that there is no visible braced bay.

Figure 2.3: The collapse of the Rosemont Horizon Arena, USA in 1979. All the images availible were in the public domain - downloaded from https://failures.wikispaces.com.

2.3 Failures of roof structures due to snow loading

As mentioned earlier, many roof structures, all of basically the same types, have failed during winters in which there have been heavy snow loads. In most cases, however, the snow loads did not exceed the design values stipulated by the different building codes, indicating that other factors may have contributed to the failures that took place. It should be noted that many of the failed buildings were of a very basic type, i.e. of a quite simple and common structural type such as that of beams and trusses supported on columns, and of portal framed structures.

Most of the failed structures were of a slender type made either of timber or of steel. Such slender structures can in many cases be very economical due to the limited amount of material needed in relation to the span of the structure. On the other hand, they are known to be sensitive to instability, e.g. to lateral torsional buckling. Thus, in order to ensure the safety of slender structural members it is essential to verify the adequateness of their bracing systems (Yura, 1996; Winter, 1958). Design considerations placed on bracing systems are discussed further in Section 3.

Winters having unusually high snow loadings, this resulting in a large number of structural failures, include 1) the winter of 1976/77 in Sweden (>86 collapses), 2) the winter of 2006/07 in Germany, Poland and Austria (>50 collapses (Dietsch & Winter, 2009)), 3) the winter of 2008/09 in the US (>100 collapses), and 4) the winters of 2009/10 and 2010/11 in Sweden (>180 collapses). Winters (1), (3) and (4) are further described below, in chronological order.

The winter of 1976/77 in Sweden

During the snowy winter of 1976/77 in Sweden, at least 86 slender steel and timber structures failed (Johannesson & Johansson, 1979). In only 12 out of the 86 failure cases did the snow load exceed the design snow load, i.e. a uniformly distributed snow load acting on a flat surface to be used in the design of the buildings. Snow pockets, on the other hand, were found to have been inadequately accounted for in the designing of about 19 of the failed structures. The structural failures occurred basically because of 1) erroneous design and/or design calculations and 2) construction errors/mistakes. A brief overview of different failure modes is provided in Table 2.1.

Table 2.1: A compilation of the structural failures that occurred during the winter of 1976/77 in Sweden. The failure modes and the frequency of each mode are provided. *Pure material failure* includes bending-, tensile-, compression- and shear- failures. *Something else* refers to unknown failure modes.

Instability	20
Pure material failure	35
Joint failure	12
Excessive deformations	9
Something else	10
Summary of failure cases	86

An interesting failure case, one that is a clear example of erroneus design, reported by Johannesson & Johansson (1979), concerns the failure of a hall type building in which a continuous corrugated steel sheet was used to distribute the snow load upon the primary load-bearing members (a system which is a commonly employed in Sweden). For a continuous member held up by three rigid supports, the reaction force acting on the middle support would be greater than the force acting on the supports at the ends. In contrast, in the designing carried out, it is commonly assumed that the load-bearing members in such systems act as flexible supports, so that the vertical reactions of the three supports are virtually identical. For this to be entirely true, however, the bending stiffness of the steel sheet in question needs to be very large in relation to the stiffness of the supports (such as in the case of a wall placed on supports). The study in question emphasizes the fact that, although supports such as those involved are obviously flexible, the most accurate (safe) model assumes something in between rigid and completely flexible supports in estimating the reaction forces involved, such as there being a 15-25% larger force on the support at mid-span; see Figure 2.4.

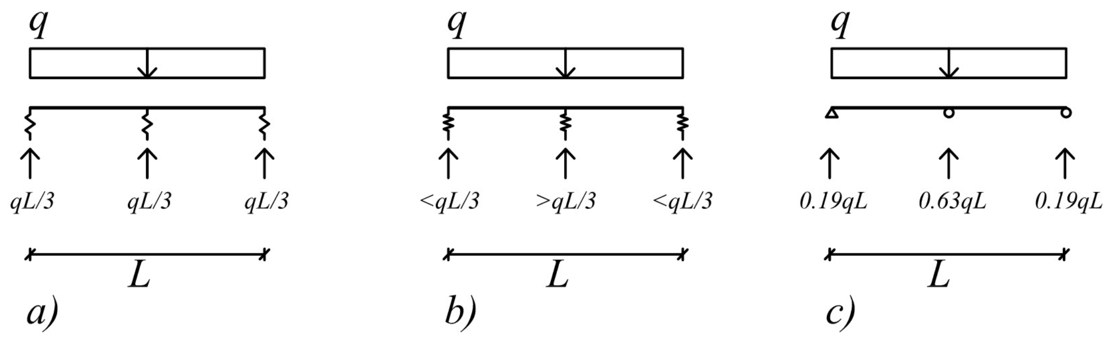


Figure 2.4: A uniformly loaded beam held up by three supports: a) supports of very low stiffness in relation to the bending stiffness of the beam, b) moderately stiff supports, and c) rigid supports.

Instability was the direct failure mode in about 20 of the 86 structural collapses that occurred during the winter of 1976/77, involving such failures as the buckling of steel frames, column buckling, lateral torsional buckling and local buckling (i.e web and flange buckling). In 9 of these 20 cases, the secondary system (i.e. members of the bracing system) buckled, the global stability thus being lost and, accordingly, the building having failed. Examples of failures that occurred due to instability are provided in Figures 2.7a and 2.11b.

The winter of 2008/09 in the US

During the snowy winter of 2008/09 a substantial number of structural failures occurred locally in the Spokane area of the state of Washington in the US (SEAW, 2009). Most of these failures occurred at snow loads that were less than the design loads. The study concludes that properly designed and constructed structures should have resisted the snow loads that were presented. The failure cases indicate gross design and construction errors or mistakes having been present as primary causes. Data were collected concerning 108 failure cases, 95 of them being evaluated more in depth. The structural types involved included slender timber trusses with nail plated joints, heavy timber trusses, framed timber structures and framed steel structures. A majority of the failed structures were made of timber, the failure modes including member fracture, joint failure and instability; certain statistics, based on interpretations by the author, of the failure information that was available are provided in Table 2.2.

The most common structural type that failed during the winter in question could be described as being a slender timber truss having nail-plated joints; see Figure 2.5. The causes of failure reported for systems of this particular type include member fracture, joint fracture and lateral instability.

It should be noted that designing for stability of the compression chord of the abovementioned truss can be rather involved since it is subjected to the combined effect of bending and compression. Instability of the compression chord can thus (theoretically) occur due to lateral torsional buckling, in-plane buckling, lateral buckling and/or torsional buckling. Also, determination of the buckling length of the web members can be rather uncertain due to the complexity of estimating the stiffness of the connections involved. One can obviously make safe assumptions, assuming e.g. there to be a buckling length in the range of 1-2 times the length of the web member, yet the goal of building as cheaply as possible (due to competition between the different contractors that can be involved) often forces structural engineers to make less conservative assumptions.

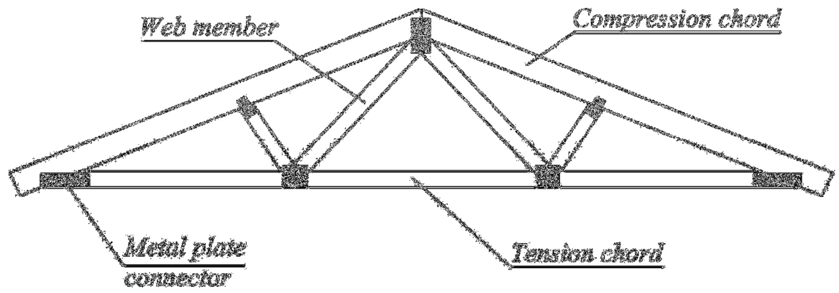


Figure 2.5: Example of a timber truss with nail-plated joints.

Table 2.2: Compilation of structural failures that occurred during the winter of 2008/09 in the US, failure modes and the frequency of each mode being provided. *Pure material failure* includes bending-, tensile, compression and shear failures. *Something else* involves unknown failure modes.

Instability	9
Pure material failure	24
Joint failure	13
Excessive deformations	10
Something else	52
Summary of failure cases	108

The winters of 2009/10 and 2010/11 in Sweden

During the winters of 2009/10 and 2010/11, more than 180 structures in Sweden failed due to heavy snow loading (more than 3000 structures being damaged). Generally speaking, however, the snow loads obtained did not exceed the design loads specified by the building codes that applied at the time of design. About 30% of the collapsed structures were agricultural buildings, these being buildings that can be built without the Swedish authorities having provided specific building permits. Several potential explanations for the collapses appeared in the failure investigations that were carried out (Johansson *et al.*, 2011; Boverket, 2011). These explanations include snowdrift possibly not having been adequately taken into account in the design of some of the structures, snow pockets having been present, nonconforming construction work having been carried out, and the design work being inadequate in a general sense. Failure modes include instability, pure material failures (such as bending failures) and joint failures. Unfortunately, the information available concerning the collapses that occurred during this winter is not as detailed (as it was for the other winters described earlier), so that it has not been possible to provide exact statistics regarding the frequency of the different failure modes in question.

In about 10% of the above-mentioned failure cases, the collapse occurred in conjunction with the removal of snow on the roof. In general, one needs to be careful in removing snow from slender roof structures. For example, when removing snow from a curved roof, such as an arch, one needs to be careful not to create an asymmetric load case that could lead to large bending moments being present in the structure.

The building code (BKR) that applied in Sweden earlier did not specify flat roofs (roof slope $< 15°$) to be designed for snowdrifts, i.e. asymmetrical loading of snow. Investigations of the collapsed buildings have shown, however, that snow drift can also occur on flat roofs (Johansson *et al.*, 2011). Accordingly, asymmetric load cases need to also be considered in connection with flat roofs (consideration of asymmetrical load cases is recommended by the European design code EN 1991-1-3 (2003)). Despite this, snow drift should normally not be a major issue for flat roofs, in particular not for roofs involving the presence of simply supported beams. The total loading on the roof is usually less in the case of asymmetric loading than in the case of uniformly distributed loading; see Figure 2.6. The total loading in the uniform case is $L\mu_1$ whereas the total loading in the asymmetric case is $\frac{3}{4}L\mu_1$. Also, the maximum bending moment and the maximum shear force are larger in the uniform case than in the asymmetric one. Curved roofs, involving arches for example, may on the other hand be more sensitive to asymmetric loading, as was described earlier.

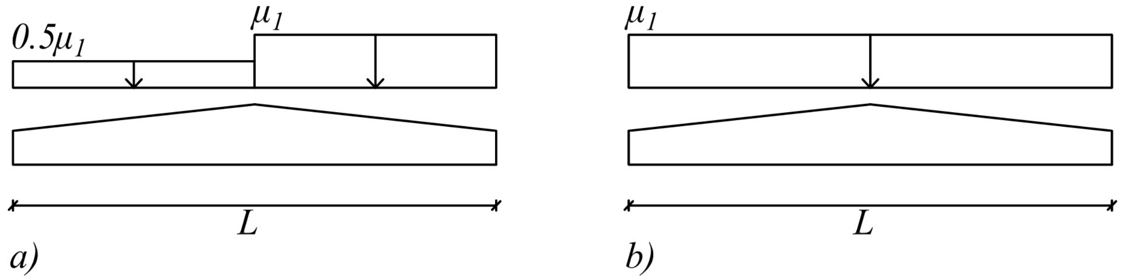
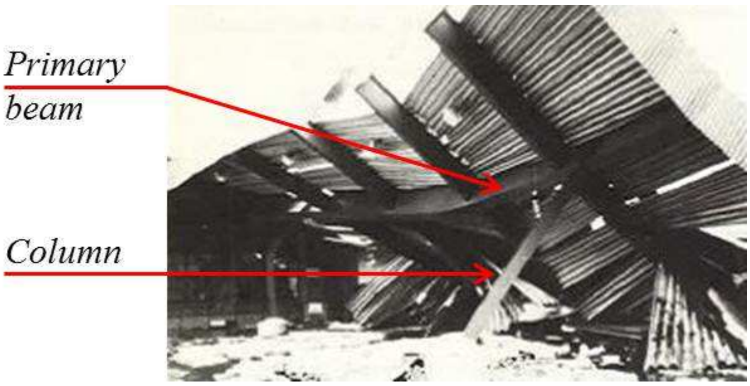


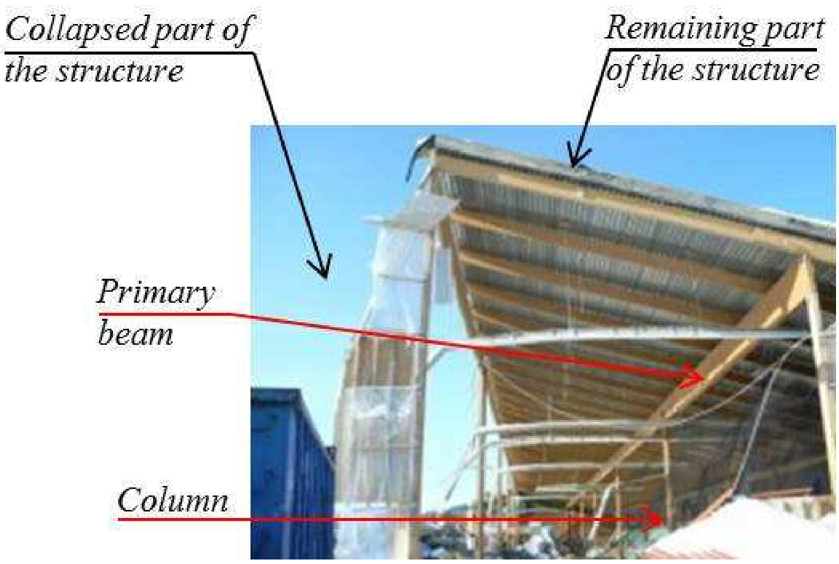
Figure 2.6: Snow loading on a roof according to EN 1991-1-3 (2003): a) asymmetric loading, and b) uniformly distributed loading.

A majority (60%) of the buildings that failed during the winters of 2009/10 and 2010/11 were built after 1980. However, most of the collapses that occurred during the winter of 1976/77 took place in the same region. This may partly explain why a majority of the buildings that failed during the later winters were built after 1980, older buildings that were lacking in capacity having already collapsed during the winter of 1976/77.

An interesting failure case reported by Johansson *et al.* (2011) concerns the failure of a hall type building having a continuous timber beam on top of several supports (columns); see Figure 2.7. The continuity of such beams gives rise to negative bending moments over the intermediate supports when the beam is loaded by gravity loads (e.g. snow); see Figure 2.8. This means that the bottom side of the beam is in a state of compression within this section of the beam for the load case in question. In the failure case that was reported, the beam was not braced on the compression side of its cross section over the intermediate supports, meaning that the beam was stabilized merely by the potential torsional rigidity provided by the steel sheeting and the purlins, and by the member itself (usually very small for rectangular timber sections); see Figure 2.9a. Such torsional rigidity, on the other hand, is commonly very low for timber structures. The connection between the purlin/steel sheeting and the main beam behaves similarly to a pinned one; see Figure 2.9b. Thus, the most probable failure mode of the building in question was lateral torsional buckling initiated over the intermediate supports. Bracing of the type shown in Figure 2.9c would have been required in order to ensure a sufficient lateral torsional buckling capacity of the structure in question.



(a) Winter of 1976/77 in Sweden (Johannesson & Johansson, 1979).



(b) Winter of 2010/11 in Sweden (Johansson *et al.*, 2011).

Figure 2.7: Lateral torsional buckling, due to inadequate bracing, of primary beams that are continuous over supports (columns); the compressed sides of the primary beams being unbraced at the positions of which the columns are located.

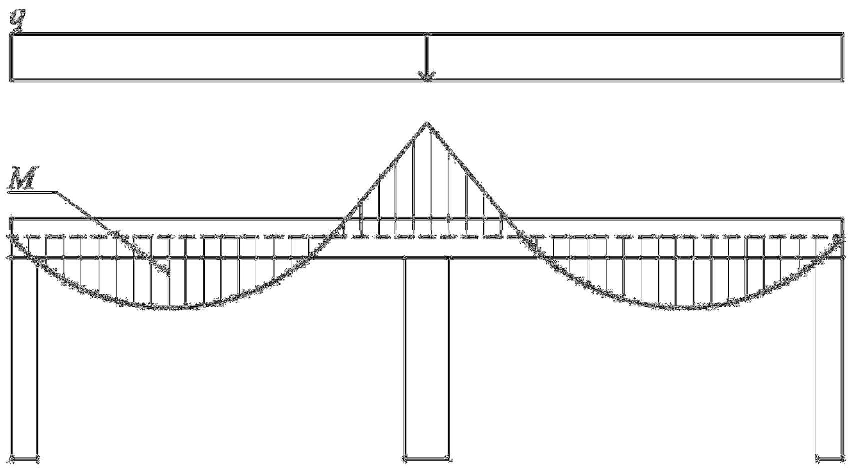


Figure 2.8: Moment distribution (M) of a uniformly loaded continuous beam.

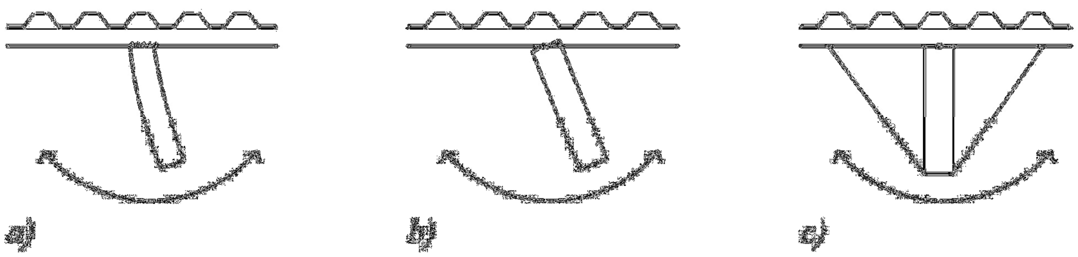


Figure 2.9: A primary beam in a roof structure rotating around its longitudinal axis due to negative bending: a) torsional rigidity being provided by the purlins and the roof cladding, b) the joint between the primary beam and the purlin being a pinned one, and c) torsional bracing of the cross section being provided by means of tension ties connected to the bottom side of the member.

Another interesting failure case reported by Johansson *et al.* (2011) is the collapse of a three-hinged portal frame structure used as a riding hall. Similar structures have failed during all the winters referred to above; see Figure 2.11. Similar to the case described above in which negative bending over the intermediate supports of the beams occurred, negative bending here would occur at the knee points of these frames when they are loaded by gravity loads; see Figure 2.10. Many of the failure cases in question indicate an inadequate lateral and torsional stiffness of the cross-section at these knee points, due to knee buckling laterally taking place, as in the example provided in Figure 2.11c. If the lateral (and torsional) stiffness of the "knee" section of a portal frame is too low to resist buckling, bracing (similar to the bracing shown in Figure 2.9c) is obviously required.

Altogether, failures of continuous beams and of portal framed structures indicate that the need for bracing in areas of negative bending appears to easily be overlooked in designing of the structure.

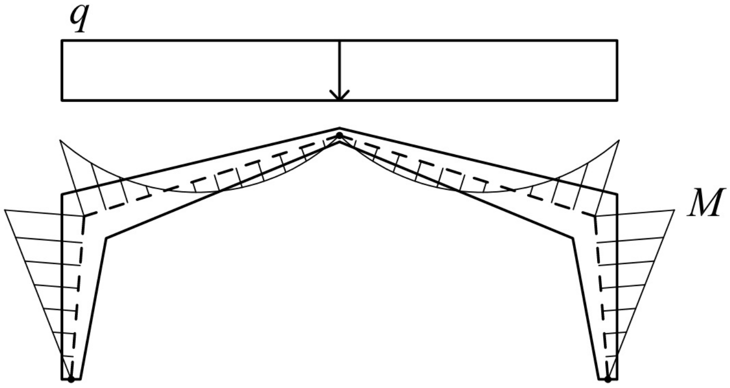
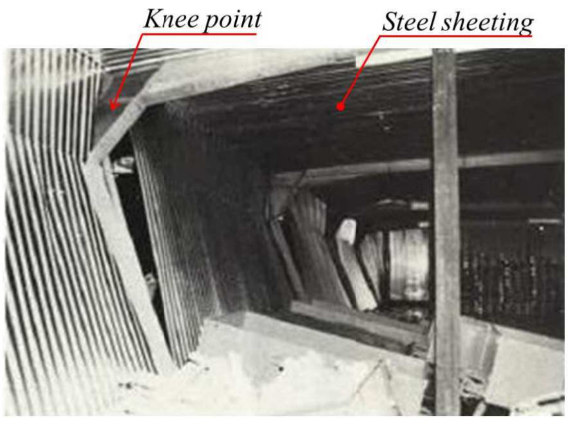
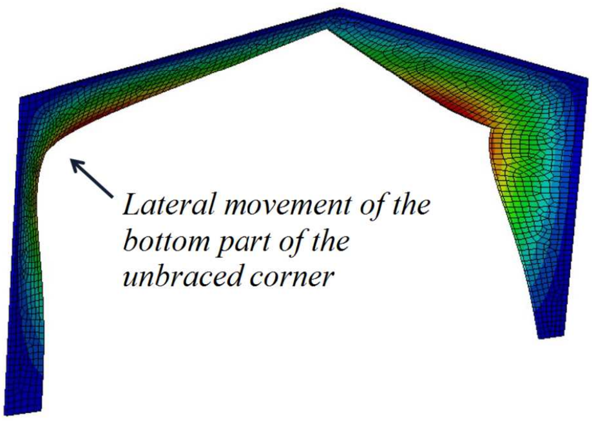


Figure 2.10: Moment distribution of a uniformly loaded three-hinged portal framed structure.

(a) Collapse that occurred during the winter of 2010/11 in Sweden (Johansson *et al.*, 2011). The frames were stabilized laterally by the use of purlins made of wood.



(b) Collapse that took place during the winter of 1976/77 in Sweden (Johannesson & Johansson, 1979). The three hinged portal steel frames were stabilized laterally by use of a corrugated steel sheet (diaphragm action) that was assembled on top of the frames.



(c) Buckling analysis of a slender portal framed structure subjected to gravity loading. The steel frame is restrained laterally along its top side.

Figure 2.11: Buckling (at the knee points) of portal framed structures made of steel, examples from two different winters being shown.

3 Bracing of slender structures

Bracing is important for controlling lateral deformations (including second-order effects), buckling lengths and the load-bearing capacity of slender members, and for stabilizing structures against the effects of horizontal loading (such as through wind or earthquakes), during both the construction phase and the service phase of the structure (as discussed in Section 2). A discussion of important aspects of bracing systems is provided in Section 3.2. Different approaches to how the adequate bracing of slender structures can be accomplished are provided in Section 3.3. How to model bracing and various uncertainties that such models involve is discussed in Section 3.4.

What is a slender structure? The definition of it is not always univocal. A discussion of this topic is provided in the next section (Section 3.1).

3.1 Definition of slender structures

It is generally assumed that a *slender structure* consists of one or more *slender structural members*.

Commonly, the so-called relative slenderness ratio, λ_{rel}, is used to define the slenderness of a structural member; the higher the value of it, the more slender the structural member is. The slenderness ratio is very useful in the designing of a slender member since it correlates with the load-bearing capacity of the member. The load bearing capacity differs from the elastic capacity of a member by taking the material limitations (e.g. the yield strength), the geometrical imperfections, eigenstresses and the second order effects involved into account (Timoshenko & Gere, 1961; Trahair, 1993). The "design curve" involving the relative slenderness ratio, $\chi(\lambda_{rel})$, is calibrated so as to be sure that most of the test results obtained for a particular structural member are on the safe side, i.e. so that the expected load-bearing capacity of the member is greater than the design value that is recommended for most cases that are encountered; Figure 3.1 presents an axially loaded column as an example of this. Design codes, such as the Eurocodes, usually provide design curves for beams and for columns.

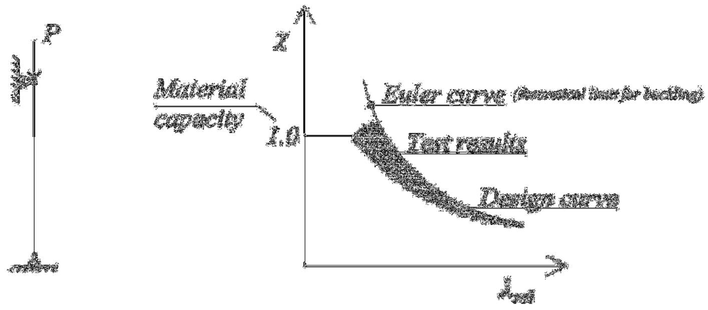


Figure 3.1: A schematic diagram of a typical design curve for the design of columns: $\lambda_{rel} = \sqrt{N_p/N_{cr}}$ and $P_u = \chi N_p$. N_p being the plastic capacity of the column, N_{cr} the Euler buckling load, and P_u the "capacity" of the column.

The relative slenderness ratio can be explored further by the following example (Equations 3.1-3.2) involving a compressed strut (steel column).

$$N_{cr} = \frac{\pi^2 EI}{L_{cr}^2} = [\lambda = \frac{L_{cr}}{i}] = \frac{\pi^2 EA}{\lambda^2} \tag{3.1}$$

where N_{cr} is the elastic critical load of the strut, EI its bending stiffness, L_{cr} its buckling length, λ its slenderness ratio, i its radius of gyration, and A its cross sectional area.

The relative slenderness ratio is defined in accordance with Equation 3.2 where the expression arrived at in Equation 3.1 is also being implemented.

$$\lambda_{rel} = \sqrt{\frac{N_{pl}}{N_{cr}}} = \sqrt{\frac{f_y A}{N_{cr}}} = \frac{\lambda}{\pi^2}\sqrt{\frac{f_y}{E}} \tag{3.2}$$

where N_{pl} is the plastic capacity of the member, f_y the yield strength and A the cross-sectional area.

From Equations 3.1 and 3.2 it is evident that the relative slenderness ratio (λ_{rel}) of a member increases as L_{cr} increases. Also, if a member is braced at one or more intermediate points, L_{cr} will decrease, meaning that the slenderness ratio of the member would decrease. Using the the slenderness ratio to define slender members when found in a structure may thus be misleading.

Instead, the following set of points provides a better definition of a *slender structural member* when found in a structure.

- Its out-of-plane stiffness is substantially lower than its in plane stiffness (e.g. height > width).
- Its in-plane load-bearing capacity (e.g. the moment capacity) is substantially greater than its out-of-plane load-bearing capacity.
- It is sensitive to imperfections, so that 2^{nd} order effects increase the moments and the stresses in the member appreciably.
- Its load-bearing capacity is generally dependent upon the performance of its bracing system.

3.2 Bracing requirements

Bracing systems of slender structures usually serve three different purposes, namely 1) to stabilize the structure against external lateral loadings such as wind loads and possible loads from earthquakes, 2) to restrain the structural members from buckling, and 3) to keep lateral displacements of the structure within acceptable limits.

Literature dealing with structural stability, such as Galambos & Surovek (2008), usually defines bracing of four different types, namely those of 1) discrete bracing, 2) continuous bracing, 3) relative bracing, and 4) lean-on bracing. Obviously, actual bracing systems may be combinations of these four types of bracing. The four bracing types can be described further as follows:

1. **Discrete bracing:** It controls the movements of a structural member at a certain point, sometimes called a nodal point; see Figure 3.2.

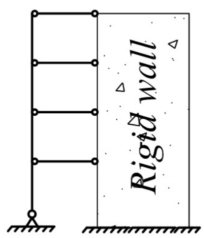


Figure 3.2: Discrete bracing of a column.

2. **Continuous bracing:** It controls movements along an entire face or line of a structural member, its thus being a sort of a continuous spring (despite its being consisting of many successive springs) that is connected to the member; see Figure 3.3.

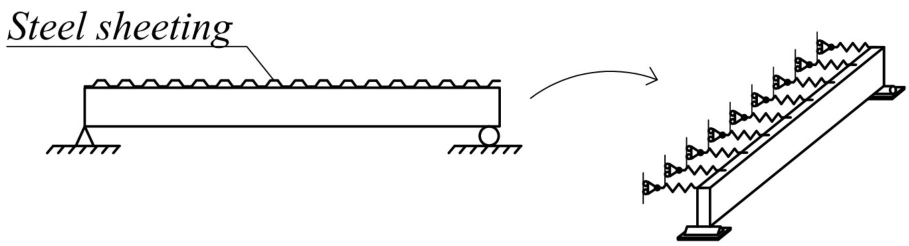


Figure 3.3: Continuous bracing of a beam by decking (with use of a corrugated steel sheet).

3. **Relative bracing:** It controls the *relative* movement (the movement(s) in relation to one another) of two or more members; see Figure 3.4.

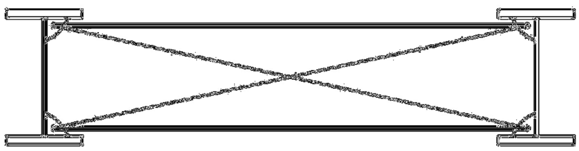


Figure 3.4: The relative bracing of two steel girders, both torsionally and translationally.

4. **Lean-on bracing:** It refers to cases in which a structural member is supported by another, similar member that is not very rigid, such as a pinned column that is braced by a column that is fixed at its base; see Figure 3.5.

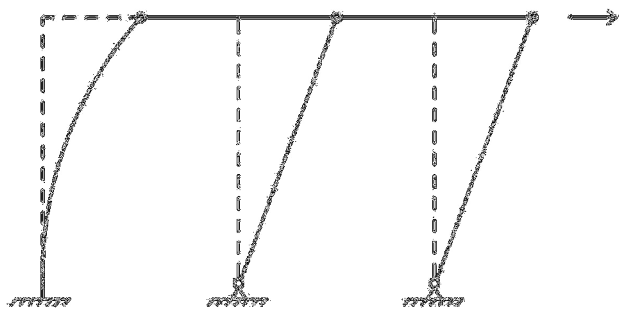


Figure 3.5: A lean-on system of columns.

In addition, the action of bracing can be classified as being purely translational, purely torsional or as being a combination of the two. For many structural members, either translational or torsional bracing may be sufficient to prevent instability of the member.

There are cases, however, in which one of these two types of bracing is clearly more effective than the other. Torsional bracing is obviously required in cases in which torsional buckling and/or lateral torsional buckling needs to be controlled. Pure translational bracing is usually very effective in controlling column buckling, which occurs due to lateral deflections (second-order effects) in one plane. The effects of torsional bracing may typically best be achieved by use of more than one translational brace, the braces that are used being connected to different points on the cross-section of the structural member; see Figure 3.6.

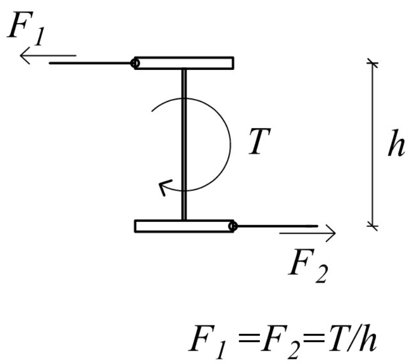


Figure 3.6: Torsional bracing of an I-girder as achieved by means of two translational bracings, i.e. by the force couples F_1 and F_2. It is assumed that the bracings can take both tension and compression.

Loads generated by external loading (wind, etc.) in bracing systems are significantly greater in size than those generated by second-order effects. It may thus appear as though it were sufficient to design bracing systems simply for such loading. However, as pointed out already during the 1950s by Winter (1958), bracing systems also need a certain minimum stiffness in order to adequately prevent the buckling of a given structural member. A bracing system needs to be adequately designed in terms of both strength and stiffness. If the stiffness of a bracing system is too low, the bracing forces can increase dramatically (way beyond loads generated by the wind) as the buckling load of the braced member is being approached. This can be described further by means of a practical example:

Consider the braced strut shown in Figure 3.7, which could be the upper chord of either a truss or simply a column. If the bracing stiffness is set to zero (i.e. $k = 0$), it is obvious that the theoretical buckling length of the strut will be equal to the entire length of the strut (i.e. $2L$). This means that the buckling load of the strut would be $\frac{\pi^2 EI}{4L^2}$, which is the well-known Euler buckling case of a column that is pinned at both ends. It is also obvious that the minimum possible theoretical buckling length of the strut would be half the length of the span (i.e. L), provided the spring is sufficiently stiff ($k > k_{min}$). The

Euler buckling load would in this case be $\frac{\pi^2 EI}{L^2}$, i.e. 4 times as great as in the unbraced case (when $k = 0$).

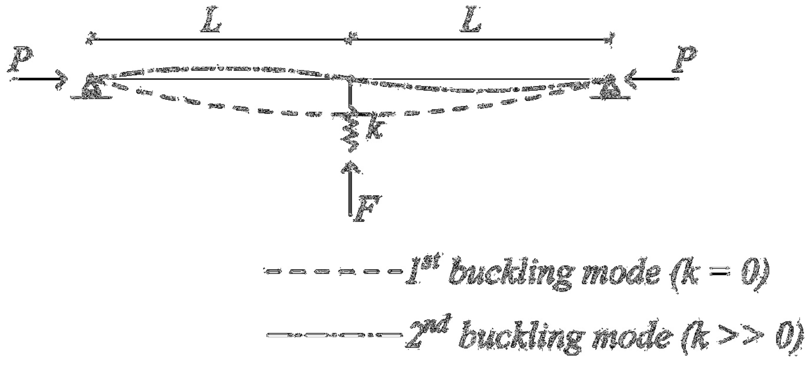


Figure 3.7: A braced strut.

Both the minimum and the maximum possible theoretical buckling load of the strut are thus known. The question that remains is for which minimum bracing stiffness, also termed the ideal stiffness, the maximum buckling load can be reached. Analytical solutions to this relatively simple problem and to similar problems are provided e.g. by Timoshenko & Gere (1961), Bleich *et al.* (1952) and later e.g. by Plaut & Yang (1993, 1995) and Trahair (1999). A more approximate but very practical approach to calculating the ideal stiffness of braced colums was presented by Winter (1958), namely the famous rigid link model. This model is demonstrated below and is used to demonstrate the fundamental stiffness criteria for bracing systems.

In the rigid link method of braced columns, one assumes the presence of fictitious hinges at the brace joints (the contribution of the bending stiffness of the column is neglected at these points), and considers the parts between the bracings as being *rigid links*; see Figure 3.8. For a certain minimum stiffness, k_{ideal}, the column buckles between successive pairs of restraints (a Euler buckling mode). For a bracing stiffness of less than k_{ideal}, the buckling of the column occurs instead as a movement at the brace joint. The ideal stiffness, k_{ideal}, can be calculated on the basis of equilibrium considerations (since the fictitious hinges made the system statically determinate), the buckling being assumed to occur between successive pairs of bracing points. The equilibrium value is calculated for a slightly displaced system, one having a fictitious infinitesimal displacement, Δ, which is assumed to occur at the bracing point. This is a mathematical trick used to obtain an expression involving the bracing stiffness. The method assumes further that the length of the rigid bars is unaffected by the loading and that the geometry of the structure as a whole remains unaffected by the loading (small displacement theory).

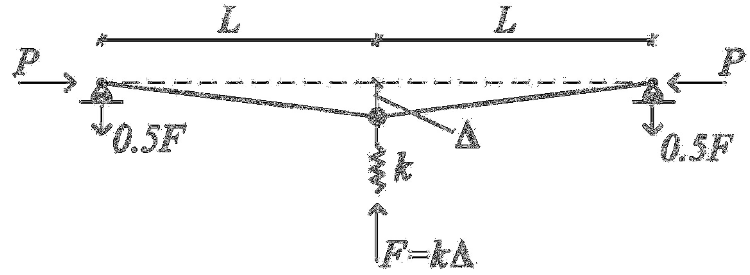


Figure 3.8: A rigid link model.

The first step then is to set up the moment equilibrium for the deformed shape at the point of the fictitious hinge, in line with Equation 3.3 below.

$$P\Delta = 0.5FL = [F = k\Delta] = 0.5k\Delta L \tag{3.3}$$

where P is the axial load, that is applied, Δ is the displacement at the bracing point, F is the force of the bracing, k is the stiffness of the bracing and L is the distance between the restraints.

Due to the assumption of there being only small displacements and of the geometry not being affected by the loading, the fictitious displacement, Δ, in Equation 3.3 cancels itself out. The remaining expression is then solved for the bracing stiffness k, in accordance with Equation 3.4, as follows:

$$k = 2P/L \tag{3.4}$$

The maximum possible load that the system can sustain is that of the Euler buckling load of the segments between the restraints. This means that the ideal stiffness of the system (the minimum bracing stiffness required to ensure buckling between successive pair of bracings) is obtained by simply exchanging P from Equation 3.4 with this Euler buckling load ($P_e = \pi^2 EI/L^2$), in accordance with Equation 3.5 below,

$$k_{ideal} = 2P_e/L = \frac{2\pi^2 EI}{L^3} \tag{3.5}$$

where P_e is the Euler buckling load of the column (having a buckling length of L), EI is the bending stiffness, L is the the distance between successive restraints and k_{ideal} is the ideal bracing stiffness of the column.

A plot of the relationship between the bracing stiffness and the buckling load of the strut (Figure 3.7) is provided as follows in Figure 3.9.

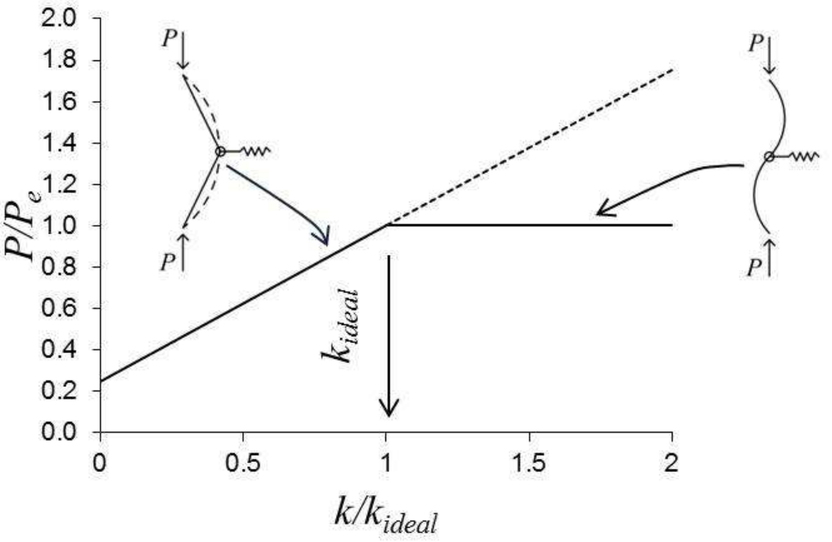


Figure 3.9: The elastic buckling capacity of a braced strut shown as a function of the brace striffness, k_{ideal} is the ideal bracing stiffness, and P_e is the the Euler buckling load when buckling between successive restraints occurs.

The rigid link method can be used for an arbitrary number of bracing points, a plot of a column with three bracing points being provided in Figure 3.10. This system would be able to buckle in accordance with one of four different modes, which of these it is being dependent upon the stiffness of the bracings, as indicated in the plot in Figure 3.10 and as also shown in Figure 3.11.

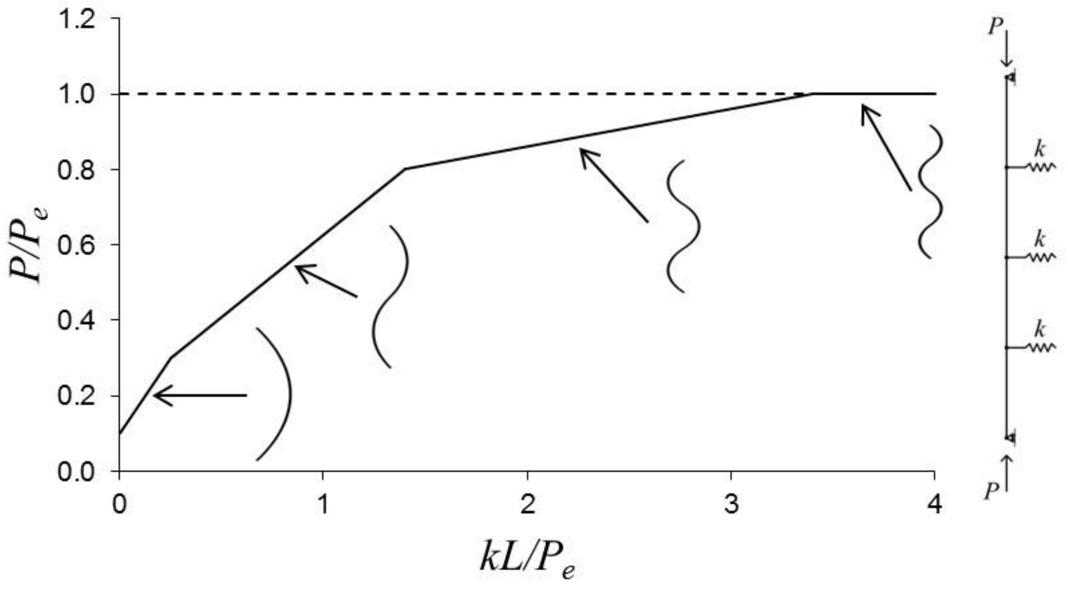


Figure 3.10: Plot of the buckling load P in relation to the bracing stiffness of a braced strut, where k is the bracing stiffness, L is the distance between successive restraints and P_e is the the Euler buckling load when buckling between successive restraints occurs.

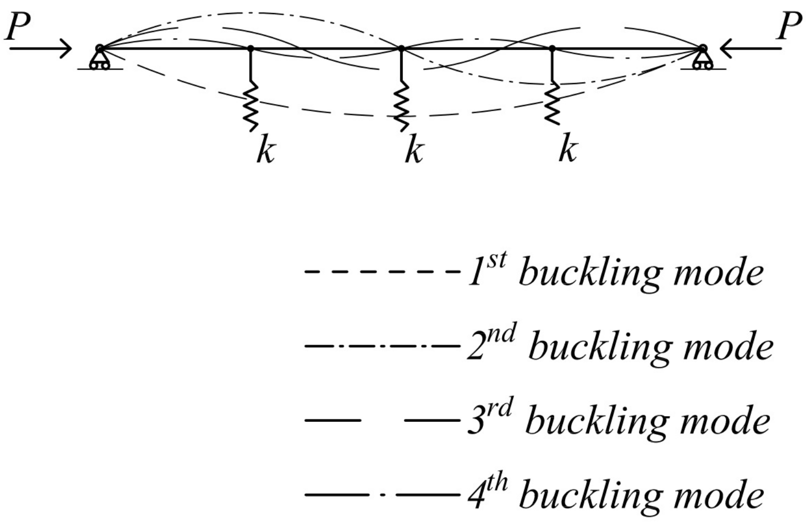


Figure 3.11: Buckling modes of a strut that is being braced at three points.

The examples using the rigid link method above all deal with a theoretically perfect system, i.e. one in which the structural members are assumed to be free of imperfections. It is possible, however, to introduce an initial imperfection, δ_0, into the equilibrium-oriented considerations employed in the solution. When this is done, the method can also provide an indication of the bracing forces that will occur. This can be demonstrated in the following manner:

One would first set up the moment equilibrium for the deformed column, including both the initial displacement (δ_0) and the additional displacement (Δ) that would occur due to the loading, as shown in Figure 3.12. Equation 3.6 below presents the moment equilibrium taken at the point of the bracing. Note that the force of the the bracing is dependent only upon the additional displacement (Δ) that would occur as a result of the loading.

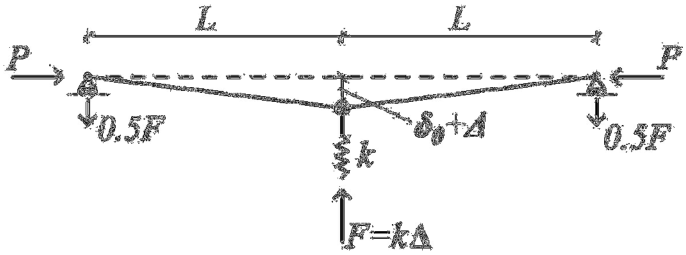


Figure 3.12: A rigid link model involving an initial imperfection (δ_0).

$$P(\delta_0 + \Delta) = 0.5FL = [F = k\Delta] = 0.5k\Delta L \tag{3.6}$$

where P is the axial load that is applied, δ_0 is the initial imperfection at the bracing point, Δ is the displacement occurring at the bracing point, F is the force at the bracing, k is the stiffness of the bracing, and L is the distance between the restraints.

Due to the introduction of the initial imperfection, the previously fictitious displacement, Δ, now no longer cancels itself out of the solution, this displacement being part of the solution to P, in accordance with Equation 3.7 that follows:

$$P = \frac{kL}{2} \frac{\Delta}{(\Delta + \delta_0)} \tag{3.7}$$

Substituting $\delta_t = \Delta + \delta_0$ (the total displacement at the bracing point under a particular load) into Equation 3.7, and solving for δ_t, results in Equation 3.8:

$$\delta_t = \frac{\delta_0}{1 - \frac{2P}{kl}} \tag{3.8}$$

By combining Equations 3.7 and 3.8, and assuming that 1) $k = k_{ideal}$, 2) $k = 2k_{ideal}$, and 3) $k = 3k_{ideal}$, the normalized bracing force F/P is obtained, in accordance with

Equations 3.9, 3.10 and 3.11, respectively,

$$\frac{F}{P} = \frac{2}{L}\frac{\delta_0}{(1-\frac{P}{P_e})} \quad (k = k_{ideal}) \tag{3.9}$$

$$\frac{F}{P} = \frac{2}{L}\frac{\delta_0}{(1-\frac{P}{2P_e})} \quad (k = 2k_{ideal}) \tag{3.10}$$

$$\frac{F}{P} = \frac{2}{L}\frac{\delta_0}{(1-\frac{P}{3P_e})} \quad (k = 3k_{ideal}) \tag{3.11}$$

where F is the force present in the bracing, P is the current axial load, δ_0 is the initial displacement at the bracing point, L is the distance between the restraints, and P_e is the Euler buckling load in the case of buckling between successive bracings occurring.

Plots of Equations 3.9-3.11, assuming $\delta_0 = L/500$, are provided in Figure 3.13. Most importantly, for the case in which the ideal stiffness of the bracing is involved (i.e. $k = k_{ideal}$), the bracing force would tend to infinity as the buckling load is approached. In order to keep the bracing forces at a reasonable level, a bracing stiffness of at least two times the ideal stiffness (i.e. $k = 2k_{ideal}$) is recommended (Winter, 1958; Yura, 1996).

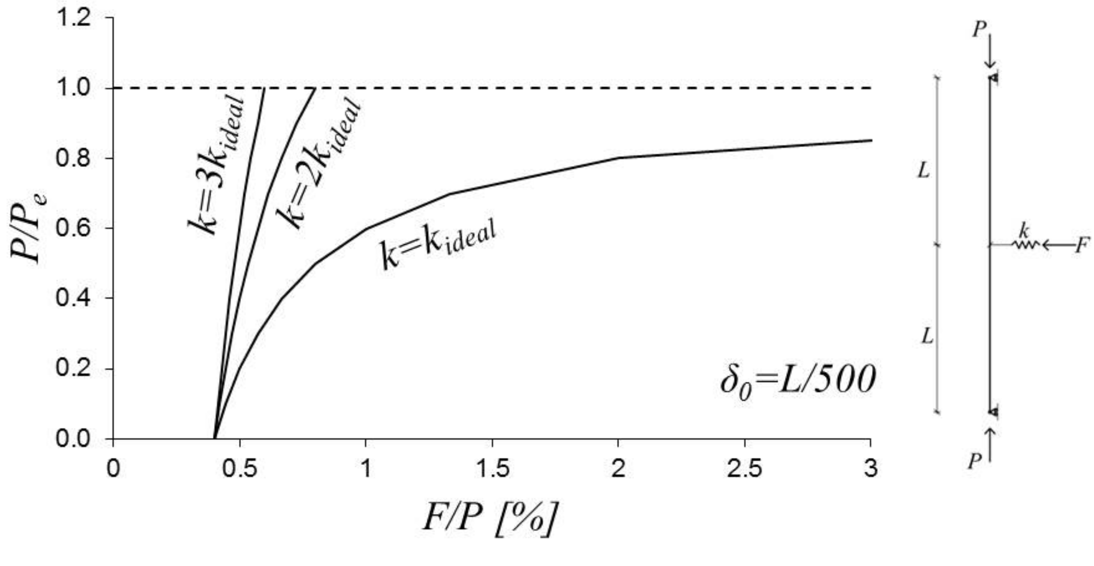


Figure 3.13: The relationship between the force that is applied and the corresponding bracing force of a braced strut obtained for three different values of the bracing stiffness, P is the load that is applied, P_e is the Euler load ($\pi^2 EI/L^2$), k_{ideal} is the ideal stiffness of the bracing ($2P_e/L$), L is the distance between successive restraints, and F is the bracing force.

The rigid link method is a highly approximate but nevertheless useful method for understanding the basic principles of bracing systems, namely those of 1) the importance of the bracing stiffness being adequate for obtaining the desired buckling mode of a structural member and 2) the importance of the stiffness being sufficient to keep the bracing forces at a reasonable level. These two principles are valid for bracing systems and braced members of all types.

3.3 Different approaches to the bracing of slender roof structures

As mentioned earlier, the bracing system of a slender roof structure serves to both stabilize it in regard to the effects of wind and other lateral loads, in whatever directions apply, and to increasing the buckling capacity of the primary load-bearing members. Figure 3.14 illustrates the load paths of a hall type building (one having wind trusses in the roof plane) subjected to wind loading in two different directions. An example of how the configuration of the wind truss affects the expected buckling mode of the primary roof members of a structure is provided in Figure 3.15, its being assumed there that the buckling length is equal to the distance between the node points of the wind trusses, this requiring that the stiffness be adequate (which needs to be ensured by the design).

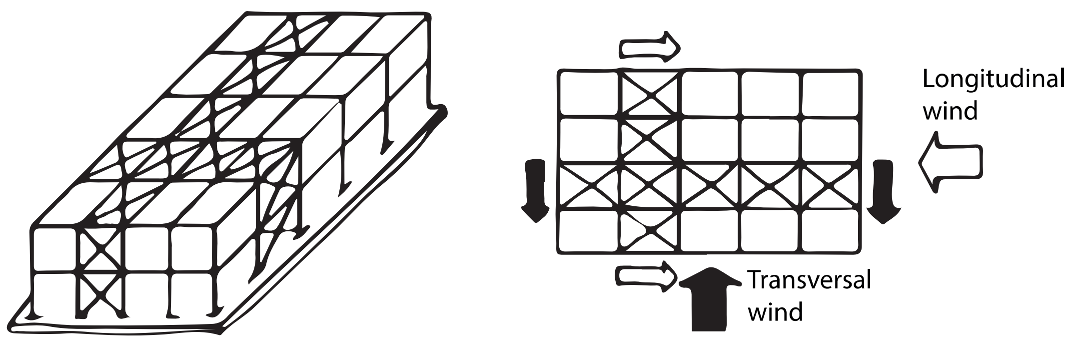


Figure 3.14: Lateral stabilization of a hall type building (Klasson, 2015).

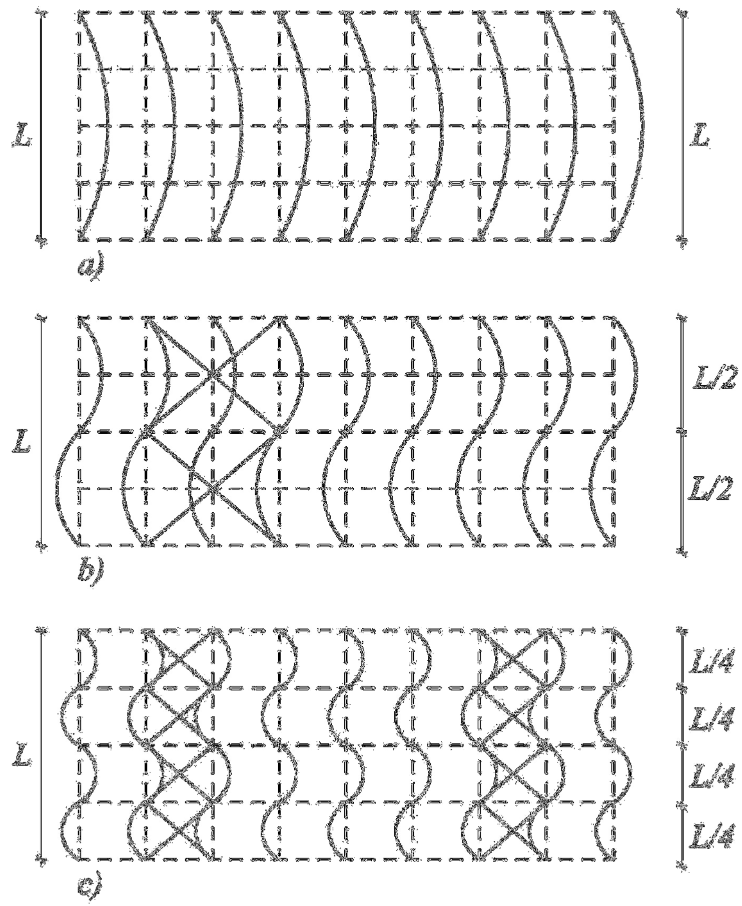


Figure 3.15: Different buckling modes of primary roof members dependent upon the bracing configuration that is found, where a) involves there being no bracing in the plane of the roof, the buckling length being equal to L, b) the wind trusses reducing the buckling length to $L/2$, and c) the wind trusses reducing the buckling length to $L/4$.

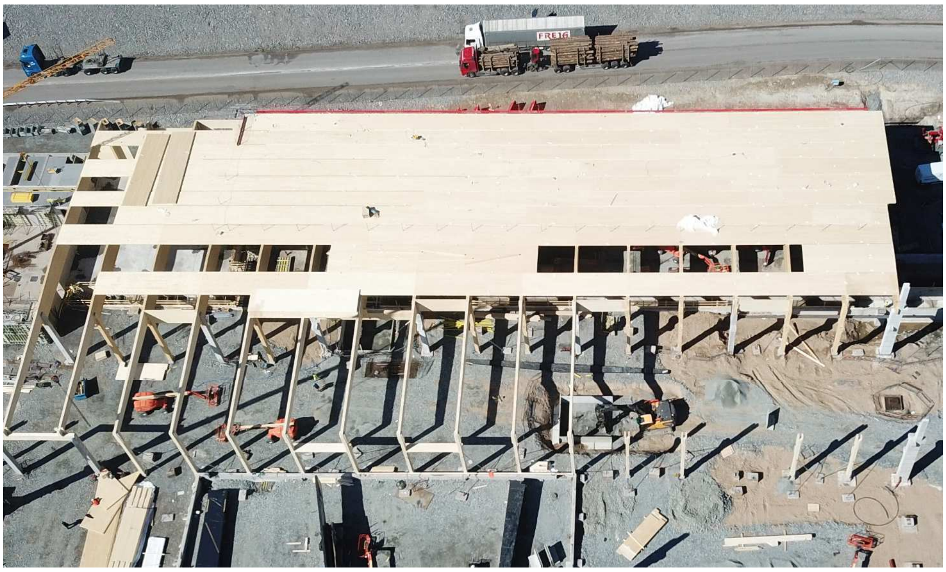
Some structural systems, such as portal frames and arches, are structurally stable in their own plane. Consequently, for roof structures using such systems, bracing is only required in the transversal direction, its being needed there in order to stabilize the building against the effects of longitudinal wind and to adequately restrain the load-bearing members from lateral buckling.

An effect similar to that of having systems that are stable in their own plane can be obtained by the use of fixed columns. If the columns and their connections to the ground are sufficiently stiff and have sufficient capacity, horizontal loads do not necessarily need to be transferred to the ground through diaphragm action in the roof plane. Thus, if the columns are fixed in both the transversal and the longitudinal directions of the building, bracing of the roof structure would mainly be required to ensure adequate buckling capacity of the roof members.

A slender roof structure is usually stabilized by restraining the relative movements of two or more adjacent primary roof members, a so-called relative bracing approach; see Figure 3.4. The roof members that are restrained (e.g. by wind trusses; see Figure 3.14) form something which is usually called the stabilizing bay of the structure. Other primary members of the roof structure are supported by this bay and can be considered as being either discretely braced (if purlins are used) or continuously braced (if a decking is used on top of the primaries in order to obtain diaphragm action). The point here is that the bracing approach of a slender roof structure is basically a mixture of the different types of bracing methods defined in Section 3.2.

The stabilizing bays of roof planes are typically constructed by use of either of two different methods, that of 1) the use of wind trusses in the plane of the roof (according to Figure 3.15) or 2) the use of diaphragm action in the plane of the roof. The latter is usually realized by use of a steel or timber decking on top of the primary roof members; see Figure 3.16 for an example of a timber structure using a CLT-decking so as to obtain diaphragm action for stabilization purposes[1]. If the decking used to produce the diaphragmatic action is assembled directly on top of the principal members, here supposedly simply supported as in the example shown in Figure 3.16, it produces the bracing needed for the buckling length of that member to be considered as being practically infinitely small, implying that buckling considerations during design of that member would normally be unnecessary. It should be noted, however, that this is true only when the diaphragm system, including the connections, has sufficient horizontal stiffness (see also the theoretical case of a continuously braced beam that has a finite bracing stiffness, Section 3.2).

[1] A Swedish project that the author of the thesis reviewed.



(a) The structure as seen from above.

(b) A close-up picture of the CLT-decking.

Figure 3.16: Stabilization of a timber hall type building achieved by means of diaphragm action, i.e. a clt-decking on top of the primary beams. The pictures were taken during erection of the building.

In principle, diaphragm action means that horizontal loads are taken by means of shear action (rectangular panels being deformed into parallelograms) in the roof panels acting as a ”deep” beam that spans across the roof structure. The lateral stiffness of the section of a roof structure used for diaphragm action is usually rather marked since the diaphragm can be designed to ”spanning” over more than one bay of the roof, analogously to a very deep beam. As a consequence, the degree of bending deformation that occurs in diaphragm systems, such as for a wall or a deep beam, is usually negliable; see Figure

3.17a.

Wind trusses in the roof plane are usually more readily deformable in terms of shearing deformation than diaphragms are, due to the high degree of potential flexibility of the connections between the web members and the chord members of the wind truss; see Figure 3.17b. Also, bending deformations are usually not negligible for wind trusses (they usually span only over one bay of the roof plane). Thus, the deformations of a wind truss in a roof structure can generally be regarded as involving a combination of shear and of bending displacements.

In general, due to the factors discussed above, systems using diaphragmatic action are commonly considered to be stiffer than systems making use of wind trusses. Whether this is in fact the case depends, however, on the width of the portion of the roof decking (similar to the height of a beam) that can be considered to be serving effectively as a diaphragm. Also, the shearing connections between the roof decking and the primary members of the roof structure, as well as between the different parts of the roof decking, have a significant effect on the stiffness of the diaphragm. Investigations have indicated, in fact, that diaphragm systems using corrugated steel sheets are not necessarily stiffer than wind trusses that are of the same height as the diaphragm are (Mehri & Crocetti, 2016; Klasson & Crocetti, 2018). The stiffness of the diaphragm is strongly dependent, for example, on the number of fasteners used to connect the steel sheeting with the load-bearing members. According to the results of the investigation presented in Paper IV (Klasson & Crocetti, 2018), however, even when many fasteners were employed, the stiffness obtained is about the same as when use if made of wind trusses. This is discussed further in Paper IV.

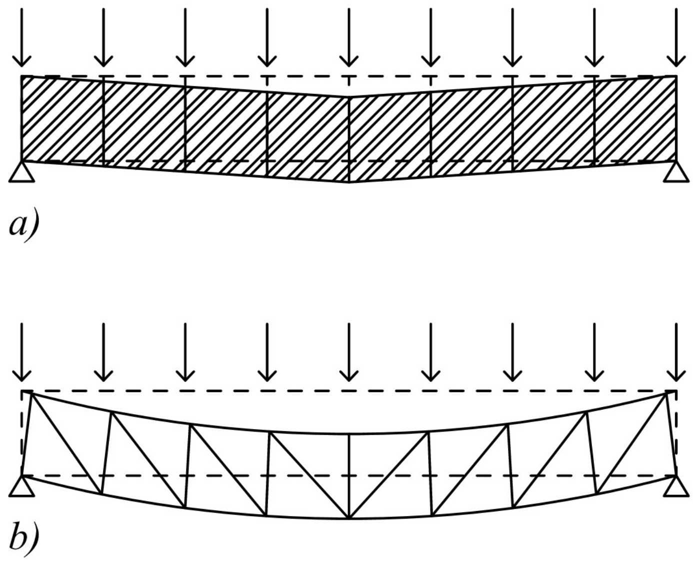


Figure 3.17: Deformations of the roof section of the stabilizing bay of a roof structure, a) when using a decking, so that diaphragm action takes place, the displacement is basically due to pure shear (the bending deformations that occur are usually neglectable), and b) when a wind truss is employed, the displacement then is a combination of shear and of bending displacements.

3.4 Modelling and uncertainties

There are various ways of dealing with the designing and modelling of structures. Prior to the advent of powerful calculating machines, verification of the structure was often carried out on the individual members of it or by use of simplified models of the entire structure, suitable for hand calculations. Typically, the use of simplified methods requires that one have a thorough conceptual understanding of the global behaviour of the structure so that the conceptualization of its different members can be performed adequately. Such conceptualization includes the determination of the loading that affects the different members, its boundary conditions and possibly the stiffness of potential bracings of the members involved.

Although in modern design approaches more sophisticated methods for analysing structures are commonly employed, the ultimate capacity of a structure is still checked on the basis of a so-called individual member design approach, one that is specified in most of the building codes. The actions acting upon a structural member that need to be taken into account in the detailed designing of it are usually extracted from 3D (or sometimes 2D) numerical analyses of the structure in its entirety. Such actions include the moments, shear forces and axial forces present in the member due to a particular loading of the structure as a whole. When solid- or shell-type finite elements are used in modelling, the stresses and strains, rather than the moments and the forces involved, can be extracted from the numerical model.

Finite element programs are widely employed in the present-day designing of structures. Commercial finite element programs are usually very powerful, their being capable of analysing large structures, ones that have many different components and can be subjected to virtually any type of loading, their thus being able to model structures in a highly detailed fashion.

In 3D finite element models, the effects of 1) the load distribution between the different members of the structure, 2) the stiffness of the different members, and 3) second-order effects, can all be accounted for automatically. On the other hand, it is obvious that the accuracy of the results of FE-analyses is highly dependent upon the accuracy of the input used to model the different parts of the structure. Many parameters involved in the structural design are rather uncertain, also when advanced modelling is employed. In addition, the uncertainty of modelling usually increases as the sophistication of the model involved increases (Schlune, 2011; Björnsson, 2015; Björnsson *et al.*, 2016). As a result, structural engineers are often required to make different assumptions regarding a number of rather uncertain parameters in order to be able to model the structure in an advanced manner. This is a matter that will be discussed further below.

For example, in the modelling of slender roof structures one can make use of simple beam models, possibly without taking account of eccentricities between the different structural members, such as the primary and the secondary load-bearing members. One might possibly take such eccentricities into account through the introduction of fictitious rigid links between beams and purlins, for example. An alternative to this aimed at improving the accuracy obtained could be to use either shell or solid elements (or a combination of these) to model the different members of the structure, so that the geometry and the positioning of the members can be more accurately accounted for in the model.

Joints in structures are often sources of uncertainty in structural modelling. Often, joints are modelled either as being pinned or as being fixed. However, joints (connections) are obviously neither purely pinned nor purely fixed. In some FE-software it is possible to model joints in a very detailed fashion, such as by including the stiffness of the fasteners and possibly the friction between the different members of the joint. However, due to the uncertainties and the complexity involved, such modelling is currently not commonly used for designing purposes.

Basically, two different approaches to studying the buckling and the stability of slender structures available in finite element software can be mentioned, those of 1) eigenvalue analysis, and 2) of 2^{nd} (or 3^{rd}) order non-linear incremental analysis. An eigenvalue analysis predicts the theoretical buckling capacity of the ideally elastic and perfect version of a structure or a structural member. The method is similar to that of searching for bifurcation points (i.e. points in the load and the displacement space in which both a displaced and an undisplaced equilibrium of the structure is possible) in classical elastic buckling theory, such as employed in the well known Euler buckling cases, with different boundary conditions, that deal with axially loaded columns. Eigenvalue analysis provides upper-limit (i.e. non-conservative) results for the buckling capacity of a structure, whereby imperfections, geometrical non-linearities and limits to the material strength of the structure are ignored. Non-linear incremental analysis, on the other hand, is usually more realistic and can provide very accurate predictions of the behaviour of slender structures. Their accuracy, however, as discussed above, is strongly dependent upon how adequate various assumptions are.

A highly important assumption in the non-linear analysis of slender structures concerns the initial imperfections of the different structural members (i.e. at a ”local level”) and of the entire structure (i.e. at a ”global level”).

Common ways of assigning imperfections to numerical models of slender structures include 1) the use of one or more buckling mode shapes obtained through eigenvalue analysis, 2) using the displaced shape of the structure as obtained from a set of different static loads, and 3) applying imperfections, i.e. geometrical deviations, directly to the different nodes of the structure in its basic configuration.

The effects of uncertainties regarding both joints and initial imperfections on the modelling of slender structural members were analysed in greater depth in accordance with the aims of the thesis. The effects of imperfections in braced steel columns were discussed in Paper I. The potential effects of slip and slack related to the bracings of slender timber beams were discussed in Paper II. A further account of these uncertainties and of how they can affect the results of advanced forms of analysis is provided in Sections 3.5 and 3.6, respectively.

3.5 Imperfections of structural members

As discussed in the sections above, the input used to model slender structures can often be very uncertain in practice. For example, the imperfections used in carrying out the non-linear modelling of slender structures have been shown to have a significant effect on the results obtained, in terms both of the predicted capacity of the structural members and the bracing forces that would be involved in bracing them during loading, as discussed further in Paper I (Klasson *et al.*, 2016).

Various studies of the effects of the imperfections of structural members have been conducted since the 1960s, three rather recent examples of this being taken up below:

Wang & Helwig (2005) investigated potentially critical imperfection shapes in the torsional and lateral bracing of steel girder systems for different load cases. The authors conclude that the most critical imperfection shape appears to have its maximum value close to the point of the maximum moment of the braced member in terms of the bracing forces involved.

Girão Coelho *et al.* (2013) studied the effects of different imperfection shapes and the presence of local defects on the expected load-bearing capacity of steel columns. They found that the first buckling mode shape of a column does not necessarily correspond to the most critical imperfect shape of it.

Mehri *et al.* (2017) studied the effects of different imperfection shapes on the expected load-bearing capacity of twin steel girder bridges. It was concluded that some imperfection shapes can lead to an overestimation of the capacity of braced beams, such as when using only the first eigenmode of the unbraced beam as the imperfect shape of the braced beam in analysing it numerically.

Altogether, considerable research efforts have been made to endeavor to show what the worst possible imperfection shape of different structures and structural members would be. This has turned out to be a rather difficult task, however. At least partly because of this, numerous studies have investigated the effects of using random imperfections in analysing the buckling capacity of slender structural members. Examples include 1) Kala (2013), who studied the elastic lateral torsional buckling of steel girders containing random imperfections, 2) Zhao *et al.* (2014), who studied the bracing forces of a column, their making use here of random initial imperfections and Monte Carlo simulations, and 3) Gordini *et al.* (2018), who employed a Monte Carlo simulation to show that a large system of space trusses was highly sensitive to random imperfections.

It is well known that structural imperfections are of a random nature. This means the current imperfect shape of the structure being virtually unknown. The application of random modelling techniques to imperfections, as described above, however, although it may be rational for research purposes in particular, is probably less practical for design purposes because of its being so complicated. Structural engineers rather need guidance, the simpler this is the better, regarding how to make qualified and safe assumptions concerning the imperfect shapes of structures in efforts to avoid nonconforming designs.

The essence of Paper I (Klasson *et al.*, 2016) is that at least two different imperfect shapes of a braced structural member need to be considered when the designing carried out is based on the use of non-linear modelling. The imperfect shape of a braced member that generates the largest bracing forces is generally not the same as the imperfect shape that is critical for the load-bearing capacity of the member itself (as described further below). In addition, in order to avoid unrealistic lateral deflection shapes in analysing a member numerically, two or more differing imperfection modes, as based on different eigenmodes of the member in question, for example, need to be represented in the imperfect shape that the model of the structural member possesses, these imperfection modes being superimposed through use of different scaling factors. The potential risks of using an imperfect shape that is too simplistic when analysing a structural member include 1) the risk of overestimating the lateral and the torsional stiffness of the member, and 2) the risk of of overestimating the ultimate load-bearing capacity of the member. In general, the greater the number of bracing points a structural member has, the more eigenmodes there are that need to be represented in the imperfect shape of it, examples concerning three different braced beams being provided in Figure 3.18.

Generally, imperfect shapes of structural members having their maximum displacements at, or close to, the bracing points, such as the first imperfection mode in Figure 3.18 for the beam having one intermediate restraint, are critical for the bracing forces of the member involved. Imperfect shapes having their maximum displacement values between different bracing points, such as in the case of the second imperfection mode of the beam having one intermediate restraint shown in Figure 3.18, are generally more critical when one anticipates the load bearing capacity of the structural members involved.

In general, the recommendations above assume there to be adequately stiff bracings (i.e. bracings that are completely effective). For cases of low bracing stiffness, that should best be avoided, buckling may fail to occur between successive bracings, and/or the bracing forces may be very large. Further advice regarding imperfections of this sort is provided in Paper I (Klasson *et al.*, 2016).

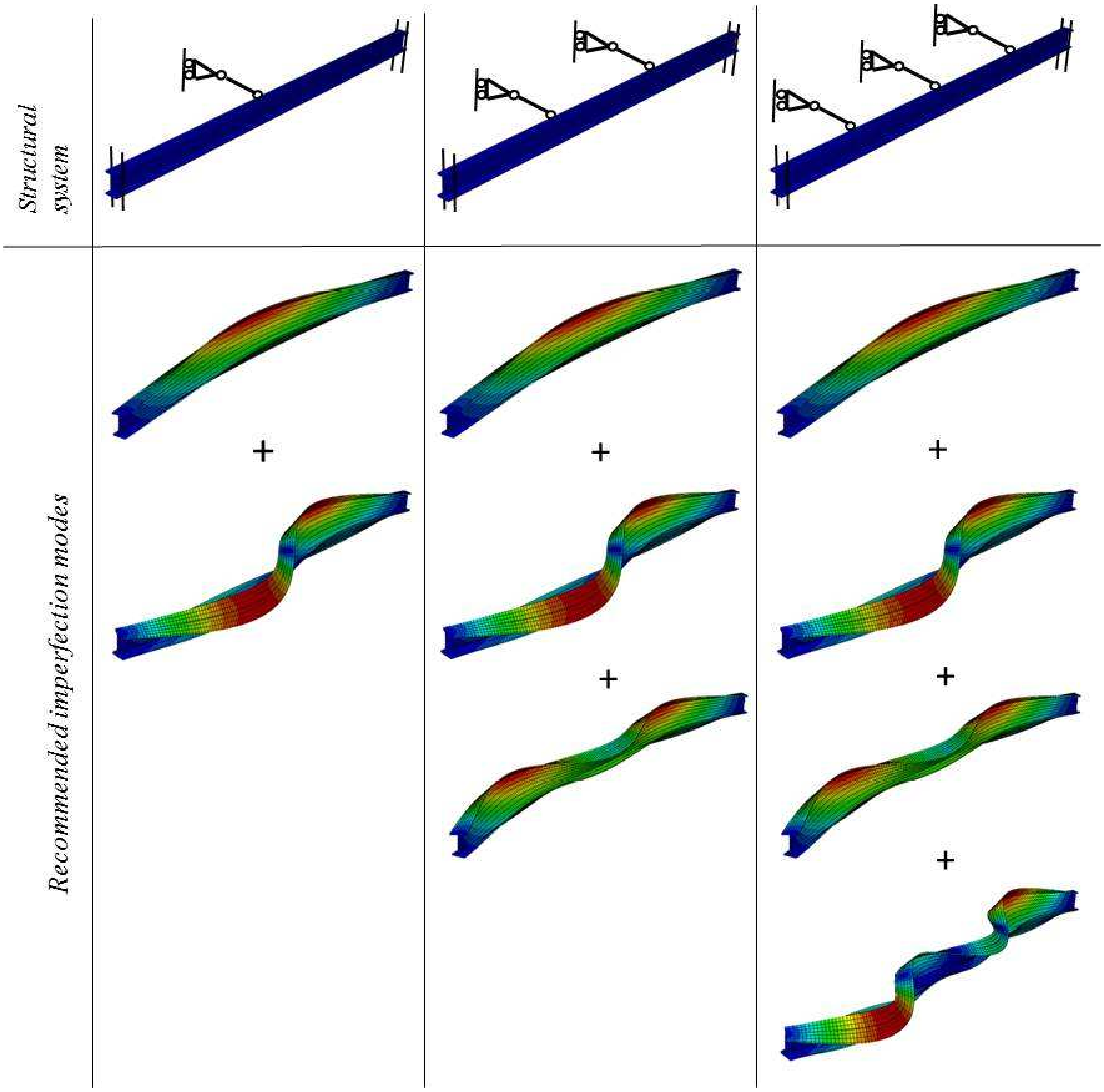


Figure 3.18: Recommended imperfection modes intended for inclusion in the imperfect shape of three different braced beams, 1) with a simply supported beam with one intermediate bracing, 2) with two intermediate bracings, and 3) with three intermediate bracings.

3.6 Slip and slack in bracing systems

Slip in bracing systems refers to case when a certain displacement at the bracing point of a braced member would be required for the bracing to be "activated". Such slip can be due to slack of different members in the bracing system or possibly due to dowelled connections using over-sized holes, for example.

In Paper II (Klasson *et al.*, 2018a), the potential consequences of slip in the bracing systems of discretely braced timber beams were investigated. The results indicate clearly that the presence of slip in bracings of slender beam members leads to increased beam stresses and larger lateral displacements, similar to the effects of having greater initial geometrical imperfections. Also, the bracing forces involved are greater in bracing systems showing slip. An important consideration in the designing of bracing systems is to evaluate the presence of potential sources of slip and of other non-linearities in the different structural components, such as the connections that are involved.

In Paper II (Klasson *et al.*, 2018a), a rather simplistic yet practical model of bracing performance with slip was employed. It was assumed that the bracing stiffness is at the zero level until a full slip in it has occurred. More accurate models of slip might assume, for example, there to be a smoother transition from zero stiffness to full stiffness than might otherwise be conceived. It should be emphasised, however, that it would be rather complicated to design a structure while taking account of the effects of potential slip if the slip model were to be very accurate. Also, the uncertainties involved probably do not justify overly complicated and accurate models, in particular for designing purposes.

It should be emphasized that the possible existence of slip in different bracing systems along with its magnitude are not known in any general sense. Full-scale testing of the bracing systems of real structures and/or of their different structural components would be required to learn more about these matters.

It is possible, however, to make different engineering assumptions in order to evaluate the probability of slip in a specific bracing system. A discussion of three important potential sources of slip, and/or of very low stiffness at the beginning of the loading of a bracing member, is provided in the subsequent sections.

Slip found in connections

The connections involved, especially bolted ones, are very likely an important potential source of slip in the purlins of roof structures, for example. Slip and consolidation in timber connections of this type have been detected in laboratory tests, e.g by Dorn *et al.* (2013).

The slip in question may be related, for example, to the presence of intentionally oversized holes. For ease of construction, the width of holes for bolts (and dowels) usually needs to be about $2mm$ larger than the diameter of the bolts. This holds for both timber and steel structures. Given this ratio of the bolts to the hole dimensions involved, the slip would be between 0 and $2mm$, depending upon the position of the bolt at the moment of application of the load; see Figure 3.19. If more than one connection in a row is involved, the total slip would be the sum of the degree of slip occurring in each of the connections.

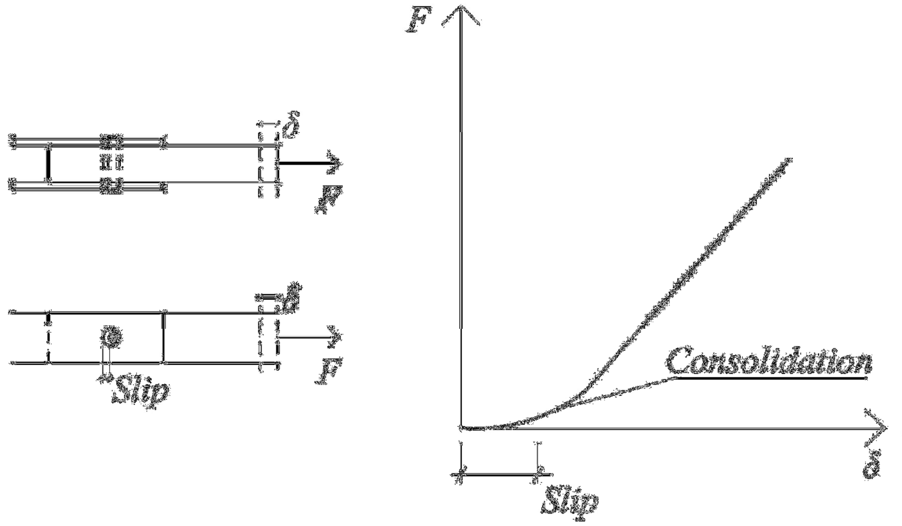


Figure 3.19: Potential slip in bolted connections due to the holes being oversized.

Sagging of members in the stabilizing systems

Another type of slip could be that due to the potential sag in the steel rods commonly used as diagonals in the stabilization of wall sections or as diagonals in the wind trusses of the roof plane of a structure. Due to their self-weight, along with their very low bending stiffness, the steel rods sag - the degree of sag being greater as the length of the rod increases.

In order for a stabilizing bay of this type to immediately resist lateral loading, the steel diagonal rods would need to be slightly prestressed. However, due to effects such as those of the relaxation of the prestressed steel members and/or the creep in the timber members, the prestress may become significantly reduced after a sufficient period of time has passed. Thus, a certain displacement, "i.e. a slip" in the structure, would be needed to straighten the sagging steel rod diagonal out before it can resist loading as intended. This can be illustrated by the following example (Figure 3.20):

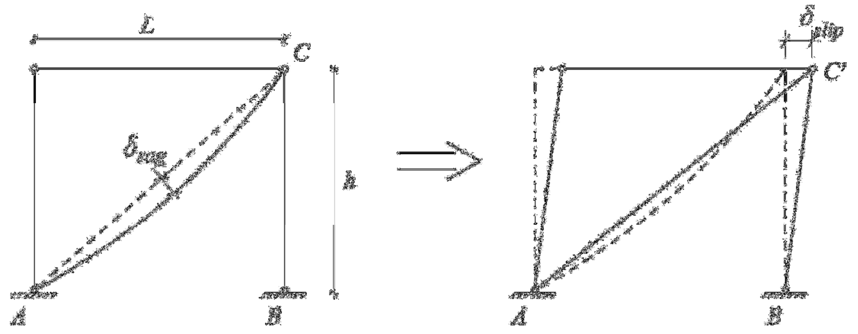


Figure 3.20: An illustration of how a sagging diagonal needs to be straightened our before it can resist a lateral load.

Assume for simplicity's sake that the sagging diagonal in Figure 3.20 has a parabolic shape, such as shown in Figure 3.21.

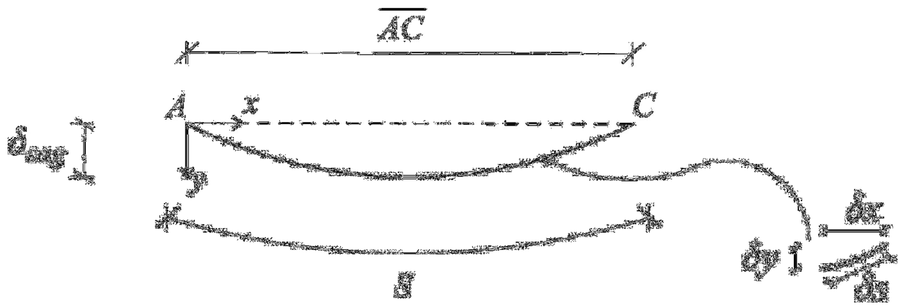


Figure 3.21: The parabolic shape of a sagging diagonal.

The shape of the parabola $y(x)$ is given by the following formula (Equation 3.12):

$$y(x) = \frac{4\delta_{sag}}{L^2}(x^2 - Lx) \tag{3.12}$$

The length (S) of the parabola shown in Figure 3.21 can then be calculated in accordance with Equations 3.13-3.17, as follows.

First, note that the length of a segment of a cable of infinitesimal size, δs (see Figure 3.21), can be solved by the Pythagorean relationship it has, in accordance with Equation 3.13:

$$\delta s = \sqrt{\delta x^2 + \delta y^2} \tag{3.13}$$

Equation 3.12 can be rewritten in the following way (Equation 3.14):

$$\frac{\delta s}{\delta x} = \sqrt{1 + (\frac{\delta y}{\delta x})^2} \tag{3.14}$$

where the $\delta y/\delta x$ term is the derivative (y') of the shape function of the parabola that is defined in Equation 3.12. This means that the length of the cable, S, can be calculated on the basis of the following integral (Equation 3.15):

$$S = \int_0^L \frac{\delta s}{\delta x}\delta x = \int_0^L (\sqrt{1 + (\frac{\delta y}{\delta x})^2})\delta x \tag{3.15}$$

Assuming then that the displacements involved are small, Equation 3.15 can be simplified effectively by use of the Taylor expansion of the expression (Råde & Westergren, 2004), only the first term of the series in question being included; see Equation 3.16.

$$S = \int_0^L (\sqrt{1 + (\frac{\delta y}{\delta x})^2})\delta x \approx \int_0^L (1 + \frac{1}{2}(\frac{\delta y}{\delta x})^2)\delta x \tag{3.16}$$

Noting that $\frac{\delta y}{\delta x} = y' = \frac{4f}{L^2}(2x - L)$ and solving the integral of Equation 3.16, the length S of the parabola will, according to Equation 3.17, be:

$$S = L(1 + \frac{8}{3}(\frac{\delta_{sag}}{L})^2) \tag{3.17}$$

Using an approach that is similar to ignoring the effects of small angular changes (i.e. ignoring vertical movements at node C), the horisontal movement (δ_{slip}) required at the top of the frame (at node C) in order to just straighten the sagging diagonal out so that $\overline{AC'} = S$ (see Figure 3.20) would be as follows; see Equations 3.18 and 3.19.

$$\delta_{slip} = \sqrt{(S^2 - h^2)} - L \tag{3.18}$$

$$\frac{\delta_{slip}}{\delta_{sag}} = 2\frac{\sqrt{\frac{2}{3}}(\sqrt{S^2 - h^2} - L)}{\sqrt{-L^2 + LS}} \tag{3.19}$$

Plots of the normalized slip, in accordance with Equation 3.19 ($\delta_{slip}/\delta_{sag}$), shown in relation to the ratio of the sag to the distance between nodes A and C of the undisplaced structure ($\delta_{sag}/\overline{AC}$), for each of three different ratios of L to h, are provided in Figure 3.22.

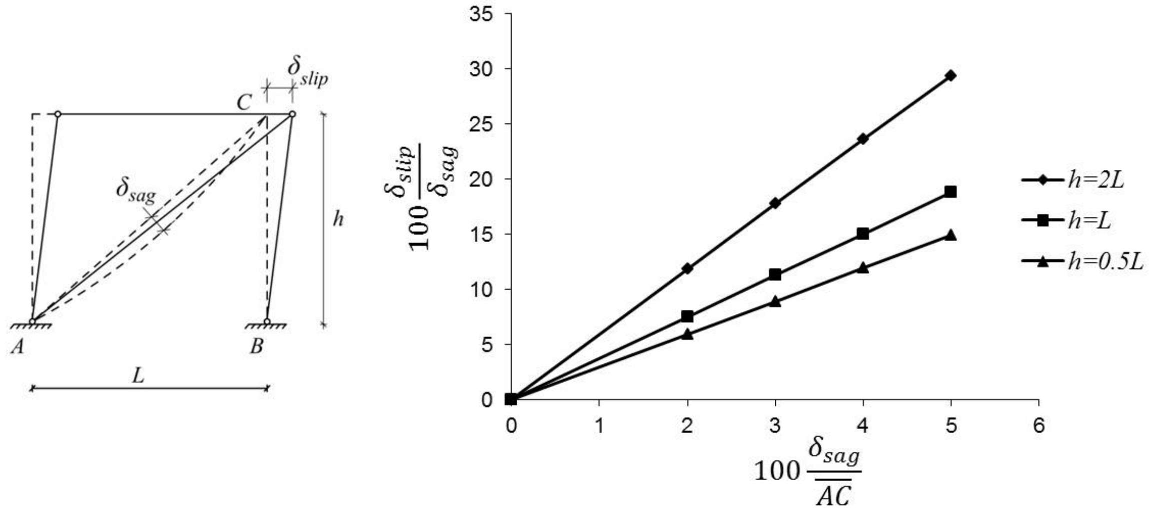


Figure 3.22: An example of how the sag of a diagonal member would require that a movement (δ_{slip}) at the top of the frame take place in order for the diagonal to be activated (i.e. straigthened out).

Conclusions that can be drawn from the plot in Figure 3.22 concerning the effects of potential sag of the diagonals in the bracing systems include 1) geometries in which $h > L$ is more sensitive to potential sag than geometries in which $h < L$, and 2) the potential horizontal slip due to the sag in question is generally small as compared with the sag itself, i.e. for reasonable magnitudes of the sag involved.

For example, if $h = 2L = 12m$ and the sag, δ_{sag}, were 50 mm in size, i.e. $\approx \overline{AC}/250$, the potential slip at the top of the frame would be about $1.3mm$. In practical terms, this obviously means that the expected effects on a wall section that a slip of this kind would normally have would be rather small.

However, a sagging of the steel rods used as diagonal members in horizontal bracing units, would also lead to slip. If the slips of all the diagonals of both the horizontal and the vertical stabilizing system of a structure are summed up, the total magnitude of the slip thus obtained might have a non-negligible effect on the bracing performance.

The crookedness of bracing members

Another source of slip, or rather of an initial movement of this sort that would occur in connection with a reduced degree of stiffness, could be the potential initial crookedness of the secondary members (e.g. purlins) that transfer horizontal loads through axial action. In principle, this means that such secondary members would need to straighten out a bit before they would be able to resist loading by showing the intended degree of stiffness; see Figure 3.23. It is assumed that the bending stiffness of such secondary members is very small in their out-of-plane direction. This phenomenon can be compared to the action of a cable (which has no bending stiffness), the axial stiffness of which is very small when it is slack.

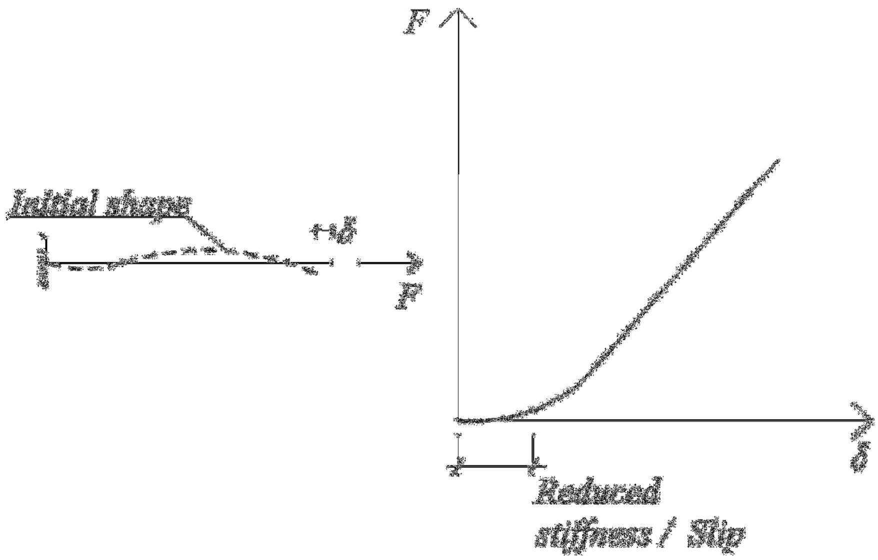


Figure 3.23: An illustration of how the initial crookedness of a bracing member, having a low degree of bending stiffness in its out-of-plane direction, can result in a reduced initial bracing stiffness (or slip).

How large this effect might be for slender roof structures was not investigated here, but the deviation-from-straightness of construction wood members is usually considered to be in the range of $L/250 - L/500$.

Assuming then that the crooked shape of a bracing member situated between two adjacent primary members (i.e. located in one bay) of a roof structure is parabolic in shape, the slip needed then to straighten it out can be calculated in the same manner as was demonstrated above for the case of the potential sag of members located within the stabilizing system. In such a case, the slip would simply be the initial difference between the length of the parabola and the straight distance between two adjacent primary members. As an example, if the distance between the restraints of that member were $6m$ at the same time as the deviation-from-straightness of that same member was $L/250$, then the "slip" would be $0.3mm$. Although this is only small in size, it could become non-negligible when summed together with the slip contributions of all the bays of the structure.

4 Full-scale testing of roof-bracing systems

As described in the previous sections, the designing of slender structures usually involves various uncertainties. One important way of reducing these uncertainties is to collect data, such as by means of different laboratory tests of structures and of structural members.

Below, a general discussion of the testing of structures is provided. In Section 4.1, a method of determining the lateral stiffness of slender roof structures, one that was developed as a part of the present project, is described. In Section 4.2 a test rig that can be used for both field testing and laboratory testing of the horizontal stiffness of roof structures is likewise described. Finally, in Section 4.3 the laboratory testing of bracing systems commonly used for timber structures that are reported on in Paper IV (Klasson & Crocetti, 2018) is discussed.

Full-scale testing can be conducted either on existing structures (field testing) or on structures built for the specific purpose of testing them in laboratory controlled environments.

An advantage of laboratory testing is that important parameters, such as those of boundary conditions and of climate, can be more precisely defined and more easily controlled. Also, the loading and the measuring of displacements and strains, for example, are normally more precise in laboratory environments. Another potential advantage of laboratory testing as opposed to field testing of structures, if these are to remain in service after testing has been completed, is the possibility of readily carrying out destructive testing by loading the structure of interest until failure occurs.

The following set of points provides an overview of what is usually meant in referring to the full-scale testing of a structure in a laboratory environment:

- All the different types of members and details of interest, such as primary load-bearing members, connections, and bracing members that would be of central interest in a corresponding real-life-type structure are represented in the full-scale model employed.

- The dimensions of the structures and the members that are being tested in the laboratory are realistic in terms of what one would expect to find in real structures.
- The dimensions of the structural members that are tested in the laboratory are realistic in terms of the loading that would be present in the case of real structures corresponding to these, the very same spans, heights or loads (that are defined by different design codes) being involved.

Commonly, however, laboratory tests are restricted for practical reasons to structures of reduced scale or to the testing of isolated structural members or components of the structure of interest, such as different structural connections, for example. Less common are laboratory tests in which the entire building involved is represented, despite numerous full-scale tests having indeed been performed over the years, common examples of these being ”shaking table” tests aimed at evaluating the seismic performance of different structures.

As mentioned earlier, one advantage of tests carried out on isolated structural members is the more limited degree of uncertainty involved, i.e. that most circumstances of interest, such as loading and boundary conditions, for example, are quite well defined; see for example the torsionally rigid ”fork supports” used for the stability testing of a timber beam, as shown in Figure 4.1. The theoretical assumptions applying to ”fork” supports (e.g. restrained rotation around the longitudinal axis and free warping at the supports (Sundström, 1995) obviously resemble rather closely most approaches to classical beam theory, making the results obtained very easy to compare with corresponding analytical models, despite such well-defined boundary conditions obviously not commonly being found in real structures.

Thus, the results of testing isolated structural members are hopefully quite straightforward to interpret. A disadvantage, however, is that the boundary conditions of the members would potentially be quite different from what would be expected if these where members of a real structure. On the other hand, a given type of structural member or component might be used in various structures and applications of rather differing character, this meaning in a general sense that the results might be more useful at a component level than for full-scale testing of a specific structure in its entirety, the results of the full-scale testing of structures obviously being more difficult to generalize.

Figure 4.1: Stability testing of a simply supported and discretely braced timber beam subjected to four-point bending (Klasson *et al.*, 2014).

4.1 Non-destructive testing of the bracing system of slender roof structures

Despite the issues involved in the full-scale testing of real structures, as indicated in the sections above, the testing procedure used for the testing reported on in the thesis and in Paper IV (Klasson & Crocetti, 2018), described further in this section, was originally developed to be used for the non-destructive field testing of real and existing roof structures, both in the case of partially finished and of fully completed buildings[1]. The main purpose of this was to find out how stiff real roof structures, including all types of secondary details that are possible can be expected to be, for this, full-scale testing being the only reasonable option. The more specific aims pursued include the following:

- To develop a method to be used to ensure adequate bracing stiffness during the construction phase in the erection of new buildings. The results of the testing could potentially be returned to the engineers for their approval in the finalizing of the building; i.e. for the engineers to compare the actual stiffness values obtained with their own design assumptions.

[1]Unfortunately, due to unclear terms of insurance and difficulties in finding suitable objects, a full-scale laboratory test was conducted instead.

- If finite element models are used in the designing of a structure, the results of the testing carried out can be used to update the model by means of so-called finite element updating approaches; where finite element model updating refers simply to the process of ensuring that the results of the finite element analysis reflect the measured data better than the original model did (Friswell & Mottershead, 2013), such approaches being widely used for tasks of differing levels of complexity, e.g. in the modal analysis of structures (Girardi *et al.*, 2018) and in the static loading testing of bridges (Wu *et al.*, 2017).

- To learn more regarding the expected stiffness of common bracing systems used in connection with slender roof structures in a general sense, both for finalized and for partially completed buildings. The stiffness contributions of secondary structural detailing that is of potential importance, such as external cladding, which is normally not accounted for in the design of structures or in laboratory tests, would be included in the results.

- To obtain valuable information at a conceptual level regarding the performance of bracing systems that can be used to validate the results of the advanced numerical modelling of structures in a general sense.

The non-destructive testing method concerning the bracing stiffness of slender roof structures that was developed as a part of the present research project, and was used for the tests reported on in Paper IV, can be described as follows:

The method had to be non-destructive since it was originally developed to be used in connection with existing structures, structures that were to remain fully functional after the testing that was carried out had been completed. This means that the testing can only be carried out in the assumed linear elastic range of the structure (and of the structural members) for the test loading in question; any deformations caused by the testing should be fully reversable at unloading.

In principle, the method aims at determining the translational stiffness, in the direction parallel to the applied force at a point in the plane of the roof structure, more specifically at an arbitrary point along one of its purlins, or at different points in the lateral direction of the primary members.

By use of a special testing rig (described in greater detail further on), a horizontal point load is applied at a desired point along one of the purlins of the roof structure. In order to determine the stiffness at the point in question, both the force and the displacement need to be monitored.

It is important that the measured displacements are independent of the testing rig. In the present investigation a separate apparatus was employed in order to sustain use of the measurement instruments.

It is also important to avoid the effects of potential local deformations that can occur at the load application point, in particular in the case of timber members. For this purpose, an offset of approximately $100mm$ between the load application point and the point at which the axial displacement is measured was adopted in the present investigation; see Figure 4.2.

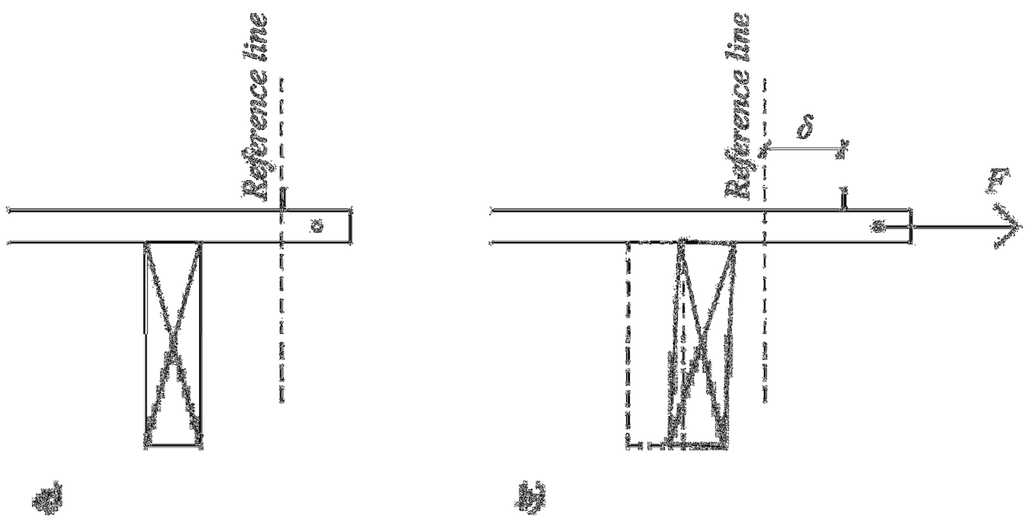


Figure 4.2: Principles concerning how to determine the lateral stiffness at a specific point in a roof structure in accordance with the method proposed, where a) is at prior application of the load, and b) is at loading. The stiffness would be $k = \frac{F}{\delta}$.

The number of points that would need to be tested for a given roof structure in order to adequately evaluate the stiffness of its bracing system depends upon the number of uncertain (or unknown) parameters involved.

For instance, the bracing stiffness may vary significantly along the span of the braced members. The position of the point of application of the horizontal load "F" in the longitudinal direction of a purlin, i.e. the distance from the stabilizing bay at which the forces would be transferred to the ground, can also be expected to affect the stiffness. This is due both to the reduction in axial stiffness that occurs when the length of members is increased and (perhaps to a greater extent) to the slip that occurs at each connection at a given member.

Also, the direction of loading can have an effect on the stiffness, due e.g. to the purlins and all their potential connections being loaded either in compression or in tension.

The following example[2] can provide valuable information to be used by structural engineers for adequate FE-updating of a roof structure. It should be noted, however, that in order to be able to determine the stiffness of the different parts of the roof structure more specifically, the exact stiffness of different connections would need to be known and some additional testing would probably be necessary as well.

The example involved is the following:

A slender steel structure (span > $30m$), built up of steel trusses that are supported on steel columns (see Figure 4.4) is braced by means of wind trusses in the plane of the roof, as shown in Figure 4.3. Purlins are placed on top of the trusses in order to connect the different members to the wind trusses and to support the roof cladding. The horizontal forces directed at the wind trusses in the roof plane are transferred to the ground by vertical bracing systems (cross bracings) located in the walls, as shown in Figure 4.5.

In order to determine how the bracing stiffness varies along the span of the trusses, testing needs to be carried out at two points at least. However, in order to know whether the variation in bracing stiffness along one of the primary members is linear or non-linear, testing needs to be carried out at three different points at least, e.g. at points 1, 4 and 7 in Figure 4.3. Similarly, in order to determine how the stiffness varies as one moves away from the wind truss, additional points along the same purlins at a different distance from the wind truss would need to be tested, e.g. points 2, 5, 8, 3, 6, and 9 in Figure 4.3. It is also possible, in order to obtain more data, to reverse the loading, meaning that each point is being both ”pulled” and ”pushed”.

[2] A Polish project that was reviewed by the author was used here.

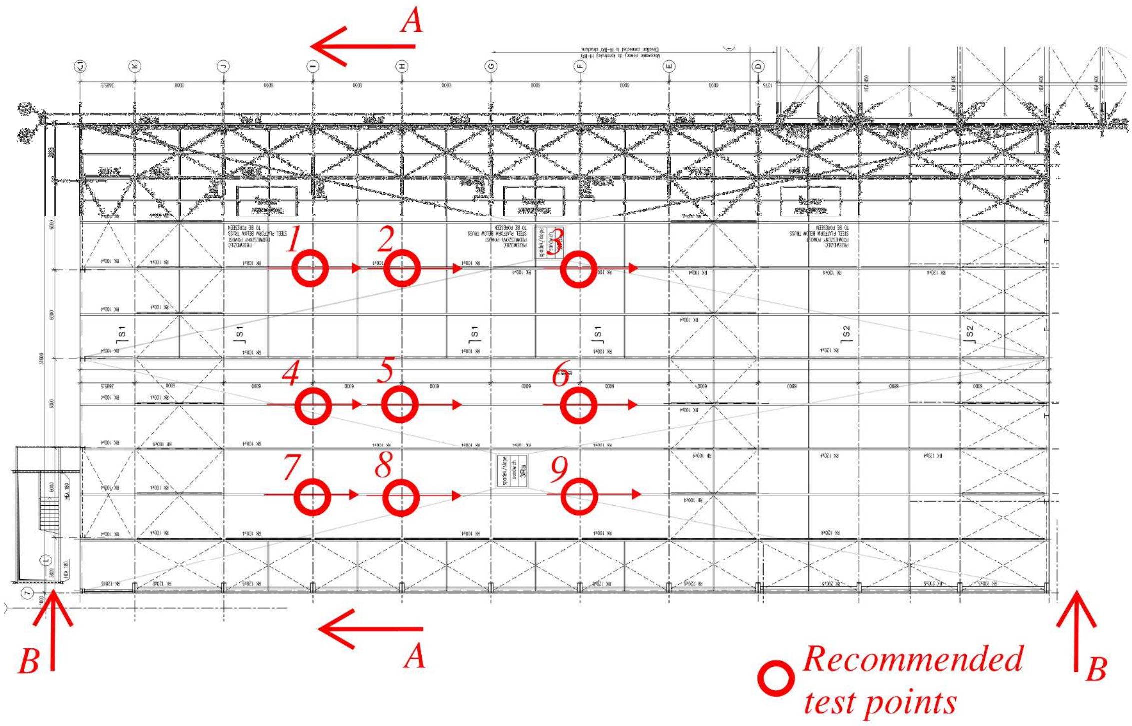


Figure 4.3: Examples of recommended testing points in a slender roof structure, plane view.

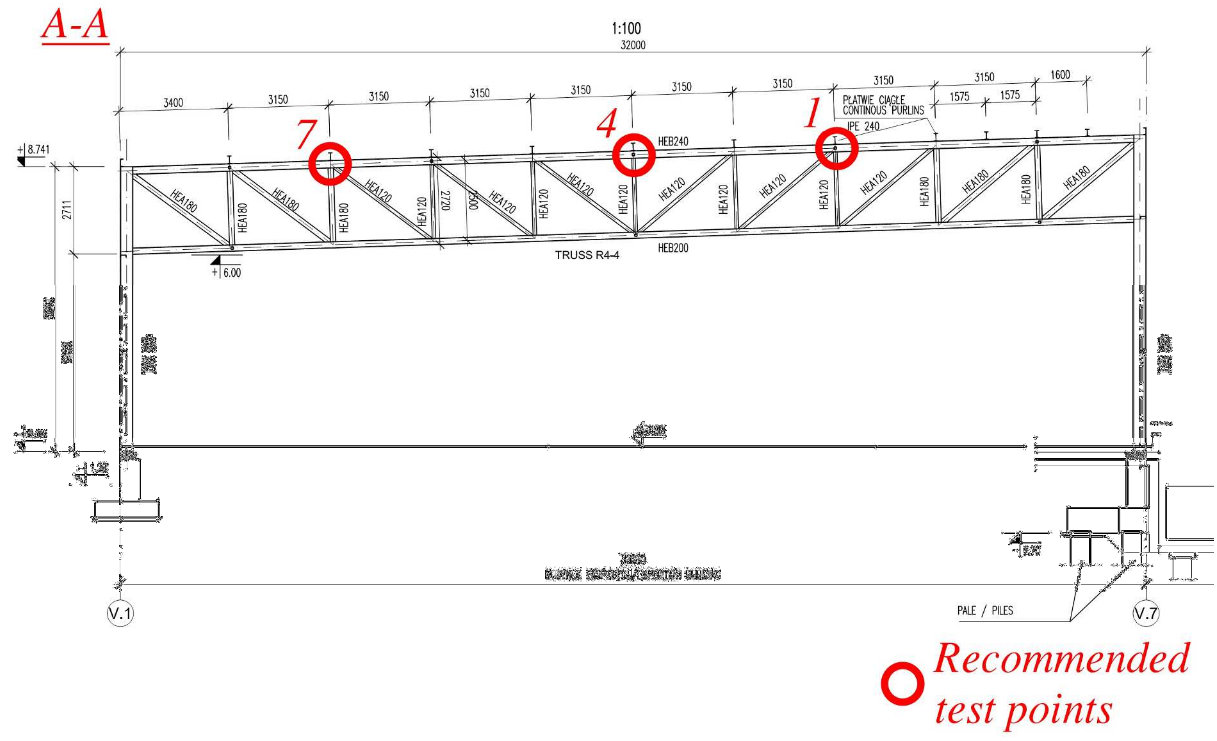


Figure 4.4: Examples of recommended test points in a slender roof structure, at cross section A-A (see Figure 4.3).

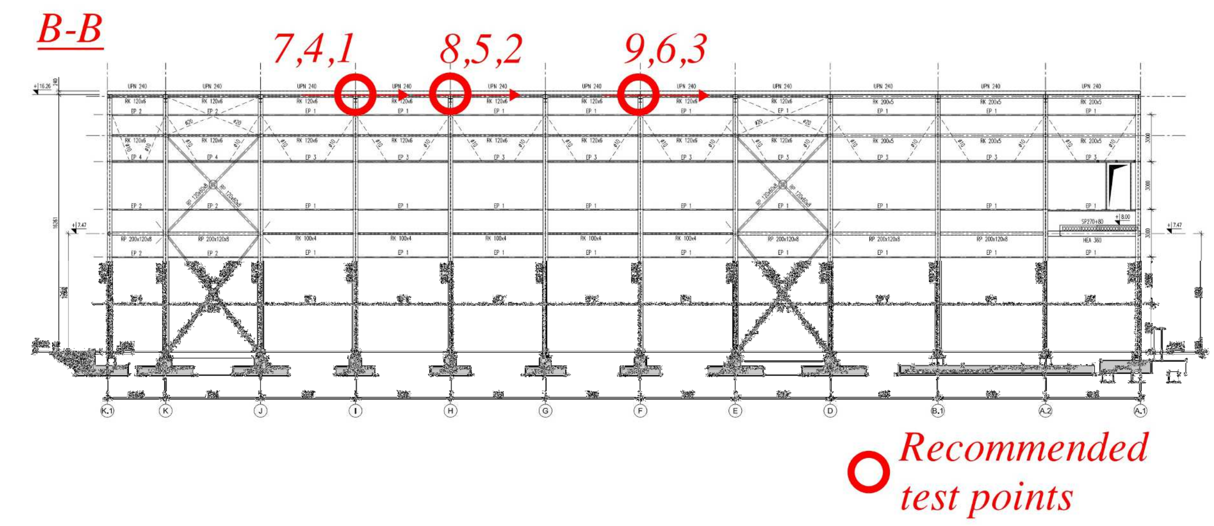


Figure 4.5: Examples of recommended testing points in a slender roof structure, at cross section B-B (see Figure 4.3).

4.2 The test rig

In order to be able to perform the tests of the type described in the sections above, a special test rig that can be used to apply a horizontal point load at the level of the roof plane was developed.

The test rig, shown in Figures 4.6 and 4.7, can be described further as follows:

- Essentially, the test rig consists of a steel column having wheel bearings at the top of it and a horizontal frame at its base that supports the column and the loading equipment (hydraulic jack).
- A cable that extends over the wheel bearings that are located at the top of the column of the test rig is used to apply the loading to the structure.
- The loading of the cable is produced by an hydraulic jack that pushes a ”sledge” forward, the sledge being attached to the cable, which extends over the column of the test rig.
- The force involved is measured at the top, by use of a load cell that is attached to the cable, to the right of the column shown in Figure 4.6. This is so that any possible frictional effects along the cable (e.g. at the point of the wheel bearings) will not affect the results of interest, in terms of the force applied to the structure being overestimated.
- In order to stabilize the column of the test rig, a back stay (bracing) is attached to the top of the column at one of its two ends and to the base frame at its other end.
- In order to prevent the entire test rig from tilting when a load is applied, a counterweight is placed on the back of the test rig.
- In addition, in order to enable the test rig to be used for field testing of the roof structures, the test rig is designed so as to be placed (and used) on the backside of a lorry.
- In order to make the rig adjustable, and thus adequate for testing structures of differing heights, the column is constructed in a telescopic manner.
- The displacement transducer is placed on a separate structure so that deformations of the test rig do not affect the measurements (as was described earlier).

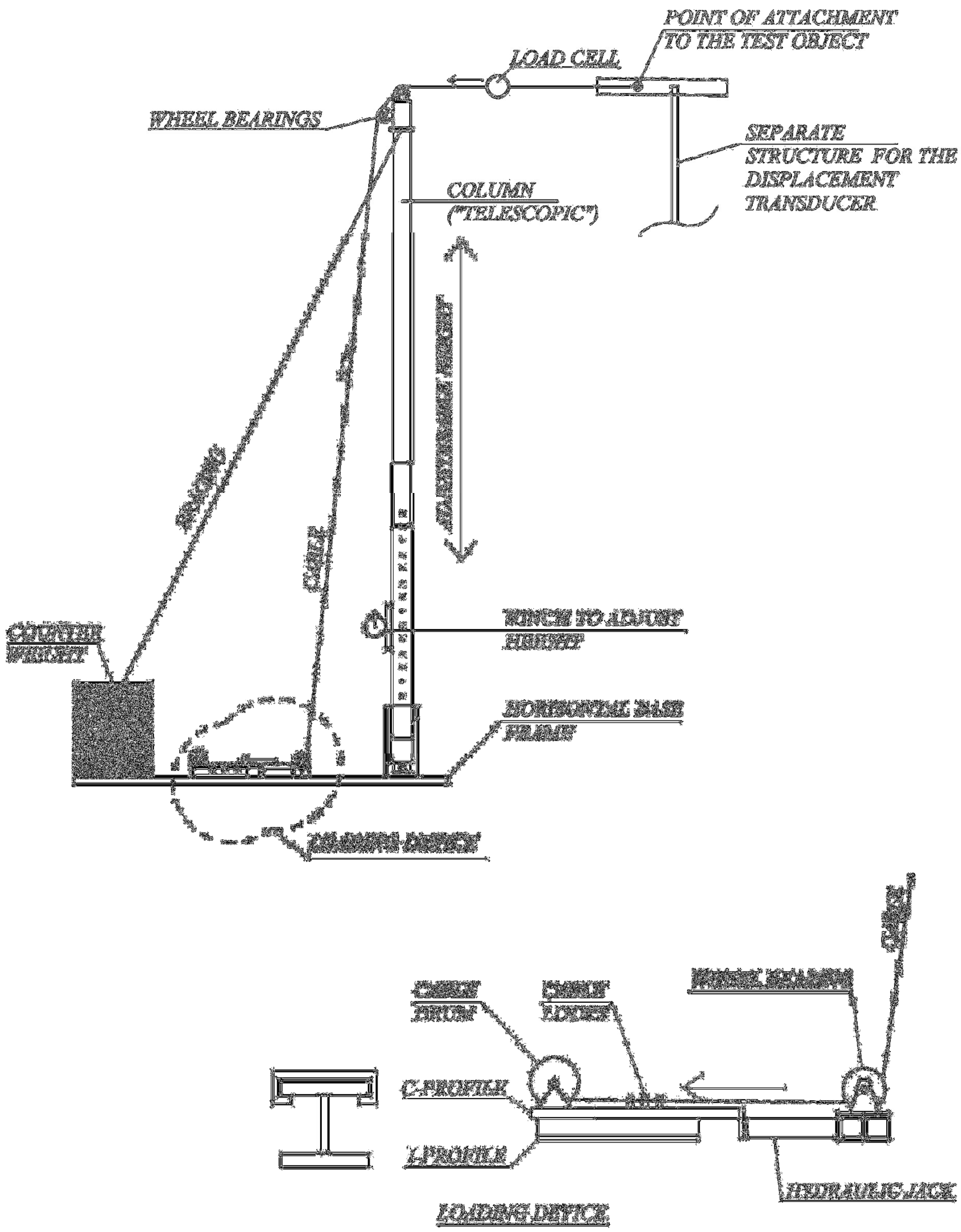
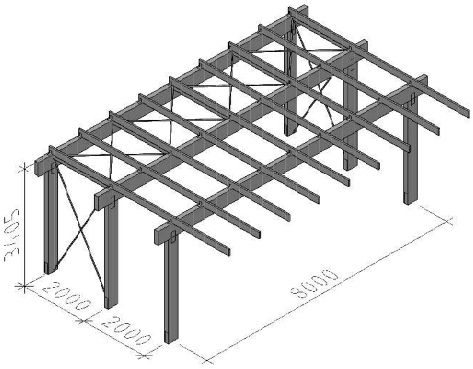


Figure 4.6: A drawing of the test rig.

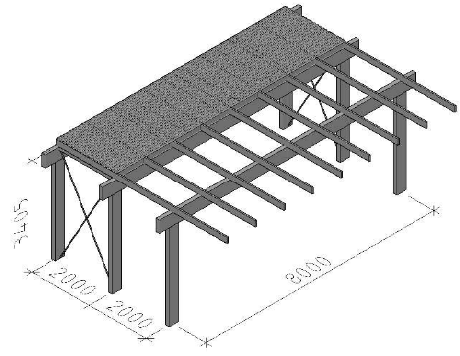
Figure 4.7: A picture of the test rig.

4.3 Laboratory tests of bracing systems commonly used for timber structures

For the purpose both of putting the method described in the previous sections to test, and learning more regarding the stiffness of common roof bracing systems, a full-scale laboratory test of the method was conducted. The structure involved was of a very basic type, two different bracing methods being employed, those of 1) using a wind truss in the plane of the roof for stabilization purposes, and 2) using a steel decking on top of the roof, likewise for purposes of stabilization; see Figures 4.8a and 4.8b, respectively.



(a) A wind truss in the plane of the roof.



(b) A corrugated steel sheet decking in the plane of the roof.

Figure 4.8: Models tested in the laboratory.

The test rig and the methods in question turned out to work well, the results obtained being used for the successful finite element model updating of a corresponding numerical model.

Major findings of the study include the following:

- The stiffness of the connections could be readily evaluated.

  In general, connections that transfer load through compression perpendicular to the grain can be expected to significantly reduce the stiffness of the bracing systems of wood structures.

  The shearing connections (angle bracket connections) between the purlins and the primary beams, on the other hand, were rather stiff, if a sufficient number of screws was employed.

- The effects of using differing numbers of fasteners in connections could be clearly detected. When an adequate number of screws were used in the shearing connections of the purlins, the connection obtained matched closely the FE-results of a model based on use of rigid connections.

- The stiffness of two different bracing systems - wind trusses and steel sheet decking - could be evaluated properly.

  The two systems were about equally stiff in terms of the horizontal displacement produced by a given horizontal force, as measured in the longitudinal direction of the purlins.

The stiffness provided by the steel decking was highly dependent, however, upon the number of fasteners used along its edges to connect it to the purlins of the timber structure.

The test results are described further in Paper IV.

The results obtained for a specific test are presented here in the form of force-displacement curves; see Figure 4.9. Some of the main observations that are made include the following: 1) that the behaviour is essentially linear elastic in the range of the horizontal load that was applied to the structure, 2) that the stiffness for loading and that for unloading are basically the same, and 3) that the curve for unloading lies below that for loading, the difference in the level of the two curves providing an indication of the friction that takes place in the connections within the structure, this representing the energy that is dissipated by the structure during one loading-unloading cycle.

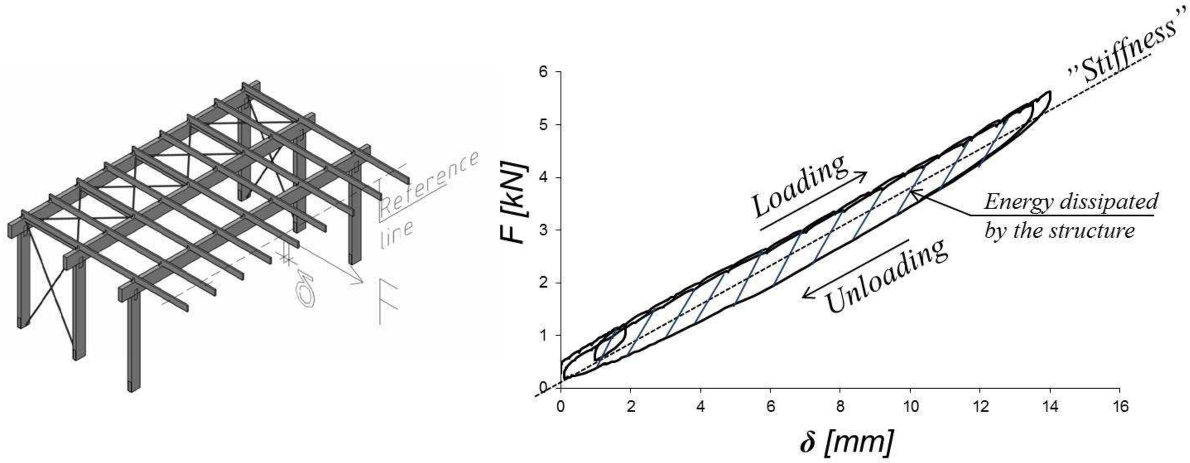


Figure 4.9: The load-displacement curves obtained in one of the tests.

5 How to improve safety of slender structures

Primarily, this section is based on the experience the author gained in working as a structural engineer for a period of about 10 years, together with input received from the supervisors of the thesis as well as from a reference group consisting of 5-10 experienced structural engineers, together with information obtained in the survey reported on in Paper III (Klasson *et al.*, 2018b). The specific aim here is to provide an overview of the results of the survey of structural engineers that was conducted (reported on in Section 5.1) and to provide a proposal of how best to deal with the design and creation of slender structures in a manner serving to improve the safety of those making use of such structures (Section 5.2).

5.1 A survey of the views of experienced structural engineers

The failure of slender structures, when and if it occurs, can be due to any of several different factors. Section 2 describes different ways in which the failure of slender structures can occur, for example, as instability of the structures and/or erroneous design work involving mistakes made by engineers.

In Section 3, different uncertainties involved in the design of slender structures were discussed, uncertainties that force a structural engineer to make subjective choices that may be erroneous in the designing of slender structures. Previous studies have indicated that structural engineers differ markedly in the designing work they carry out (Fröderberg, 2014; Fröderberg & Thelandersson, 2015). It is obvious that designing work involving subjective choices is particularly likely to be nonconforming.

For the sake of improving structural safety, further research is needed in efforts to identify the most critical issues in connection with the designing and production of slender structures. One way to do this is to survey experienced structural engineers concerning their approach and their experience in the designing of slender structures. Thus, a survey in this regard of experienced structural engineers was conducted as a part of the thesis.

It was decided that the survey should involve the use of an anonymous questionnaire. The questionnaire consisted of nine questions altogether. The first five were quite general in character and aimed at providing an indication of what the engineers perceived as being important sources of error in the designing and constructing of slender structures generally. Questions 6-9 were more specific and were intended to disclose possible differences between participants in their views regarding different assumptions concerning the design of slender roof structures. The original questionnaire is presented in the Appendix at the end of the thesis.

A mixture of closed and open-ended questions was employed in the survey. Questions of a closed character typically provide a set of quick answers to choose between, whereas open-ended questions usually require a lengthier response, e.g. one in writing. According to Boynton & Greenhalgh (2004), both formats have their advantages. The particular advantage of open-ended questions is the potential they have of capturing opinions and data possibly not thought of in earlier studies, participants being more free to express their own opinions. A major disadvantage of open-ended questions, on the other hand, is that the results obtained may be cumbersome to interpret. A closed format enables the researcher to obtain aggregated data quickly. Also, in some of the closed-format questions used in the study, participants were invited to motivate their choices. Some questions also encouraged participants to explain their answers by means of structural sketches.

Altogether, there were 17 experienced structural engineers from 6 different nations who participated in the survey. Their average work-time experience was one of 20 years.

Major findings of the survey include the following results:

1. According to 7 of the 17 participants, the main cause of roof failures appears to be erroneous design calculations.

2. According to 6 of the 17 participants, a lack of communication with others during the design process is a major source of error in the designing of slender structures.

3. Nearly half of the participants (8 out of the 17) indicated that improved building codes and a more thorough third-party review would be useful in improving the safety of slender structures. They did not specify any details, however, concerning how best to improve the codes.

4. Questions concerning design assumptions revealed a relatively large variation between the participants.

5. Three of the participants (18%) chose a non-conservative buckling length of a braced beam.

6. Two of the participants proposed theoretically unstable structural solutions to the designing of roof structures.

In an overall sense, the survey indicates that experienced structural engineers are open to a more thorough third-party check of projects being carried out as a means of detecting errors; and that even experienced structural engineers may make inadequate design assumptions. More regarding the survey is reported in Paper III (Klasson *et al.*, 2018b).

5.2 Design and construction processes

The process of the structural designing of a building is generally a rather complex activity involving far more than simply the verification of its having an adequate load-bearing capacity. The structural engineer is usually part of a team that includes project leaders, clients, architects, other engineers, landscapers, construction teams and responsible authorities (Tunstall, 2006).

For several reasons, structural designing is best described as an iterative process, in particular when more than one discipline is involved in a detailed design. The designer of the foundations needs updated loads from the designer of the superstructure, the designers of the superstructure need to update their model with the latest input regarding the foundations, for example, and so forth. In addition, requirements stemming from other disciplines may change during the designing phase of a building and thus make initial design assumptions obsolete.

Among other things, the survey of experienced structural engineers reported on in Paper III clearly indicates the importance of proper communication between different partners and a thorough third-party check within a project for purposes of ensuring an adequate design. Many of the engineers taking part in the survey believed that erroneous design calculations are the main cause of roof failures; one promising solution is thorough review, preferably by an external and independent party, as the most important tool for detecting such errors. Also, when subjected to review, the engineer is forced to describe the overall design and the different design assumptions in a stringent way in order to convince the reviewer of the adequateness of the design. Through this process, it is likely that the designer can detect potential errors even prior to the review.

In some cases, when the design assumptions are very uncertain and are critical for the performance of the building, non-destructive testing (e.g. the testing of bracing systems described in Section 4) of the building during construction can be an alternative in order to ensure that the design assumptions match the actual building.

Obviously, construction errors may also be responsible for building failures, such as in cases in which the structure that has been built differs in some way from the original, intended design of it. Construction companies may not always fully understand the intended design of a structure or may perhaps choose simpler solutions that may or may not be completely adequate. It is recommended that representatives of the construction companies involved in a project take an active part in the detailed designing process. This is important for two reasons, 1) that of making sure that the design teams understand important aspects of construction and how different construction methods may affect the design assumptions, and 2) making sure that the construction teams are fully aware of the design requirements in a general sense.

Figure 5.1 provides an overview of the process of the designing and the construction of buildings. It is based on a traditional way of distinguishing between different stages of a construction project but also includes important steps to ensure that the design is both adequate and safe, in accordance with the discussion of this matter above. The procedure of external review and the step-by-step approval of the execution of a building project can be clearly justified for large structures that are sensitive to the design assumptions involved and in which the costs of different procedures in the testing of it would represent only a very small part of the total costs of the project; for minor projects these actions are possibly not required/justified. Points 1-4 in the figure can be further described as follows:

1. The detailed design process is usually based on a conceptual or preliminary design of the structure. In the conceptual design phase, multiple design alternatives are usually evaluated. The solutions arrived at in the conceptual-design phase are typically not particularly detailed and may only include very brief descriptions of the overall structural conception of the building project.

   As explained earlier, a detailed designing procedure should be regarded as being an iterative process. When the design approaches its finalization, it is time for an external review.

2. Regarding the external review, if the review does not result in approval, the project is returned to the designers of it to be revised. This can in many cases be an iterative process. When the design is finally approved, the reviewers can approve construction starting.

3. The construction phase can also be seen as an iterative process, one in which step-wise approval and interaction with the design teams is required for it to continue.

4. In cases in which testing or measurements are used to validate different design assumptions as the construction process evolves, construction should stop and wait for the designers to either approve of the results or require that changes be made.

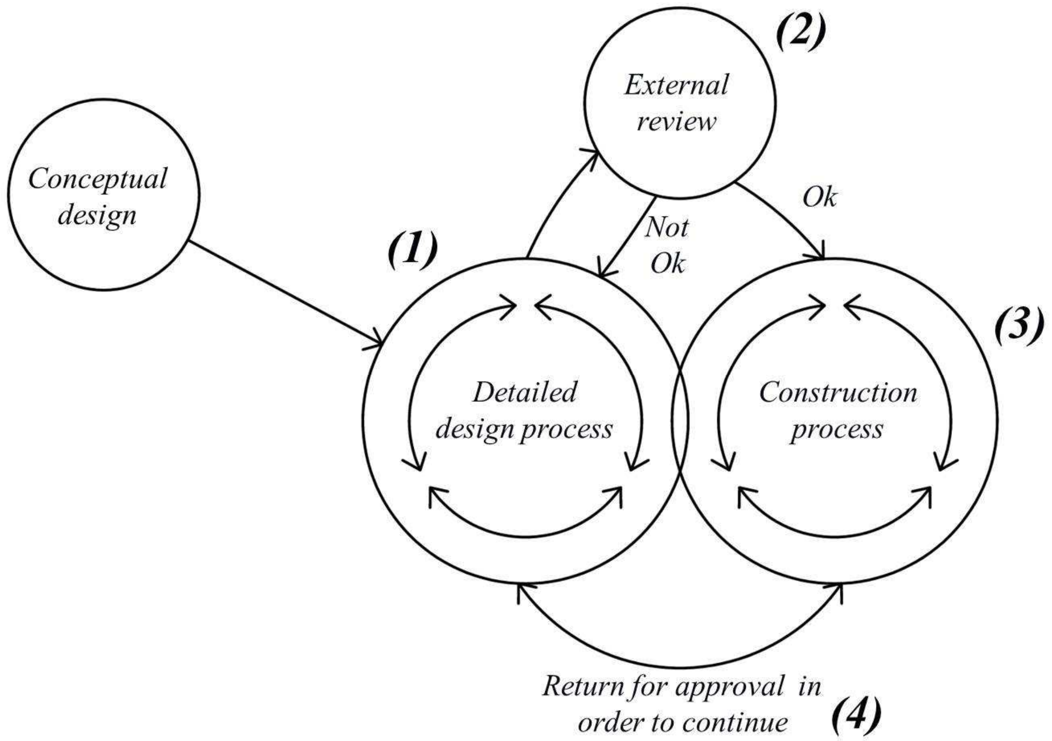


Figure 5.1: The process of designing and constructing a building, as envisioned by the author.

6 Conclusions

The conclusions listed here represent the findings both of the appended papers and of the thesis itself. The conclusions are related to the objectives of the thesis, which are listed in Section 1.2 and follow in the same order of appearance.

- Many failures of slender structures that occurred in the past were related to design errors and were not due to extreme loading.

  Similar design errors of roof structures that failed due to snow loading can be seen in failure cases from the 1970s as in cases of failure more recently involving roof structures built after 1980. This appears to indicate a poor dissemination of failure lessons in a general sense.

  Several failure investigations point to the need for proper reporting of failure cases so as to avoid errors being repeated. Failure information should preferably be made available in public databases or at least in databases that can easily be accessed by professional structural engineers.

  External third party review of design documentation is commonly considered highly important to be able to detect possible erroneous design.

- The designing of slender structures can in many cases be rather uncertain, engineers commonly needing to make subjective assumptions in order to be able to model them.

  For instance, in the modelling of slender structural members, the imperfect shape of a braced structural member has been shown to have a strong effect on both the predicted capacity of the member and the bracing forces involved. Imperfection shapes that are too simplistic may well overestimate the capacity of braced members and underestimate the forces occurring in its bracings.

  The imperfection shape of a structural member is generally of a random nature, i.e. not fully known. Thus, engineers need to use safe assumptions when modelling slender structures. However, this can be quite cumbersome; the imperfect shape of a braced member that leads to the strongest bracing forces is generally not the same as the shape that is most critical for the estimated capacity of the member itself. Thus, more than one imperfect shape usually needs to be considered when modelling braced structural members.

- It has been shown that slip in bracing systems can have a significant effect on the capacity of the braced members involved, due both to the stresses in the members as well as the bracing forces being greater.

  Slip in bracing systems can be due to oversized holes in connections, to the slack (or catenary action) of members in the stabilizing system, and to the initial crookedness of various members of the bracing system (e.g. purlins).

- A survey of experienced structural engineers concerning matters of slender roof structures was conducted in order to gain an understanding of their approach to the designing of slender structures. The engineers who participated in the survey represented 6 different nations and had an average working time experience of 20 years. Altogether, 17 engineers participated in the survey.

  According to results of the survey, many structural engineers believe that inadequate design is largely due to erroneous calculations being made. Also, they believe that a greater amount of review and improved communication between different partners in a building project would be useful for improving the safety of slender structures.

- The survey of the experienced structural engineers also included a part that was aimed at checking a participant's individual approach to different concrete problems of stability design, so as to investigate whether there were any fluctuations that were evident in their design assumptions.

  The survey revealed that some of the engineers, despite their relatively long experience, made non-conservative assumptions regarding the buckling length of a beam. Also, some of the participants proposed theoretically unstable structural solutions to roof structures. These discrepancies all have a strong potential for differing degree of structural safety.

- A non-destructive testing method for determining the bracing stiffness of slender roof structures was presented. In practice, the method can be useful for cases in which there are many uncertainties involved in the designing of the bracing system, for example. Engineers can compare the stiffness of the actual bracing system with their design assumptions or can update their structural models through use of the possibly more accurate stiffness values obtained by means of the testing.

  The method in question can be used to determine the point-wise bracing stiffness of roof structures, through the application of a horizontal load at a certain point in the structure; both the load and the displacement being monitored at this point. In order to apply this load, a special test rig was developed and was built.

The method has not been tried out in field (due to unclear insurance terms and to difficulties in finding suitable objects), but was successfully used in the laboratory testing of a slender roof structure.

- Laboratory tests of common bracing systems used for wooden roof structures were conducted. Two different bracing methods for roofs were considered here, namely those of wind trusses in the plane of the roof and of a steel decking being placed on top of the purlins (diaphragm action). The testing focused on the point-wise axial bracing stiffness obtained at the tip of the purlins of the structure. The finite element model updating of a corresponding analytical model of the structure was successfully used to determine the stiffness contributions of different parts of the structure. For example, the effects of the number of fasteners used in connections, both between the corrugated steel sheeting and the timber structure and in the shearing connections between purlins and primary beams, were assessed successfully on the basis of this testing.

  Among other things, the results of the study indicate that the stiffness of a roof structure is strongly affected by its connections, especially connections that transfer loading pressure through pressure being directed against the timber. In fact, the stiffness of a structure, especially a timber structure, can be markedly overestimated in numerical simulations if its connections are not adequately accounted for.

7 Further research

Various suggestions for further research can be provided.

One set of suggestions concerns the conducting of further surveys of the approaches used by practising engineers of long experience:

- A survey of structural engineers of the sort that was carried out can be expanded both to include more participants and to represent a wider variety of nations.

  Including more participants would be useful in a statistical sense through its providing the basis for a more confident and precise statistical evaluation of matters of interest.

  It would also be interesting to investigate possible differences between participants of different countries. This require several participants from each country that is represented in the survey.

- The effectiveness of external or independent review of designs in order to detect errors could be investigated further in the following ways for example:

  A number of structural engineers could be provided with a set of drawings, some of which are adequate and some of which are inadequate in terms of the design involved. One should measure how many errors are detected under the conditions involved.

Another set of suggestions for further research concerns further full-scale tests of different buildings using the testing methods that were developed here, taking account of the following:

- The method should be used in the field testing of both steel and timber roof structures. It is important to learn more regarding the characteristics of the structures in question, bearing in mind too the fact that all the effects of interest can probably not be captured or captured adequately on the basis of laboratory tests alone.

- It would be preferable for both the ”naked” version of a structure and the finalized version of it to be tested, so that the potential stiffness contributions from secondary structural detailing (something which is normally not considered in design) can be readily isolated. It would thus be important that the test objects be identified prior to their erection.

8 Summary of appended papers

Paper I - Slender steel columns: how they are affected by imperfections and bracing stiffness

Finite-element-based programs can be used to design both columns and their bracing systems. As is well known, however, the accuracy of the output obtained is highly dependent upon the adequateness of the input. In the present study, the effects of imperfections on the predicted strength of steel columns and on the stiffness requirements of their bracing systems were investigated. Two different systems were analysed: 1) a braced non-sway column, and 2) a braced sway column. It was found that a poor choice of the shape of these imperfectly formed columns can provide unrealistic results in terms of both the effects of the buckling load on the columns and the predicted reactions of the bracings, and that superimposing different imperfectly formed shapes can contribute to obtaining realistic and trustworthy results. It was also found that the shapes of the initial imperfections that lead to the lowest buckling load and those that result in the strongest forces that are directed at the bracings are generally not the same. Thus, at least two different imperfect shapes need to be considered in the designing of braced slender columns.

Paper II - Design for lateral stability of slender timber beams considering slip in the lateral bracing system

In this study the significance of potential slip in the bracings of simply supported slender timber members was investigated. Three bracing configurations were considered. The first case was that of a timber beam braced at one point at mid-span, the second one was that of a timber beam braced at two points and the third that of one braced at three points. Possible slip in the bracing members can be due to, for example, joint deformation, initial crookedness of purlins and slack (or relaxation) of cables in the stabilizing bay (catenary action). In the study, it was shown that slip in the bracing system can result in a reduced load-bearing capacity (due to greater beam stresses) of the beams. Also, the greater the slip, the greater the lateral deflections and the consequent bracing forces of the braced member are. A simplified approach, using a larger initial geometrical imperfection to account for potential slip in bracings was also evaluated. This approach was found to work reasonably well in terms of stresses and bracing forces, but at the same time to underestimate the lateral displacements involved.

Paper III - Slender roof structures: Failure reviews and a qualitative survey of experienced structural engineers

Many slender roof structures have collapsed due to snow loading and to instability. Although accurate stability calculations can be performed using theoretical models, these calculations may not always reflect the behaviour of real structures, as a result of uncertainties regarding such matters as loading, material behaviour, geometry, initial imperfections and boundary conditions of the structure. Accordingly, the approach to stability design that is adopted requires subjective decisions on the part of the structural engineer concerning the loading and the modelling assumptions involved. In this paper the significance of decisions of these types made by structural engineers in designing slender roof structures was investigated. The study involves a review of previous studies of failures together with a survey of the views of 17 experienced structural engineers. The results obtained indicate most structural failures to be the result of human errors, a suitable strategy for avoiding errors thus being one based on quality control and design checking. In addition, significant discrepancies were observed regarding the design assumptions made by the engineers in the study. Some of these assumptions, such as those connected with a non-conservative choice of the buckling length of a beam, have a significant negative impact on structural safety. It is thus recommended that the structural engineers involved in the design of a structure have adequate experience and a holistic mindset. Another recommendation is that both drawings and design calculations be thoroughly reviewed prior to construction. Also, any temporary bracing to be used during construction should be included in the design. Finally, it is important that the communication between the different parties involved in the process of designing a structure be adequate and satisfying.

Paper IV - The effects on the bracing stiffness of timber structures of the stiffness of its members

The design of slender structures is often associated with a number of assumptions made by the engineer or engineers involved in order to check the stability of the structure. Not seldom, these assumptions are rather uncertain. In this paper the effects of the stiffness of different members on the bracing stiffness of timber structures produced is studied. Both full-scale laboratory testing and FE-modelling are employed in the study. In particular, two different bracing approaches are analysed, namely 1) cross bracing and (2) diaphragm action (use of a steel sheeting) in the plane of the roof. In addition, the effects of the connections and of the number of fasteners used in the structure are evaluated. The stiffness of the connections is obtained through use of an FE-updating approach, that of the relevant parts in the FE-model being calibrated so that the FE-results match the laboratory results. The findings obtained indicate 1) that connections can have a significant effect on the stiffness of bracing systems, 2) that cross-bracings close to the mid-span of roof structures are less effective than bracings close to supports, 3) that the lateral stiffness obtained using a diaphragm approach is strongly related to the number of fasteners used between the steel sheet and the timber parts in the roof, and 4) that the two different bracing approaches involved result in about the same lateral stiffness of the roof. Finally, it is emphasized that FE-models may markedly overestimate the stiffness of timber structures if connections are not modelled accurately; the engineer being advised to seek assumptions that are safe.

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Scientific publications

Contributions of the authors

The authors' names are abbreviated by use of their initials. The first author was the corresponding author for each of the published papers.

Paper I: Slender steel columns: how they are affected by imperfections and bracing stiffness

AK developed the idea, carried out the modelling in question, and was main responsible for the writing of the paper. RC and EFH contributed to the paper by both checking the results presented in it and by reading the manuscript carefully.

Paper II: Design for lateral stability of slender timber beams considering slip in the lateral bracing system

AK developed the idea, carried out the modelling in question, and was main responsible for the writing of the paper. RC, IB and EFH contributed to the paper by reading the manuscript carefully and providing valuable feedback.

Paper III: Slender roof structures: Failure reviews and a qualitative survey of experienced structural engineers

AK developed the original idea of the paper. AK, RC, IB and EFH contributed in designing the questionnaire used in the study. AK was main responsible for evaluating the results of the study and writing the paper.

Paper IV: The effects on the bracing stiffness of timber structures of the stiffness of its members

AK and RC developed the original idea of the paper. AK was main responsible for executing the testing in question and writing the paper.