The effect of uncertainties in the design of complex structural systems by using FEM

Noor Alsuhairi and Martin Hedström

Division of Structural Engineering
Faculty of Engineering, LTH
Lund University 2013

Report TVBK - 5222

Avdelningen för Konstruktionsteknik
Lunds Tekniska Högskola
Box 118
221 00 LUND

Division of Structural Engineering
Faculty of Engineering, LTH
P.O. Box 118
S-221 00 LUND
Sweden

The effect of uncertainties in the design of complex structural systems by using FEM

Noor Alsuhairi
Martin Hedström
2013

Report TVBK-5222
ISSN 0349-4969
ISRN: LUTVDG/TVBK-13/5222(116)
Master Thesis
Mentor: Martin Fröderberg, Division of Structural Engineering, LTH, Kent Persson, Division of Structural Mechanics, LTH and Sven Thelandersson, Division of Structural Engineering, LTH
April 2013

Acknowledgement

The work presented in this master thesis was carried out for the division of Structural Engineering at Lunds institute of technology in collaboration with the urban planning consultancy company Tyréns. The person that has made this thesis possible and suggested the subject is Msc.Martin Fröderberg and it origanates from his ongoing Ph.D research.

We would like to thank Martin Fröderberg for giving us the opportunity to do this exciting project. We would also like to thank Tyréns for giving us the chance to work at their office in Lund. Furthermore we would especially like to thank Professor Sven Thelandersson for the excellent guidance along the way. We would also like to to thank lecturer Kent Persson and Dr Oskar Larsson for contributing with the guidance of BrigadePlus. Additionally we would like to thank Mikael van de Leur and Fredrik Lagerström at Strusoft for giving us the opportunity to get educated within FEM-design and for the loan of license of FEM-design and Johan Kölfors at Scanscot for providing all the help and material for BridgadePlus.

Moreover we would like to show our gratitude to Professor Roberto Crocetti, Structural Engineering in bridges Thomas Kamrad, Structural engineer Martin Larsson, Regional Director Jan Nord, Structural engineer Mikael Rosengren, Structural engineer David Persson, Business Director Mats Persson, Head of department Mats Skällenäs and Tekn Dr at the construction management department and President at Swedish Council Quality Kristian Widén for participating in the interviews that have been carried out through the thesis and being a part of this process.

Last but not least we would like to thank Dr Peter Lundqvist and fellow students and friends for the support we have had during the period spent writning the thesis at Lunds institute of technology.

Lund Mars 2013

Noor Alsuhairi and Martin Hedström

Abstract

In today's society increasingly complex structures are designed whilst at the same time the design of a single structure ought to occur in a shorter period of time. This fact has undoubtedly contributed to that computational strength calculation that often is based on the finite element method instead of the classical hand calculation methods, is applied more frequently. Therefore an important aspect to consider when analyzing the results is whether the end results vary between the different calculation methods or not and if the outcome differ between diverse FE-programs. The quality control procedure of the calculation process is also of interest to be analyzed in order to study how companies in the industry ensure themselves that the appearance of uncertainties is minimized.

The aim of this master's thesis is to analyze the reaction forces that will appear in the columns and the shear walls in a building. To achieve this result the master thesis has been carried out both through static hand calculations and by finite element analyses. Interviews with ten influential people within the construction industry has also been carried out in order to obtain how companies in the business deal with the issue of quality control and how the execution is carried out in the field. Nowadays, there is a variety of software for the analysis and design of buildings supported by the finite element method. In this thesis, three computer programs have been examined and compared with both 2D and 3D simulated models. Sensitivity analyzes and parametric studies have been performed with all three programs where the results have been compared against each other and against the static hand calculation. The building was analyzed with the program Ramanalys, FEM-Design where the 3D structure was applied, and with Brigade / Plus. The results indicate on a number of similarities in the reaction forces obtained from the different software's when the 2D-model was analyzed while significant differences in 3D-model have been obtained. The explanation to why a variety of the results were obtained in the 3D-model can be found when taken the non-rigid motion of ground into account. By analyzing the interviews, conclusions can be drawn that all the participants agreed that society today have a problem with quality assurance and uncertainties in the process. However, the participants had different opinions in how the issue could be solved. Since the results received from the various models indicate a range of differences, conclusions can be drawn that it is extremely important to implement sensitivity analysis and parametric studies when needed. Furthermore an accurate quality control and documentation of the calculated steps ought to be carried out.

Sammanfattning

I dagens samhälle projekteras alltmer komplicerade konstruktioner, samtidigt som projekteringen av enskilda konstruktoner skall ske i en allt snabbare takt. Detta har utan tvivel bidragit till att datorbaserade hållfasthetsberäkningar som ofta baseras på finita elementmetoden istället för de klassiska handberäkningmetoderna tillämpats allt mer frekvent. En viktig aspekt att beakta vid analys av resultaten är därför huruvida resultaten skiljer sig åt mellan de olika beräkningsmetoderna samt mellan olika FE-program. Kvalitetssäkring av processen är även av intresse att analysera för att undersöka hur företag inom branschen går tillväga för att försäkra sig om att uppkosten minimeras.

Syftet med examensarbetet var att analysera de reaktionskrafter som uppstår i pelarna och stabiliserande väggarna i en byggnad. Intervjuer med tio inflytesrika personer inom konstruktionsyrket har även genomförts för att ta del av hur företag inom branschen ser på frågan om kvalitetssäkring samt hur utförandet av denna utförs ute i verksamheten.

Nuförtiden finns det en mängd olika mjukvaror för analys och dimensionering av byggnader med stöd av finita elementmetoden. I detta arbete har tre datorprogram granskats och jämförts där både 2D- och 3D- modeller simulerats. Både känslighetsanalyser samt parameterstudier har utförts med alla tre programmen där resultatet jämförts mot varandra samt mot den statiska handberäkningen. Byggnaden har analyserats både med programmet ramanalys, FEM-Design, där 3D structure har tillämpats, samt med Brigade/Plus.

Resultaten indikerar på en rad likheter i reaktionskrafterna som erhölls från de olika programmen när 2D modellen analyserades samtidigt som påtagliga skillnader erhålls då 3D modellen studeras. Förklaringen till skillnaden i 3D modellerna är främst hur eftergivlighet i marken modelleras.

Från intervjuerna kan slutsatser dras att alla medverkande anser att samhället idag har problem med kvalitetssäkring och osäkerheter i processen. Däremot skiljde sig de medverkarndes syn på hur lösningen till problemen skulle kunna vara. Då resultaten som erhållits från de olika modellerna tyder på en rad skillnader kan slutsatser dras att det är ytterst viktigt att genomföra känslighetsanalyser och parameterstudier. Dessutom bör en ordentlig kvalitetsäkring samt dokumetation av beräkningsgången implementeras.

List of Abbreviations

Latin upper case letters

A	Cross sectional area
A_{column}	Cross sectional area of column
A_s	Cross sectional area of reinforcement
A'_s	Cross sectional area of reinforcement, upper
E	Young's modulus of elasticity
E_{cd}	Design value of modulus of elasticity of concrete
E_{cm}	Secant modulus of elasticity of concrete
EI	Bending stiffness
F	Action
$G_{kj,sup}$	Upper/lower characteristic value of permanent action j
G_f	Fracture energy
G_{f0}	Base value of fracture energy which depends on maximum aggregate size
I_c	Second moment of area of concrete section
I_s	Second moment of area of reinforcement section
K	Stiffness matrix
K_C	factor regarding cracking, creep etc
K_S	factor regarding contribution of reinforcement
L_{cr}	Euler buckling
M	Bending moment
M_{0d}	First order bending moment according to ultimate state load
M_{0Eqp}	First order bending moment according to quasi-permanent serviceability state
M_{0Ed}	Design value of the applied internal bending moment
M_{sd}	Design moment
N	Axial force
N_b	Euler's buckling load
N_{inst}	Installation load, included in live load
R	Reaction force
T	Tensile force
$Q_{k,i}$	Characteristic variable action

Latin lower case letters

a	Displacement vector
a_d	Design of geometrical data
b_i	Overall width of snow area
c_{norm}	concrete cover
d	Effective depth of a cross-section
e	Eccentricity
f	Force vector
f_{cd}	Design value of concrete compressive strength
f_{ck}	Characteristic compressive cylinder strength of concrete at 28 days
f_{cm}	Mean value of concrete cylinder compressive strength
f_{cm0}	10 MPa
f_{ctm}	Mean value of axial tensile strength of concrete
f_{yd}	Design yield strength of reinforcement
f_{yk}	Characteristic yield strength of reinforcement
h_i	Height building part i
h_0	Fictitious height
k	Spring stiffness
k_i	National parameter
k_{ground}	Spring stiffness ground
k_{column}	Spring stiffness column
k_{system}	Spring stiffness system
n	Relative axial force
n	Quantity
r_m	Torque ratio
q_p	Distributed load
s_k	Characteristic value of snow on the ground at the relevant site
t_i	Thickness of building part i
t_0	The age of concrete at the time of loading
u	Displacement
v_b	Wind velocity
y	Coordinates
y	Displacement

Greek upper case letters

Δk	Difference in spring stiffness

Greek lower case letters

α_h	Reduction factor for the length or height
α_m	Reduction factor for the quantity of construction parts
β	Moment assuming a sinus shaped distribution
γ_c	Partial factor for concrete
γ_d	Partial factor
γ_i	Weight density for part i
γ_S	Partial factor for reinforcing or pre-stressing steel
ε_c	Compressive strain in the concrete, inelastic
ε_{cu}	Ultimate compressive strain in the concrete
ε_{0c}	Compressive strain in the concrete, elastic
θ_i	Angle of imperfection
θ_0	0.005
ι	Radius of gyration
λ_{lim}	Slenderness ratio
μ_i	Snow load shape coefficient i
ν	Partial factor
σ_c	Compressive stress in the concrete
σ_{c0}	Yeild stress
$\sigma_{Rd,max}$	Maximum applicable stress
$\varphi(\infty, t_0)$	Creep coefficient, defining creep between times t and t_0 , related to elastic deformation at 28 days
ϕ_{eff}	Effective creep
ψ_i	Deformation at 28 days

Contents

Chapter 1

Introduction

In the introduction the reader will be presented to the master thesis background, aim and scope. The thesis is divided into two parts, analysis of the building and quality control

1.1 Background

Today buildings become more and more complex and computer based tools give the possibility to increase the effectiveness of the planning process. Advanced and complex structures are simulated, calculated and designed much faster and much more effective but at the same time the time given to perform an examination and too reflect is decreasing. This requires large knowledge and experience of a engineer.

Martin Fröderberg, Industrial Ph.D student at the department of Structural Engineering at The faculty of engineering, Lund University, created a test case based on earlier building projects.A number of structural engineers was asked to solve several tasks in the test case. The task was to check the dimensions of the concrete columns, calculate the force transferred to the ground in a five story building. The result concerning forces transferred to the ground differs up to 300 % between the participants. It is of interest to investigate the issue further more due to the apparent difference. The difference in result may occur due to uncertainties and lack of knowledge in the design process.

Several collapses has occurred in Sweden over the past years and many of them would be possible to prevent with a more effective quality assurance system. Quality control is an essential part in the planning process and an important step to ensure that the calculations and assumptions are reasonable.

1.2 Aim and scope

The master thesis aims are summarized in the list below:

- How do uncertainties affect the result of advanced calculations done with FEM?
  - Choice of boundary conditions
  - Choice of material model

- Which factors can affect a difference in result?

- How to improve quality assurance?

    - How should the calculations and result be documented?

    - How should a third person check that the results of the calculation are reasonable?

1.3 Limitations

Is necessary to set some limits so that the master thesis does not become too extensive. The master thesis should be seen as a start and a feasibility study for larger a project.

- Only three FE-programs were used for analyzing and compared with hand calculations.
- The ground is modeled with springs instead of a solid mass.
- The quality assurance part of the thesis does not cover the whole process from planning to production. Only the planning part with focus on calculations and simulations/-modeling is considered.

Part I

Analysis of the building

Chapter 2

Background and basic concepts

In this chapter the reader is introduced to the case study. Background and basic concepts are described.

2.1 Assumptions

2.1.1 Geometry

Figure 2.1 and appendix 10.1 illustrate the overview of the building that is analyzed. The first floor of the building is used for restaurant, kitchen etc. and carries the other floors by columns and two small walls on each side of the building and one small wall in the middle. The remaining floors consist of apartments. The facade walls on the long sides consist of several layers of material such as a brick layer, plaster, isolation etc. The facade walls have, in theory, no stabilizing function and will only be simulated as a line load on each floor, see appendix 10.2 and load $G_{outerwallperfloor}$. The geometry of columns, slabs and walls are illustrated in table 2.1. In the simulation windows and doors are neglected and will add more loads on the columns as a result.

Table 2.1: Thickness of structural elements

Part	thickness (m)	height (m)
Shell wall	0.25	2.525
Slab	0.225	-
Outer wall	0.25	-
column	0.3	3.68

2.1.2 Concrete

The structure is made of reinforced concrete. The strength of the concrete is assumed to be C45 during the entire calculation process and the design strength is calculated with Eurocode. This procedure is the same for the reinforcing bar B500BT. According to [30] the design strengths of concrete in compression and yield stress of reinforcement steel are:

$$f_{cd} = \frac{f_{ck}}{\gamma_C} \tag{2.1}$$

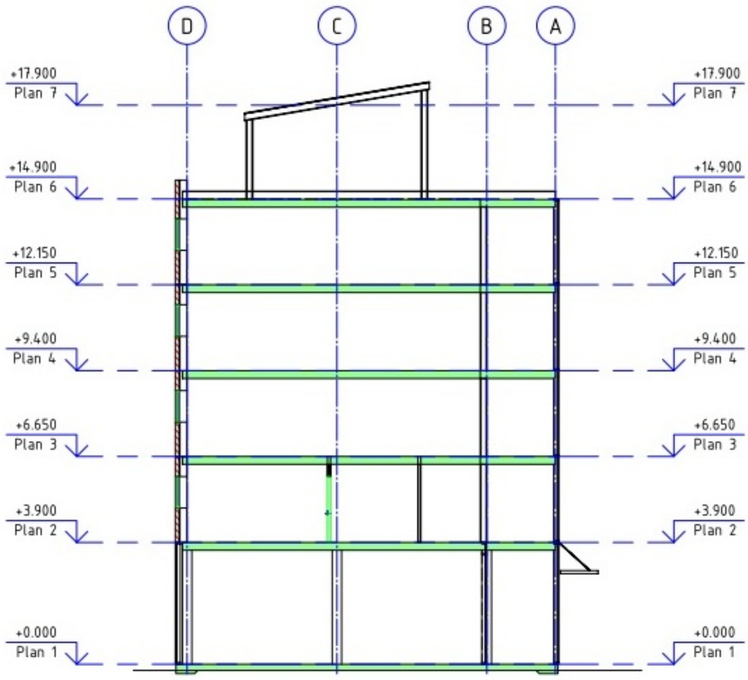


Figure 2.1: Section of building

$$f_{yd} = \frac{f_{yk}}{\gamma_S} \tag{2.2}$$

where $\gamma_C = 1.5$ and $\gamma_S = 1.15$

$$f_{cd} = \frac{45MPa}{1.5} = 30\,MPa$$
$$f_{yd} = \frac{500MPa}{1.15} = 435\,MPa$$

According to [30] the ultimate compressive strain in the concrete is $\epsilon_{cu} = 3.5\,‰$ and the secant modulus of elasticity of concrete $E_{cm} = 36\,MPa$ according to table 3.1 [30]. The density of reinforced concrete is 2500 kg/m^3 and Poissons's ratio is 0.2 for uncracked concrete according to [30].

2.1.3 Reinforcement

The reinforcement has a Young's modulus of 200 GPa and a Poissons ratio of 0.3 [30]. The density of the reinforcement is 7850 kg/m^3 according to [30].

2.1.4 Ground

The ground consist mainly of filling material and the structure will be supported by concrete piles, see section 3.3.2 how this is taken into account.

2.1.5 Orientation of direction

The concept 'reaction force' refers to reaction force in the z-direction.

2.1.6 Stabilization

The stabilization of the structure will not be included in the analyses.

2.2 Plastic analysis

2.2.1 General

Plastic analysis should only be used for the calculation of the ultimate limit state. If the design is based on a force distribution determined by plastic analysis, the force distribution in the serviceability limit state is calculated by another method, usually linear elastic analysis [2].

In a statically indeterminate structure yielding occurs before the ultimate load is reached. With continued loading, yielding will spread within a limited area adjacent to the zone where the yielding is started so that a plastic hinge is formed, see figure 2.2 [2].

The plastic hinge must then be able to rotate plastically until a the failure load occurs, i.e. the plastic hinge must have sufficient plastic rotation capacity. This means that the critical sections must have sufficient ductility [2].

Plastic analysis is based on either lower limit (static method) or the method of upper limit (kinematic method). For high beams and discontinuity zones the strut and tie, which is based on the lower limit method, is applied [2].

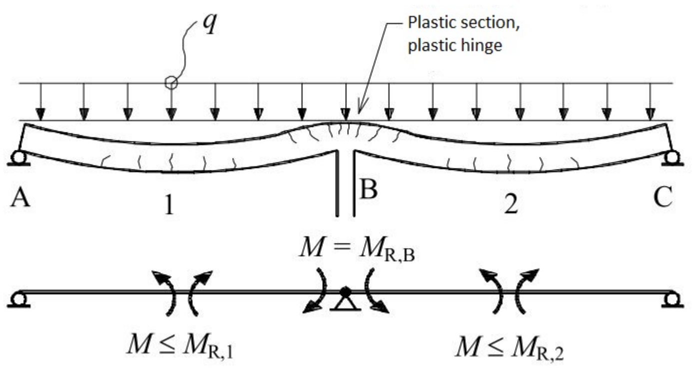


Figure 2.2: Continuous beam over three supports with a plastic hinge [2]

2.2.2 Upper and Lower Bound Solution

Sections 2.2.2 - 2.2.9 are based on theory from the literature ”Design and analysis of deep beams, plates and other discontinuity regions ” by Björn Engström, if no other source is given.

The upper-bound theorem is based on solutions that are on the unsafe side with respect to the theoretical carrying capacity of plasticity [2]. It is not possible for the real plastic capacity to be greater than the calculated, which means that the method is on the unsafe side. It is possible for the structure to find a more effective way to fail with the assumed capacities.

The lower-bound theorem states that solutions based on assumed equilibrium stress distributions are on the safe side with respect to the theoretical carrying capacity of plasticity [2]. It is not possible for the real plastic capacity to be less than the calculated, which means that the method is on the safe side.

Numerous different stress fields or failure mechanisms can be assumed and it is possible to calculate the failure load for each stress field or failure mechanism. For the lower bound approach, the most “true” plastic solution is the one with the maximum resistance. For the upper bound the result is the opposite. The ‘true’ plastic solution is found when, if possible, the upper and lower bound solutions coincide. Due to diverse stiffness distribution and different obtained equilibrium the numerous reinforcement arrangements will give different stress fields. The final stress field is either similar or equal to the designer’s choice will progress successively due to plastic redistribution until the capacities provided are fully used.

The strut and tie method is based on idealizations in theory of plasticity where the real materials, especially concrete, are assumed to be ideally plastic and have limited plastic deformation capacity. Moreover the structure ought to fulfill requirements for both the ultimate and the serviceability limit state. However when using the strut and tie model one should keep in mind that this method only concerns the resistance in the ultimate state. Just because a design fulfills sufficient load-carrying capacity does not automatically mean it will fulfill needs in the service state.

2.2.3 Design on the basis of plastic analysis

There are numerous methods available for plastic analysis. “Plastic hinge method” for continuous beams and frames, the “strip method” for slabs and flat slabs and the “strut and tie” method for discontinuity and continuity regions of various structural elements are three used methods for plastic analysis. When analyzing reinforced concrete structures all three methods simulate the stress field in cracked reinforced concrete in the ultimate limit state after plastic redistribution.

2.2.4 Uncertainties

“With regard to the limited plastic deformation capacity of the real materials, the stress field in the ultimate limit state should be chosen such that it is similar to the one found by linear analysis” ([7], p 36).

2.2.5 Deformation compatibility

Deformations are not treated in theory of plasticity.

2.2.6 Design with regard to the serviceability limit state

As previously mentioned, the strut and tie method is based on theory of plasticity and evolves the resistance of the structural element in the ultimate limit state. However, in the design of structural members a suitable structural performance in the service state must also be considered. Therefore, the stiffness of reinforced concrete members in the service state ought to be sufficient to avoid excessive deformations and wide and deep cracks. The crack widths should be limited to acceptable standards with respect to the risk of corrosion and aesthetics.

2.2.7 Statically determinate and statically indeterminate problems

In a statically determined structure the sectional forces can be determined by means of equilibrium conditions only. A simply supported beam with distributed load is in general characterized as a statically determinate problem. Therefore the sectional forces can be determined through equilibrium conditions only. If the beam is placed on three or more supports the situation will be different. To be able to solve the sectional forces in such cases the equilibrium conditions must be combined with compatibility conditions and constitutive relations. Constitutive relations are often depending on molds. An example of this is that for a structure there is a linear relation between moment and curvature in a section. In deep beams and other discontinuity areas the stress field is statically indeterminate which means that it is independent of if the structural member on the global level is statically determinate or not. For discontinuity regions a simple compatibility condition does not exist. The solution of the stress field will therefore become very difficult.

2.2.8 Typical behaviour of discontinuity regions and modeling

Uncracked state

When the concrete is solid the influence of the reinforcement is limited and the behavior under load is nearly linear. In this state it is suitable to study the structural member by linear analysis assuming a homogenous material. Results obtained from a uniform analysis are unique stress field configurations independent of load intensity. The stress field can later be used to find regions with large tensile stress that are prone to cracking and where reinforcement might be necessary. Solutions for statically indeterminate problems are obtained by combining equilibrium, compatibility and constitutive relations where the constitutive relation is a linear stress-strain relationship for the assumed homogenous material.

Linear analysis will result in one unique solution. Therefor if the stress field is determined for one value of the load, the configuration of the stress field will remain when the load increases. The only thing that will increase is the magnitude of the stress. For the same load case the stresses and deformations will increase linearly with the load. Since the stiffness of the structure is determined by the geometry and the elasticity of the material it will result in this linear behaviour.

In a linear analysis little information about the structural element is required. It is often enough to only analyze a gross geometry and the load that affects it. For that reason linear analysis can be carried out early in the design process.

Cracked state

A drastic change of the stiffness conditions in the structural member will be a result when cracking occurs, Figure 2.3. The stiffness will differ among varying regions depending on if they are cracked or not. Besides, the stiffness of the cracked regions is basically influenced by the quantity and arrangement of the reinforcing steel in that region. The stress distribution will determine how the actual stress field will appear. Stiffer regions will attract forces from softer regions in each load step. Therefor a continuous change of the stiffness distribution will be a result of load increasing and crack developing, this behaviour is known as 'stress redistribution due to cracking'. In each load step stresses reallocate since forces are controlled by the stiffness distribution that changes constantly.

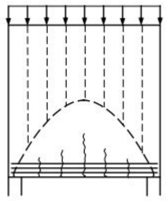


Figure 2.3: Deep beam of reinforced concrete in the cracked state [7]

Once the configuration of the stress field changes under increasing load, the increase of the stresses and deformations will not be in proportion to the load. A nonlinear behavior will instead be obtained, regardless of the fact that the materials that are concrete and steel still have linear elastic material responses..

As it is not possible to predict the stress field or the behavior in the cracked state by linear analysis, non-linear analysis is required. This can be done as a non-linear FE analysis. More advanced analysis are made when studying reinforced concrete member, a non-linear analysis considering cracking of concrete by fracture mechanics, reinforcing steel and the interaction between steel and concrete. An analysis of that kind requires complete information about the structural member concerning material properties and reinforcement arrangement and cannot be carried out as confirmation before the end of the design process when the data is available. Additionally one should keep in mind that a reinforced concrete element will behave non-linearly even if it is designed based on linear analysis.

Plastic redistribution

Development of plastic deformations before the ultimate state is reached in an area is equal with a radical decrease of, or even loss of, stiffness. Plastic behavior in an extremely stressed region results in stress redistributed in the structural member and a change of the stress field configuration. As a consequence of that, each load step in stiffer regions attracts forces from softer regions. Stress redistribution that is caused by plastic deformations is known as 'plastic redistribution'. Figure 2.4 illustrates how the main tie in the deep beam begins to yield. The tie stiffness is reduced and the deflection starts to increase significantly.

As a result, the cracks spread upwards and the crack sizes increase. Consequently the compressive arch will be forced upwards. Despite the fact that the tensile force in the main tie is constant after yielding, the load on the deep beam can increase since the internal lever arm increases. The configuration of the stress field will change because of plastic redistribution in the ultimate state. The process can continue as long as there is sufficient room for the compressive arch to rise and there is no critical parts that limit the load-bearing capacity. Critical subjects in the design are the anchorage of the main tie and the resistance of the highly compressed areas at the supports [7].

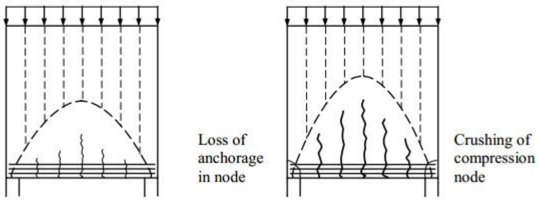


Figure 2.4: Plastic redistribution of stress field [7]

Ultimate limit state

The ultimate limit state is the point where the structure is about to collapse. Due to no remaining capacity for plastic redistribution the structure develops a collapse mechanism. In the ultimate limit state some of the critical regions of the structural member have reached its plastic resistance. This plastic resistance determines the resistance of the whole structure.

The theory of plasticity assumes ideally plastic behavior of materials and is applied when studying the final equilibrium condition. This type of analysis is based on 'plastic analysis'.

Extension of discontinuity regions

The definition of discontinuity regions can be based on Saint-Vénant's principle. Figure 2.5 illustrates a body that is subjected to a system of forces in self equilibrium.

Pursuant to Saint-Vénant's principle stresses will emerge in the body as a local consequence of the applied forces. As illustrated in the figure 2.5 the length d of the stressed region is equal to the maximum distance h between the forces in the applied system of forces.

Figure 2.6 illustrates how the size of the discontinuity region can be approximated. The figure shows a structural part with a concentrated compressive force F in the center of the end section. A bit further down in the part the strain distribution is uniform, which results in a uniform stress, Figure 2.6 also shows the load case divided in two parts. When superimposing the parts the outcome will be the original load case [26].

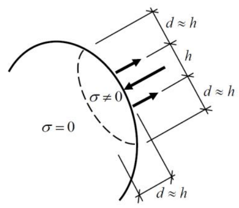


Figure 2.5: St. Venants Principle [26]

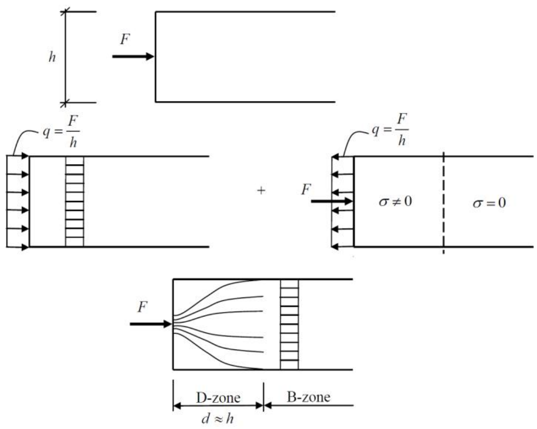


Figure 2.6: Extension of the discontinuity regions [26]

In the first load case the part is subjected to an evenly distributed compressive stress $\sigma = F/A$. In this load case the entire part will be a continuity area with the same stress σ in all units. In the second load case a force system with one centric concentrated force acting to the right and a uniformly distributed stress to the left. In the illustrated case according to Saint-Vénant's principle the height h is where stresses will appear and outside of this region no stresses will appear. The stress field under the concentrated force F is found when the first and second load cases are superimposed. Uniformly distributed stresses will emerge after the end zone with length h. The region deep inside the part is called the "continuity zone". The uniform stress field is disturbed by local effects in the region with length h close to the concentrated force. The end region is called the "discontinuity region" [26].

2.2.9 The Strut and tie method

The Strut and Tie method is an excellent method for analyzing the structure in the early design process, for example arranging reinforcement steel.

Procedure

The Strut and tie method is based on the theory of plasticity and the purpose of the method is, in the ultimate limit state, to simulate the stress field in cracked reinforced concrete. This is done with struts, ties and nodes. Although some critical regions have reached plastic behavior it is recommended to apply a strut and tie model that is close to the linear elastic stress field. A linear elastic stress field is used because of the limited ability of plastic redistribution in reinforced concrete.

Application

The strut and tie model can be established on the basis of the load path method or stress trajectories or principal stresses from a linear FE-analysis. The load path method divides the load where the shear force is zero and carries the load down to the nearest support, Figure 2.7. Due to equilibrium condition a load path cannot change its direction without a transverse force. Where a load path is "bent", a transverse force must be introduced when modeling the truss. For example this transverse force could be a tensile force near the edge at the bottom in a deep beam.

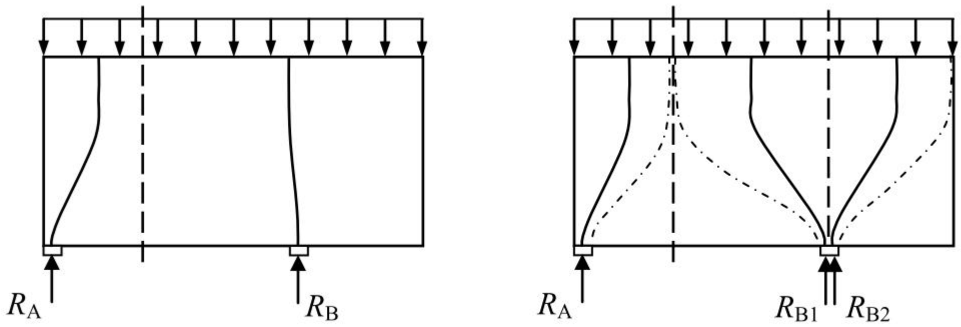


Figure 2.7: Stress field and load paths in a deep beam with cantilevering end [7]

Struts, ties and nodes can be classified as "concentrated" or "distributed". Concentrated nodes is located at the boundaries of the D-region whereas distributed nodes are located where distributed stress fields meet.

Application recommendations

When a strut and tie model is developed, one or more angles have to be chosen. Angles that are dependent on the chosen angle can be solved with geometrical conditions. Due to service state and the need for ductility it is recommended to follow some application recommendation. Figure 2.8 and 2.9 below illustrates some recommended angles according to.

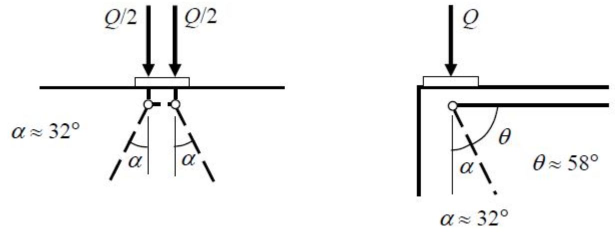


Figure 2.8: Deviation of concentrated forces [7]

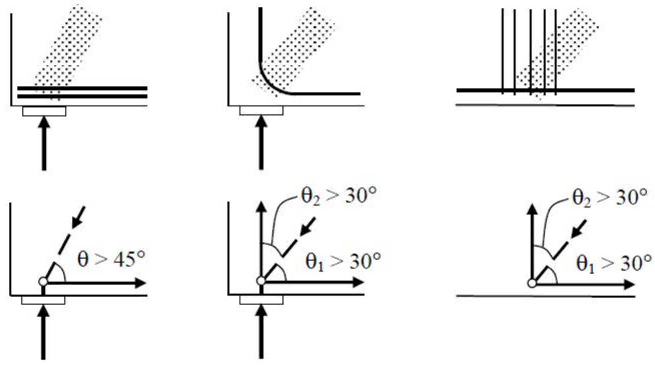


Figure 2.9: Recommended minimum angles between struts and ties [7]

Design of compression-tensile nodes

The stresses in nodes are influenced by bearing, anchor plates, loading plates etc. In the structure a node needs to fulfil several stress criteria based on the concrete compression strength. These criteria are stated as, according to:

- Compression nodes, no ties anchored in node

$$\sigma_{Rd,max} = k_1 \, v \, f_{cd} \tag{2.3}$$

$$v = 1 - \frac{f_{ck}}{250} \tag{2.4}$$

Where $k_1 = 1.0$ is a national parameter and f_{cd} is in MPa.

- Compression nodes, anchored ties in one direction

$$\sigma_{Rd,max} = k_2 \, v \, f_{cd} \tag{2.5}$$

Where $k_2 = 0.85$ is a national parameter. $\sigma_{Rd,max}$ is the maximum compressive stress that can be applied at the edges of nodes.

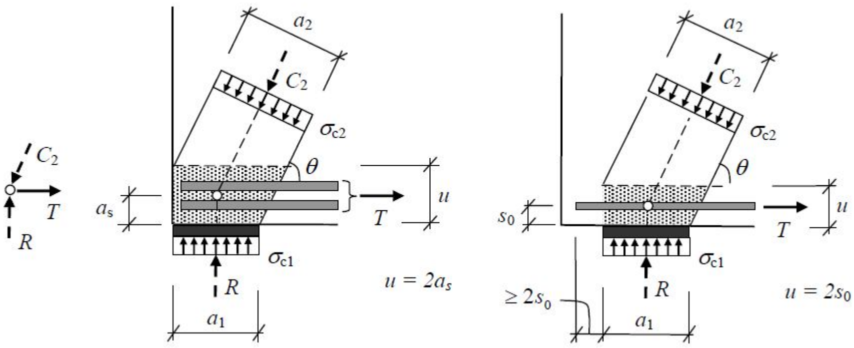


Figure 2.10: Compression – tension node with anchored reinforcement [7]

A node with one tie anchored is shown in figure 2.10. The stress σ_{c1} acting on the support must be less than $\sigma_{Rd,max}$. C2 is the compression strut which is acting on the node and should also be checked with regard to the stress limit. The stress limitation for the strut is described in chapter 2.2.9 below.

- Compression nodes, anchored ties in more than one direction

$$\sigma_{Rd,max} = k_3 \, v \, f_{cd} \tag{2.6}$$

Where $k_3 = 0.75$ is a national parameter.

Ties

The tensile force, calculated for the ties, should represent the reinforcement needed to maintain the equilibrium in the system. The reinforcement should be located in the same direction as the tie. If the reinforcement is in two or more layers,centroid of the reinforcement should be located at the position of the tie. The amount of reinforcement needed is calculated according to as:

$$A_s \geqslant \frac{T}{f_{yd}} \tag{2.7}$$

Where T is tensile force in tie and f_{yd} is design tensile strength in reinforcement.

The reinforcement is placed where the stress field is assumed to develop.

Struts

Figure 2.10 shows the compression strut C2 acting on the node resulting in a compression stress σ_{c2}. Same as for σ_{c1}, the compression stress σ_{c2}, should fulfill a stress criterion. If the criterion is not met, unfavourable multi-axial effects could occur. The compression stress σ_{c2} is calculated depending on the width a_2. a_2 is determined as:

$$a_2 = (a_1 + \frac{u}{\tan\theta}) \sin\theta = a_1 \sin\theta + u \cos\theta \tag{2.8}$$

If the compression strut is affected by transverse compressive stress the design stress in the strut can be calculated as:

$$\sigma_{Rd,max} = f_{cd} \tag{2.9}$$

If the strut is located in a cracked compression zone the strength of the concrete should be reduced and calculated according [30] as:

$$\sigma_{Rd,max} = 0.6 \, v \, f_{cd} \tag{2.10}$$

2.3 Finite element method

The finite element method is a numerical technique for finding approximate solutions to complex partial differential equations. In simple terms, FEM is a method for dividing a very complicated problem into small elements that can be solved in relation to each other, the differential equations are formulated in such a way that they can be solved approximately with a computer, see Figure 2.11 [19].

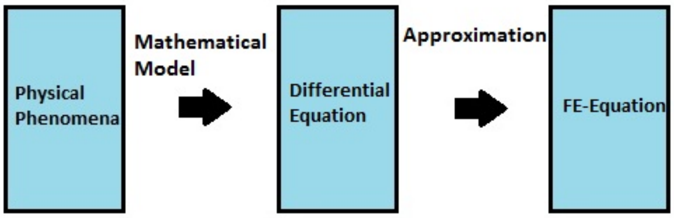


Figure 2.11: Steps when analyzing with FEM

The method involves dividing a structure into a finite number of elements. There are three main element types; beam (line), shell (surface) and solid elements (volume). Within the element the differential equation is approximated, usually with a polynomial. The polynomial is defined by a number of points in the element, these points are called nodes. The nodes are placed on the element boundaries, but sometimes also inside the element. In FE-formulation the values of the polynomial in the nodes are unknown and form a system of equations. Depending on which physical phenomenon that is being studied, each node contains a number of unknown variables. One variable in a node is called a degree of freedom. For describing thermal conductivity it is sufficient with one degree of freedom of each node, namely temperature. Structural mechanics problems with shell elements, where the displacements of nodes is of interest, require six degrees of freedom per node, namely three translations and three rotations [19]. The main challenge when solving partial differential equations is to generate an equation that approximates the equation to be studied, but is numerically stable, and that errors in the input and intermediate calculations do not accumulate and cause the resulting output to be meaningless. The finite element method is a useful option when solving partial differential equations over complicated domains [19].

The equation for describing static linear elastic problem takes the form:

$$K = f\,a$$

K is the stiffness matrix and depends on the materials and types of elements, a is the displacement vector containing the degrees of freedom of the structure and f is the force vector containing the forces acting on the structure. In order to describe the size of the equation system the number of degrees of freedom in the structure is specified [19].

Figure 2.12: Steps in FEM-program [19]

FE-software is often divided into three parts, see Figure 2.12 above. In the pre-processor the FE equations are prescribed. For that purpose the nodes, elements, storage, materials and loads are specified. In the pre-processor an automatic element subdivision is made. Pre-processor often has a graphical interface, which facilitates accurate data entry. The system of equations is solved by the FE solver and results illustrated in the post-processor through charts, isolines etc [19].

Since FEM is an approximation the result of a FEM calculation will be an estimation of the solution to the original differential equation. The result is more consistent with the real solution if the structure is divided into smaller elements or modeled with element types of higher polynom range. Both procedures mean that the number of degrees of freedom increases in the model, which will make the equation system grow. [19].

2.4 Software

The two programs compared in this study are Fem-Design 11 and Brigade / Plus. Both programs are able to analyze and design slabs, walls, columns and beams. Both programs can handle the Swedish structural codes and can analyze complex structures such as 2D frames and 3D structures with various materials such as concrete, steel and wood [37] [27].

Both programs have an easy to understand user interface, with the benefit of Fem-Design which is very user friendly. The two programs both have a graphic window where the geometry can be plotted. It is possible for both programs to define a variety of load types. Fem-Design handles the breakdown of the structure by itself so that an optimal finite element mesh with the built in mesh generators is obtained. It is also possible to build up an own mesh or adjust the automatically generated. Within Brigade / Plus meshing must be done by the user [27] [37].

Moreover both programs have roughly similar functions for postprocessing. In Fem-Design calculation there are various available results, such as moments, shear forces, normal forces, deflections, crack widths and so on [27] [37].

2.5 FEM-design

2.5.1 Overview

FEM-Design is an advanced software for modelling FE-analysis and design of load-bearing structures made of concrete, steel or timber. Particular elements or a whole building, made from any amount of materials and structural elements can be simulated with ease. Advanced analysis such as static, dynamic, global stability, seismic analysis, concrete and steel design calculations are possible to model with Fem-Design and can be run for the complete 3D-model. Fem-design is based on several tabs that the user goes through when creating a model. These tabs are Structure, loads, finite elements, analysis, rc design, steel design and timber design. [38].

2.5.2 Structure

When creating a model, the structure tab contains the tools for defining axies, storeys, structural elements, supports and connections. Each tab opens further options that can be configured after the need of the model [38].

2.5.3 Loads

Tab that contains the tools for creating different load cases, load group and load combinations [38].

2.5.4 Finite elements

Tools to define the finite element mesh of the model. FEM-design use beam, rectangular and triangular finite elements [38].

2.5.5 Analysis

When the model is ready to be analyzed several options of which analysis is to be made can be chosen. When the analysis is done the result is displayed under this tab [38].

2.5.6 RC design, steel design and timber design

Tools to design concrete elements, steel bars or timber elements and to display the result of the design [38].

2.6 BrigadePlus

2.6.1 Overview

Brigade/Plus is an advanced software for modeling all types of bridges and civil structures. The program is an extension of the finite element program ABAQUS. Brigade/Plus takes account of predefined loads, vehicle load and load combinations and includes a wide variety of design codes containing the Eurocodes with National Annexes. Advanced analysis such as transient and steady state dynamic analysis, non-linear material models and contact interactions are possible.

Brigade/Plus is built-up of different modules. The various modules handle different parts of the modeling process. Below is a description of the various modules and how they are defined [25].

2.6.2 Part module

The part module defines the model geometry. The geometrical model can be created in many different ways. , depending on if the model ought to be based on shell, beam or solid elements [27].

2.6.3 Property module

The property module defines material and sectional properties [27].

2.6.4 Assembly module

When a part of the model is created in BRIGADE it exists in its own coordinate system independently of other parts of the model. The Assembly module brings together the created parts into one model with a common coordinate system [27].

2.6.5 Step module

The next step is to determine how the model will be analyzed. In Step Manager it is possible to select for the analysis to be static or dynamic, and whether the analysis should take into account geometric nonlinearity [27].

2.6.6 Interaction module

The Interaction module defines the interaction between different parts of the model. This type of interactions can be defined differently for the different steps of the analysis, as determined in the Step module. It is therefore possible to choose one kind of interaction in the static stage, and then define a new interactions in the dynamic step [27].

2.6.7 Load module

All loads that are acting on the model are applied in the Load module. The Load module also defines the boundary conditions [27].

2.6.8 Mesh module

In the Mesh module the element breakdown of the model is made. There are a numeral ways to create element divisions depending on how the model looks like and what kind of selected element form. Examples of element shapes are triangular and quadrilateral elements. There are also element forms for the element division of the 3-D models [27].

2.6.9 Job module

The model that is created is analyzed in the job module. The job manager creates a job, which afterwards is calculated in the solver [27].

Chapter 3

Method and Result

The aim of this chapter is to make the reader familiar with the method used to achieve the result. The results are presented in connection with the related section.

3.1 Loads

According to [33], the safety class for elements in this kind of building can be class II or class III.Safety class III is here assigned for the design of loads. When designing loads, there are three combinations of actions that can be chosen. Due to the concrete structure of the building, the permanent load will have a great impact on which of these three combinations to choose. The combination of actions originate from equation 6.10 in [32].

A building can be exposed to several types of loads. The loads considered in this thesis are self-weight, live loads, snow loads and wind loads. Due to the location of the building, seismic loads will not be included. Live loads are dependent on what the building is used for whereas self-weight, snow loads and wind loads are actions of natural causes.

Applying [32] equation 6.10, safety class III selected with $\gamma_d = 1.0$. The load combination factors used for the calculation of design loads are illustrated in table 3.1.

Equation 6.10a gives the largest loads and is therefore used as the design load. Equation 6.10a is displayed in equation 3.1. In table 3.2 the characteristic loads for the building to be used in equation 3.1 is illustrated. Equation 6.10a give the largest design loads in the present application due to the large self-weight loads of the structure. Several other load combinations are calculated and given Appendix 10.2.

The load of inner walls with $q_{installation} = 0.5kN/m^2$are added to the live load according to [34].

$$Q_d = \gamma_d \, 1.35 \, G_{kj,sup} + \gamma_d \, 1.5 \, \psi_{0,i} Q_{k,i} \tag{3.1}$$

Table 3.1: Load combination factors for different types of load

Type	Reduction factor ψ_0
Snow	0.6
Wind	0.3
Live load	0.7

Table 3.2: Characteristic loads acting on the building.

Loads	Parameters	Load value	According to
Self-weight, slab	$\gamma_{con} = 25kN/m^3$ $t_{slab} = 0.22m$	$G_{slab} = 5.5kN/m^2$	[33]
Self-weight, wall	$\gamma_{con} = 25kN/m^3$ $t_{wall} = 0.2m$ $h_{wall} = 2.53m$	$G_{shellwall} = 12.65kN/m$	[33]
Self-weight, outerwall	$\gamma_{brick} = 19kN/m^3$, $t_{brick} = 0.12m$ $\gamma_{minerite} = 21kN/m^3$, $t_{minerite} = 0.005m$ $\gamma_{plaster} = 15kN/m^3$, $t_{plaster} = 0.013m$ $\gamma_{iso} = 1.4kN/m^3$, $t_{iso} = 0.195m$	$G_{outerwall} = 31.4kN/m$	[33]
Self-weight, column	$\gamma_{con} = 25kN/m^3$ $A_{column} = 0.3m^2$ $h_{column} = 3.68m$	$G_{column} = 8.3kN$	[33]
Snow, drifted	$s_k = 1kN/m^2$ $\mu_1 = 0.8kN/m^2$ $\mu_s = 0.4kN/m^2$ $\mu_w = 1.23kN/m^2$ $\mu_2 = 1.23kN/m^2$ $\mu_m = 1.466kN/m^2$	$Snow_{drift} = 1.5kN/m^2$	[31]
Live load	$q_k = 2.0kN/m^2$ $q_{installation} = 0.5kN/m^2$	$q_{liveloadtot} = 2.5kN/m^2$	[33]
Wind load	$v_b = 26m/s$ $q_p = 1.27kN/m^2$ Terrain 0	$q_{wind} = 1.27kN/m^2$	[33]

Table 3.2 contains the characteristic loads that are acting on the building due to building geometry and where the building is located geographically. The calculation for the loads and the design values of loads can be found under Appendix 10.2. The loads in table 3.2 are combined according to equation 6.10 in [32] and are displayed for equation 6.10a in table 3.3. Equation 6.10a gives the worst loads and therefore determines the design load. Note that only vertical loads are applied in the 2D-model.

Table 3.3: Combination of actions according to [32]. Result design loads

Equation	Design load, 2D-model	Design load, 3D-model
6.10a Ultimate limit state	$q_{d1} = 147.8kN/m$ $q_{d1,shellwall} = 68.3kN/m$ $p_{d1,outerwall} = 152.5kN/m$ $p_{d1,column} = 11.2kN$	$q_{snow} = 1.32kN/m^2$ $q_{Liveload} = 2.63kN/m^2$ $q_{wind} = 0.57kN/m^2$

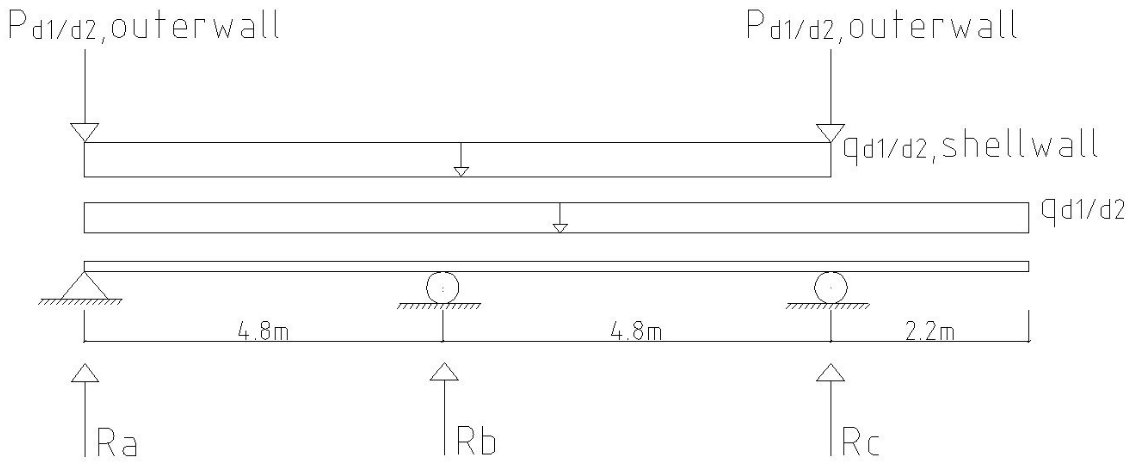


Figure 3.1: Placement of design loads in a 2D-model

Placement of design loads are illustrated in figure 3.1. Design loads according to table 3.3 are modified to a span of 3.6 meter

3.2 Strut and Tie Calculations

3.2.1 Assumptions and simplification

When using the strut and tie method some parameters have to be chosen. Before calculation of reinforcement the reaction forces in the supports have to be determined. To be able to use simple calculations, the geometry of the structure must be simplified. The structure is divided into ten 3.6 m spans by walls. Suppose that each wall carries a load from a 3.6 meter influence length down to the columns a simplified model of the problem can be described. With consideration to the cantilever part of the slabs the problem can be assumed as a elastic beam on three support, see figure 3.1. The reactions is then determined by linear elastic distribution of the load see subsection 3.2.2 [9].

3.2.2 Hand calculations

In Appendix 10.3 the reaction forces calculated by hand and the Strut and Tie calculations can be found.

Table 3.4 contains the result from the hand calculations.

Table 3.4: Reaction forces of the columns

Support	Force (kN)
R_a	560
R_b	1193
R_c	977

3.2.3 Calculation of reinforcement and stress control

The high shell wall is similar to a high beam and a good approximation is to use of the Strut and Tie method. With applied recommendations according to section 2.2.9, figure 3.2 illustrates the struts and ties selected for this case. The support in the middle is divided into two parts depending on the magnitude of the shear force from each side of the support. The cantilever slabs are included in the loads, therefore Rc will be greater.

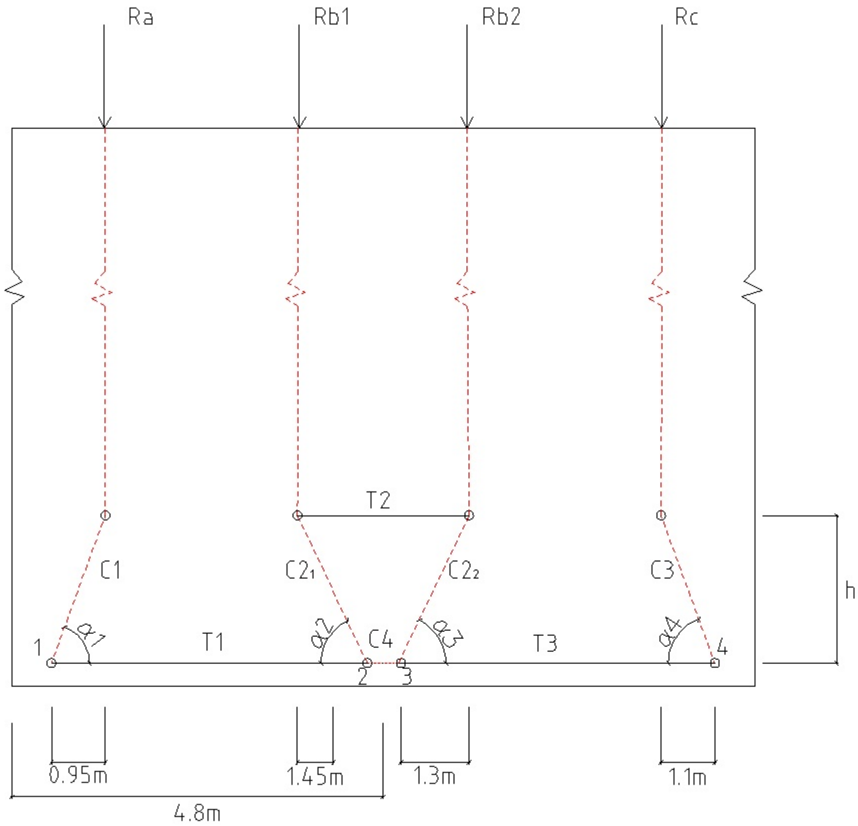


Figure 3.2: Simplified model of the shell wall. Crosshatched lines define struts and solid lines ties

In Appendix 10.3 the obtained reinforcement layout required for the building according to Strut and Tie can be found.

The strut and tie calculations gave the following reinforcement:

Table 3.5: Required reinforcement

Field reinforcement,AB (T1)	5∅12
Reinforcement over support (T2)	7∅12
Field reinforcement, BC (T3)	9∅12

The reinforcement layout will serve as input in the nonlinear FE-analysis. According to appendix 10.4 the concrete column will have a reinforcement of 4∅12 on each side. Stirrups will also be added.

3.3 Sensitivity analysis, assumptions and parametric study

To identify uncertainties when modeling the structure, sensitivity analysis and a parametric study will be performed. There are a large number of parameters defining a model and some of them are used in the parametric study to illustrate their influence over the result of the models.

3.3.1 Positioning of the columns in the model

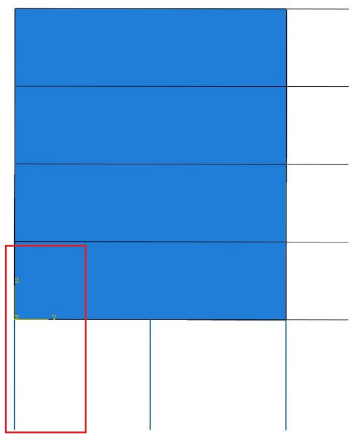


Figure 3.3: The analyzed part of the building

A factor that is of importance to analyze is the positioning of the columns in the model. If large differences between the three cases in figure 3.4 are obtained one should take this into consideration in the simulation model. Three cases have been evaluated and the part of the building that the analysis is referred to is illustrated in figure 3.3. It is of interest to study the deformation of the columns in the three cases. The stiffness is related to the deformation that will occur in the column when a load is applied according to equation 3.2. The known column have a spring stiffness of 880 MN/m according to equation 3.5. The system will behave as a serial spring coupling according to equation 3.3 [5]. The spring stiffness of the coupling between column and structure can be calculated according to 3.3.

$$K_{system} = \frac{F}{y} \ N/m \tag{3.2}$$

Where F is the applied force and y is the deformation outcome.

$$\frac{1}{K_{system}} = \frac{1}{K_{column}} + \frac{1}{\Delta K} \ N/m \tag{3.3}$$

In case 1 the column is attached at the outermost node in the shell wall. In case 2 the columns mid-point is attached 0.15 m from the edge of the wall and a very stiff, weightless beam is tied between the column and the edgenode and in case 3 the columns is modelled as a shell element with a thickness of 0.3 m. The beam attached in case 2 corresponds to width of a 0.3 m column and reduces concentrated deformations from the column. A concentrated force of 1 kN is applied in the first and second case and a edge load of 3.333 kN/m is applied on the third case. The edges of the walls have been applied as pinned boundary condition. Figure 3.4 illustrates the three cases simulated in Brigade/Plus

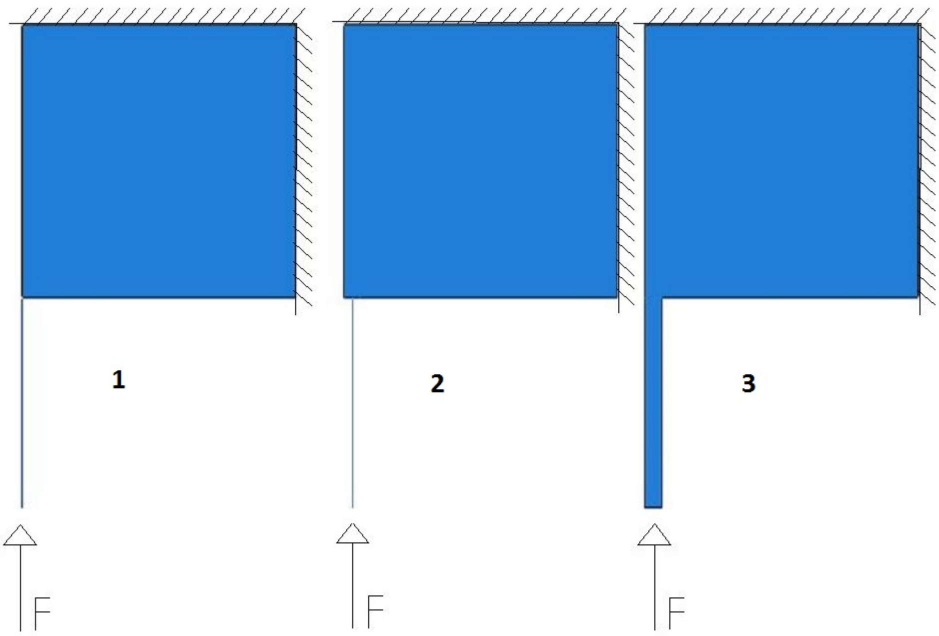


Figure 3.4: The three cases of positioning and simulating a column

Table 3.6: Deformation related to K spring stiffness

Case	y (m)	ΔK MN/m	$K_{column} MN/m$
1	0.0024	0.42	880
2	0.00189	0.53	880
3	0.0016	0.63	880

Since the obtained results according to table 3.6 not differ significantly the different positioning of the column will not be taken into account in further simulations. ΔK will have a significant role if taken into account however since this a global analysis of a system no details will be taken into account. The chosen case for the analysis of the building is case 1.

3.3.2 Non-rigid motion of supports

The ground mainly consists of filling material and due to that the building is supported with concrete piles. The piles are about 10 meters long and have the dimension $0.27x0.27m^2$. According to [42] the piles will withstand a load of 700 kN. The reaction forces calculated by hand are calculated in subsection 3.2.2 and the number of piles required to support the columns is illustrated in table 3.7.

Table 3.7: Required number of piles

Support	Piles
R_a	1
R_b	2
R_c	2

The piles will work as springs when subjected to a load. The non-rigid spring stiffness K can be approximated as a 10 meter long column. If it is assumed that the piles have a slight lower concrete quality, for example C20, the pile spring stiffness is calculated according to:

$$k = \frac{EA}{L} \tag{3.4}$$

$$K_{ground} = \frac{EA}{L} = \frac{30\,GPa \cdot 0.27^2 m^2}{10m} \approx 219MN/m$$

To compare the effect of the non-rigid piles, various pile setups are analyzed in the 3D-models. One according to table 3.7 and one with same spring stiffness at each column stated in Table 3.8. The number of piles stated in table 3.8 are supporting the columns. This spring stiffness is used in the finite element analysis, described in section 3.7 and 3.8.

Table 3.8: Three different setups of piles, label according to section 3.2.2

Support	Piles		Support	Piles
Ra	1		Ra	2
Rb	2		Rb	2
Rc	2		Rc	2

When analyzing 2D-models the stiffness of the columns will be simulated with a spring stiffness K_{column}. The columns have the same concrete strength as the rest of the building. The first floor have a height of 3.9 meters and without the thickness of the slab, according to table 2.1, the columns have a height of 3.68 meters. The spring stiffness of a column in the bottom floor is calculated as:

$$k = \frac{EA}{L} = \frac{36\,GPa \cdot 0.30^2 m^2}{3.68m} \approx 880MN/m \tag{3.5}$$

The sensibility analysis and parametric study also include comparison of linear and nonlinear material behavior of the structure. Nonlinear analysis includes cracking of concrete. The simulation programs used in the thesis will iterate equilibrium based on several load steps. The load will be applied in several steps and for each step, if cracking of the concrete occur, the program reduces the concrete stiffness. [36].

When the non-rigid motion is analyzed the boundary condition changes for the columns and the edge walls. The local rotations for the columns are free in the x- and y-direction, while the rotation around the lengthwise direction of the edge wall is free.

3.3.3 Parametric study

The parametric study contains several different tests and in every test different parameters are toggled on or off. Table 3.9 illustrates several different tests which are going to be applied to the models in Fem-design and Brigade/Plus. A short description of the parameters in the table 3.9:

- Result number
Non-rigid motion of ground - If the model is simulated with non-rigid properties or not. If the model is analyzed with non-rigid properties the type according to table 3.8 will be stated.
Connection between structural elements - The interaction between the structural elements can be stated with rigid or pinned. The connections that will be modeled are between slab-walls and slab-pillars, though continuous slab will be assumed. Figure 3.5 illustrate the connection between a slab and a wall.

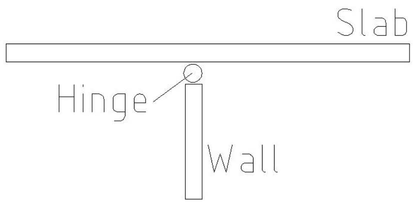


Figure 3.5: A pinned connection between slab and wall

Connection ground - States what kind of connection the columns have to the ground. If non-rigid motion of the ground is used, the box will be filled with non-rigid. rigid of pinned options will be used.
Reaction column row - The largest reaction forces of every column rows in the structure. R_a,R_b and R_c is positioned as figure 3.6. The two columnrows, marked with a square in figure 3.6 are the rows with most load. The reaction force in the table are the maximum reaction forces of one of the columnrows. Nonlinear results are inside the bracket.
Reaction edge wall - The reaction force of edge walls.

Figure 3.6 illustrate the index of each column and edge walls.

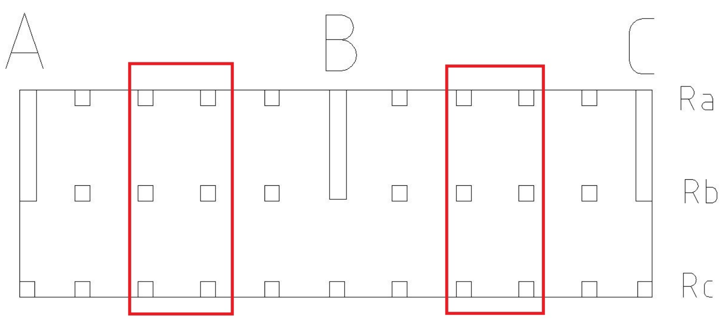


Figure 3.6: Simplified schematic view over building. Illustrating the names of column rows and edge walls.

Table 3.9: Parametric study 1 - 3D model with various inputs

#	Non-rigid support on ground	Connection between structural elements. rigid/pinned	Connection ground. rigid/pinned/non-rigid	Reaction columnrow (nonlinear)	Reaction Edge walls (nonlinear)
1	No	rigid	rigid	Ra Rb Rc	A B C
2	No	rigid	pinned	Ra Rb Rc	A B C
3	No	pinned	rigid	Ra Rb Rc	A B C
4	No	pinned	pinned	Ra Rb Rc	A B C
5	Yes, type 1-2-2	rigid	pinned	Ra Rb Rc	A B C
6	Yes, type 1-2-2	pinned	pinned	Ra Rb Rc	A B C
7	Yes, type 2-2-2	rigid	pinned	Ra Rb Rc	A B C
8	Yes, type 2-2-2	pinned	pinned	Ra Rb Rc	A B C

The result of the parametric study obtained from Fem-Design can be found in section 3.7 and from Brigade/Plus in section 3.8.

3.4 2D-Model

The 2D-models will only be analysed with Frame analysis and Fem-Design. Only vertical loads will be applied.

3.4.1 Spring stiffness in ground and column

The spring stiffness Kwill differ depending on which case is simulated. There will be four cases of a 2D-model simulated and these are illustrated in table 3.10 according to figure 3.6. The spring stiffness will vary in each case depending on if only the column is simulated or the ground is included. If the ground is included in the analysis the spring stiffness will be 175 MN/m according to equation 3.3 where $\Delta K = K_{ground}$ according to section 3.3. When two K_{system} are mentioned, two piles beneath the columns are used and the spring stiffness is then $\Delta K = 2 \cdot K_{ground}$. K_{system} is then calculated to 292 MN/m.

$$\frac{1}{K_{system}} = \frac{1}{880 \cdot 10^6} + \frac{1}{219 \cdot 10^6} \Longrightarrow 175MN/m$$

$$\frac{1}{K_{system}} = \frac{1}{880 \cdot 10^6} + \frac{1}{2 \cdot 219 \cdot 10^6} \Longrightarrow 292MN/m$$

Table 3.10: Five difference setup of spring stiffness

Support	K		Support	K		Support	K
Ra	Infinite		Ra	1 K_{column}		Ra	1 K_{system}
Ra	Infinite		Rb	1 K_{column}		Rb	1 K_{system}
Ra	Infinite		Rc	1 K_{column}		Rc	1 K_{system}

Support	K		Support	K
Ra	1 K_{system}		Ra	2 K_{system}
Rb	2 K_{system}		Rb	2 K_{system}
Rc	2 K_{system}		Rc	2 K_{system}

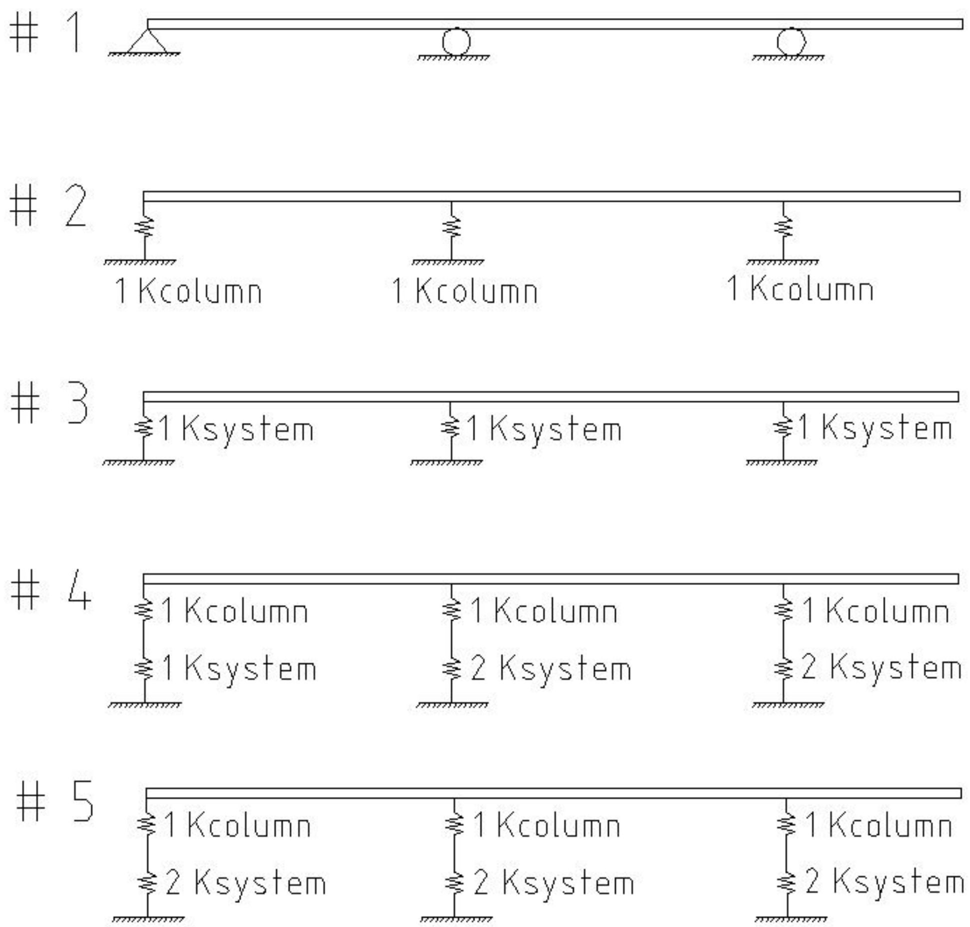


Figure 3.7: Illustrations of the different simulated 2D-models

3.4.2 Frame-analysis

An easy tool to use for a 2D-model is the program Frame-analysis. The geometry used in the 2D-analysis is illustrated in Figure 3.8 and consists of three supports and a cantilever part and is the same simplified structure as in the hand calculations. The model will also be analyzed with spring stiffness at the supports to simulate the spring stiffness in the columns and ground. This is done so that the result can be compared to the non-rigid motion test in the 3D-models.

Figure 3.8: Frame-analysis model

The properties of the beam is modeled as a HEA1000 which will give a very high bending stiffness to resemble the sturdy shell wall in the 3D-structure.

3.4.3 FEM-design

To compare the hand calculations and the 3D-models in FEM-design a 2D-model in FEM-design is simulated. Figure 3.9 illustrates the model as a beam with three supports and a cantilever part. Dimension of the beam and positions of supports according to appendix 10.1.

The properties of the beam is modeled as a HEA1000 which will give a very high bending stiffness to resemble the sturdy shell wall in the 3D-structure.

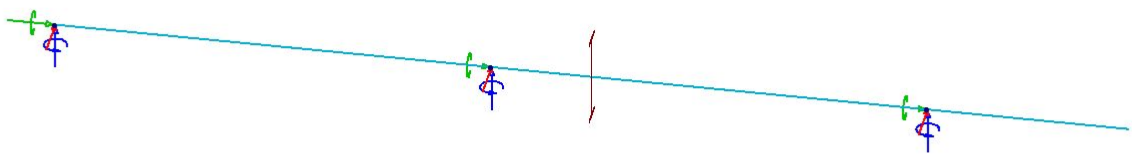


Figure 3.9: Fem-Design 2D-model

3.5 Result of 2D-models

In the table 3.11 the results from the 2D-models are assembled.

Table 3.11: Result of 2D-model simulation

	K spring stiffness at support	Result Frame-analysis (kN)	Result Fem-design (kN)
#1			
Ra	infinite	561	562
Rb	infinite	1188	1182
Rc	infinite	961	962
#2		$\sum 2710$	$\sum 2710$
Ra	1 K_{column}	575	576
Rb	1 K_{column}	1160	1156
Rc	1 K_{column}	975	976
#3		$\sum 2710$	$\sum 2710$
Ra	1 K_{system}	611	611
Rb	1 K_{system}	1088	1086
Rc	1 K_{system}	1011	1011
#4		$\sum 2710$	$\sum 2710$
Ra	1 K_{system}	580	580
Rb	2 K_{system}	1150	1146
Rc	2 K_{system}	980	981
#5		$\sum 2710$	$\sum 2710$
Ra	2 K_{system}	596	596
Rb	2 K_{system}	1118	1115
Rc	2 K_{system}	996	996
		$\sum 2710$	$\sum 2710$

From table 3.11 conclusions can be drawn that the effect of non-rigid supports can be neglected.

3.6 Comparison of 2D-model results

Since the test Martin Fröderberg carried out contained comparisons between hand calculations and computer simulated models it is of interest to analyze the relationship between the two. Even if one could think a comparison between the both is not valid since simple hand calculation is a very simplified model of the problem.

In order for the results to be comparable, the results have been divided depending on whether it is a hand calculation, a 2D-model without non-rigid motion and a 2D-model with non-rigid motion from the columns or from the columns and the ground. In order to compare 2D-models between the different calculations, the hand calculations before adding moment from wind and self-weight from column must be used.

Table 3.12: Comparison between hand calculations and 2D-models without non-rigid motion.

	Hand calculation - Frame analysis	Hand calculations -Fem-design
Ra	560-561=-1	560-562=-2
Rb	1185-1188=-3	1185-1182=3
Rc	960-961=-1	960-962=-2

Table 3.13: Comparison between hand calculations and 2D-models with non-rigid motion 1-1-1 K_{column}.

	Hand calculation - Frame analysis 1-1-1 K_{column}	Hand calculations -Fem-design 1-1-1 K_{column}
Ra	560-575=-15	560-576=-16
Rb	1185-1160= 25	1185-1160=25
Rc	960-975=-15	960-976=-16

Table 3.14: Comparison between hand calculations and 2D-models with non-rigid motion 1-1-1 K_{system}.

	Hand calculation - Frame analysis 1-1-1 K_{system}	Hand calculations -Fem-design 1-1-1 K_{system}
Ra	560-611=-51	560-611=-51
Rb	1185-1088=97	1185-1086=99
Rc	960-1011=-51	960-1011=-51

Table 3.15: Comparison between hand calculations and 2D-models with non-rigid motion 1-2-2 K_{system}.

	Hand calculation - Frame analysis 1-2-2 K_{system}	Hand calculations -Fem-design 1-2-2 K_{system}
Ra	560-580=-20	560-580=-20
Rb	1185-1150=35	1185-1146=39
Rc	960-980=-20	960-981=-21

Table 3.16: Comparison between 2D-models without non-rigid motion and 2D-model with non-rigid motion 1-1-1 K_{column}

	Frame analysis - Frame analysis 1-1-1 K_{column}	Fem-design -Fem-design 1-1-1 K_{column}
Ra	561-575= -14	562-576 =-14
Rb	1188-1160=28	1182-1156=26
Rc	961-975=-14	962-976=-14

Table 3.17: Comparison between 2D-model with non-rigid motion 1-1-1 K_{column} and 2D-model with non-rigid motion 1-1-1 K_{system}

	Frame analysis 1-1-1 K_{column} - Frame analysis 1-1-1 K_{system}	Fem-design 1-1-1 K_{column} -Fem-design 1-1-1 K_{system}
Ra	575-611= -36	576-611=-35
Rb	1160-1088=72	1156-1086=70
Rc	975-1011= -36	976-1011=-35

Table 3.18: Comparison between 2D-models with non-rigid motion 1-1-1 K_{system} and 2D-models with non-rigid motion 1-2-2K_{system}

	Frame analysis 1-1-1 K_{system} - Frame analysis 1-2-2K_{system}	Fem-design 1-1-1 K_{system} -Fem-design 1-2-2K_{system}
Ra	611-580=31	611-580=31
Rb	1088-1150 =-62	1086-1146=-60
Rc	1011-980= 31	1011-981=30

Table 3.19: Comparison between 2D-models without non-rigid motion and 2D-models with non-rigid motion 1-1-1K_{system}

	Frame analysis - Frame analysis 1-1-1K_{system}	Fem-design -Fem-design 1-1-1K_{system}
Ra	561-611= -50	562-611=-49
Rb	1188-1088=100	1182-1086=96
Rc	961-1011= -50	962-1011=-49

Table 3.20: Comparison between 2D-models without non-rigid motion and 2D-models with non-rigid motion 1-2-2K_{system}

	Frame analysis - Frame analysis 1-2-2K_{system}	Fem-design -Fem-design 1-2-2K_{system}
Ra	561-580=-19	672-580=92
Rb	1188-1150= 38	1038-1146=-108
Rc	961-980= -19	997-981=16

A behaviour obtained between all systems in table 3.12- 3.19 was that Ra and Rc carried more load when a spring stiffness was applied on each support. Unlike Ra and Rc, Rb decreased its reaction force.

3.7 Fem-design 3D-model

3.7.1 Structure

Fem-design uses shell elements which have both displacement and rotational degrees of freedom. 3D shell is used to model the slabs and walls. The shell element is capable of calculation in plane and perpendicular to plane displacements. A 3D shell has six degrees of freedom, displacement and rotation in global X, Y and Z directions.

The columns are modeled as 3D beams which applies Timoshenko beam theory. 3D beams have six degrees of freedom, displacement and rotation in global X, Y and Z directions. The slab in the model connects to the columns with a pinned boundary condition. The connection between walls and slabs are also pinned.

The Timoshenko beam theory allows the effect of transverse shear deformation which cannot be neglected when analyzing deep beams. Timoshenko beam theory assumes that plane sections remain plane but not necessarily normal to the longitudinal axis after deformation [43].

The six degree of freedom point support, describes the connection between columns and ground. The model is analyzed with both a rigid and pinned connection. When the grounds non-rigid properties are modeled, the support spring stiffness in Z-direction is changed to the spring stiffness according to section 3.3. The different pile setup, see table 3.8, is applied on each span in the whole structure when analysing non-rigid motion of the ground.

All structural elements is assigned a concrete strength of C45 according to 2.1.2.

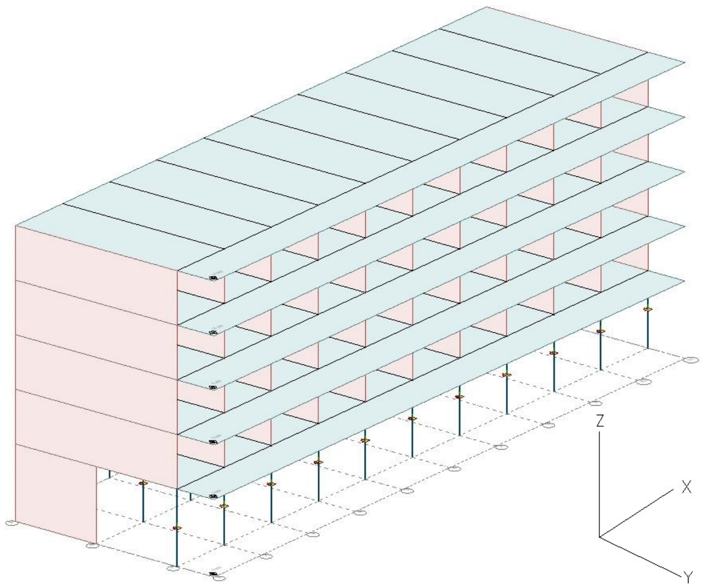


Figure 3.10: Illustrate the Fem-Design 3D-model

3.7.2 Loads

Many of the loads in Fem-Design are automatically generated for the structure when a load is toggled on. The wind load, according to [35], is applied on the long side and on the gable. FEM-design has a tool for automatic generation of wind loads but it has not been used in this analysis. Live load is applied and the snow load is adjusted to suit the design loads according to table 3.3. Load combinations are then created to be used in the analysis.

Two combinations are used in the analysis. Both combinations include self-weight, live load and snow load. One analysis includes the wind load on the gable and the other analysis includes the wind load on the long side. The wind load is equally distributed along the height.

3.7.3 Finite elements

The finite element settings are altered to get the best and most accurate results. The element sizes are selected to approximate $0.5x0.5m^2$. The element type is described in subsection 3.7.4.

Line element parameters define the accuracy of the result of an element. The parameters describe how many elements a beam element will contain when divided by a neighboring element. To achieve an accurate calculation, this parameter is chosen to be a 6 [36].

3.7.4 Analysis

An accurate finite element type is used and an additional node is added on each side of the element. The elements are described with a quadratic geometric order, see figure 3.11. The analysis will be calculated for both linear and non-linear material behavior.

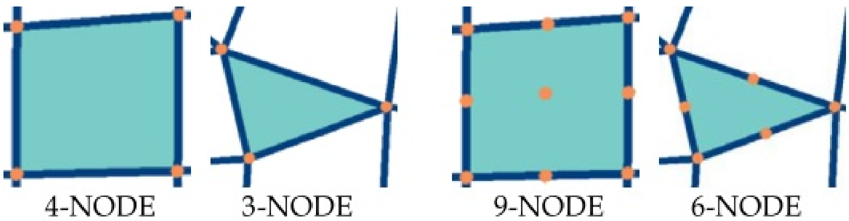


Figure 3.11: Illustrate the difference between quadratic element order and standard element order

3.7.5 RC design, steel design and timber design

According to the strut and tie calculations the concrete in the shell walls will have a reinforcement of ∅12. This size is also applied in FEM-design. The program checks if the selected reinforcement is enough to withstand the forces. If not, the program will add additional reinforcement bars defined by the user. The columns also have stirrups of ∅8 s100.

The analysis is made to calculate the forces in the columns and edge walls. It is not made to calculate the design of the reinforcement.

3.7.6 Result of Fem-Design 3D-model

In the table 3.21 and table 3.22 the results from the Fem-Design 3D-model is assembled.

Table 3.21: Parametric study 1 - 3D model with various input- Wind load applied at long side

#	Non-rigid motion of ground	Connection between structural elements. rigid/pinned	Connection ground. rigid/ pinned/non-rigid	Reaction columnrow (kN) (nonlinear)	Reaction edge wall (kN) (nonlinear)
1	No	rigid	rigid	Ra=618(603) Rb=1167(1181) Rc=1239(1237) $\sum$3024(3021)	A=1794(1803) B=2796(2821) C=1794(1796) $\sum$6384(6420)
2	No	rigid	pinned	Ra=618(609) Rb=1166(1181) Rc=1238(1236) $\sum$3022(3026)	A=1787(1801) B=2785(2821) C=1792(1795) $\sum$6364(6423)
3	No	pinned	rigid	Ra=610(623) Rb=1261(1200) Rc=1233(1273) $\sum$3104(3096)	A=1497(1531) B=2765(2862) C=1567(1591) $\sum$5829(5978)
4	No	pinned	pinned	Ra=598(607) Rb=1263(1261) Rc=1236(1193) $\sum$3097(3061)	A=1497(1530) B=2765(2862) C=1562(1585) $\sum$5824(5977)
5	type 1-2-2	rigid	pinned	Ra=646(645) Rb=1202(1207) Rc=1160(1154) $\sum$3008(3006)	A=1565(1569) B=2044(2056) C=1564(1567) $\sum$5173(5192)
6	type 1-2-2	pinned	pinned	Ra=644(646) Rb = 1253(1247) Rc=1194(1200) $\sum$3091(3093)	A=1388(1399) B= 2102(2177) C=1388(1397) $\sum$4878(4973)
7	type 2-2-2	rigid	pinned	Ra=710(706) Rb=1080(1086) Rc =1217(1217) $\sum$3007(3009)	A=1487(1492) B=2026(2041) C=1485(1489) $\sum$4998(5022)
8	type 2-2-2	pinned	pinned	Ra=778(735) Rb=1171(1131) Rc=1210(1128) $\sum$3159(2994)	A=1320(1359) B=1526(1618) C=1354(1424) $\sum$4200(4401)

Table 3.22: Parametric study 1 - 3D model with various input - Wind load applied at gable

#	Non-rigid motion of ground	Connection between structural elements. rigid/pinned	Connection ground. rigid/ pinned/non-rigid	Reaction columnrow (kN) (nonlinear)	Reaction edge wall (kN) (nonlinear)
1	No	rigid	rigid	Ra=630(616) Rb=1166(1181) Rc=1226(1225) $\sum$3016(3022)	A=1805(1815) B=2820(2845) C=1832(1834) $\sum$6457(6494)
2	No	rigid	pinned	Ra=630(624) Rb=1166(1184) Rc=1226(1224) $\sum$3022(3032)	A=1800(1810) B=2821(2849) C=1836(1795) $\sum$6457(6454)
3	No	pinned	rigid	Ra=610(638) Rb=1261(1200) Rc=1222(1254) $\sum$3093(3085)	A=1520(1540) B=2765(2848) C=1567(1602) $\sum$5852(5990)
4	No	pinned	pinned	Ra=611(621) Rb=1263(1250) Rc=1222(1197) $\sum$3096(3068)	A=1519(1538) B=2791(2849) C=1588(1601) $\sum$5898(5988)
5	type 1-2-2	rigid	pinned	Ra=663(660) Rb=1205(1208) Rc=1140(1141) $\sum$3008(3009)	A=1579(1583) B=2026(2076) C=1601(1605) $\sum$5224(5264)
6	type 1-2-2	pinned	pinned	Ra=661(633) Rb=1257(1247) Rc=1175(1181) $\sum$3093(3061)	A=1360(1376) B=2124(2174) C=1452(1436) $\sum$4936(4986)
7	type 2-2-2	rigid	pinned	Ra=727(722) Rb =1080(1087) Rc =1200(1200) $\sum$3007(3009)	A=1501(1507) B=2048(2065) C=1525(1532) $\sum$5074(5104)
8	type 2-2-2	pinned	pinned	Ra =687(710) Rb=1147(1132) Rc=1272(1264) $\sum$3106(3106)	A=1320(1375) B=1542(1583) C=1355(1408) $\sum$4217(4366)

By analysing table 3.21 and 3.22 conclusion can be drawn that nonlinear calculations almost equals the linear calculations. Other conclusion is that the reaction forces does not significantly change between the different boundary conditions. When non-rigid supports are analysed the edge walls will carry less load and the columns will automatically carry more load.

3.8 Brigade/Plus

3.8.1 Part module

Brigade should first and foremost be used for analyzing the reaction forces in the columns. The structure is to be simulated with stress/displacement elements. There are several types of stress/displacement elements and for this model, shell elements are appropriate since the thickness of the parts that are going to be modeled is significantly smaller than the other dimensions. The mesh generation and mesh option is dependent of which type of parts that are used for the model. The shell element have both displacement and rotational degrees of freedom [27].

The columns are modeled as beam elements with a square section of $0.3 \cdot 0.3\ m^2$. The beam elements are appropriate to choose when the height of the member is significantly larger than the section of the member. Figure 3.12 illustrates the model of the structure.

A weightless extremely stiff beam is attached under the three edge walls in order to resemble a stiff plate and to prevent deformation of the walls geometry. Large plastic areas will occur if the beam had not been attached and the calculation would not converge.

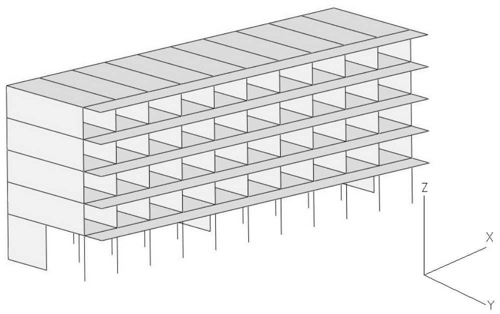


Figure 3.12: Brigade model of the structure

3.8.2 Property module

The structure is made of concrete and to mimic an appropriate concrete material the concrete damaged plasticity model is used in Brigade. The concrete damaged plasticity model uses isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behavior of a reinforced structure.

The model assumes that the main failure mechanism is tensile cracking and compression crushing. Figure 3.13 illustrates the behavior of damaged plasticity concrete. The top graph illustrates concrete exposed to uniaxial tension and the graph below illustrates the concrete exposed to compression [27].

The tension in the concrete create micro-cracking in the concrete material and after the failure stress σ_{t0} the stress-strain response softening. In the compression graph the concrete is characterized by stress hardening and after initial σ_{c0} the stress-strain response is softening [27].

The plastic part of the concrete behavior is described by several input parameters. All parameters to describe concrete damaged plasticity are default values according to section 3.28 in [28], except of the dilation angle which is recommended to 31° [14].

The compressive behavior of concrete in the property module is defined as the yield stress and the corresponding inelastic strain. First value is the initial yield stress according to 3.1.5 in [30].

$$\sigma_{c0} \;=\; 0.4 \cdot f_{cm} = 0.4 \cdot 53MPa = 21.2\,MPa$$

The inelastic strain is calculated according to 20.6.3 in [27]

$$\varepsilon_{0c}^{el} = \sigma_c/E_0 \;=\; \frac{53 \cdot 10^6}{36 \cdot 10^9} \;=\; 0.00147$$

$$\varepsilon_c^{inel} \;=\varepsilon_{cu} - \varepsilon_{0c}^{el} = 0.00350 \;-\; 0.00147 = 0.00203$$

The tensile behavior is described by a module in Brigade/Plus that is called GFI which is based on the yield stress in tension and the fracture energy. The yield stress is mentioned in subsection 2.1.2. According to [8] the fracture energy is calculated as:

$$G_f \;=\; G_{F0} \cdot (\frac{f_{cm}}{f_{cmo}})^{0.7} \tag{3.6}$$

Where $G_{F0}$is the base value of fracture energy based on the maximum aggregate size according to table 3.1-3 [8]. In this case the maximum aggregate size is assumed to 32 mm which give a $G_{F0} = 0.058N/mm$. f_{cmo} = 10 MPa.

$$G_f \;=\; 0.058 \cdot 10^3 \cdot (\frac{53 \cdot 10^6}{10^6})^{0.7} \;\approx\; 182\,N/m$$

The elastic part of the material is described as an isotropic material with a Young's modulus, density and Poissons's ratio according to section 2.1.2.

To mimic the response of the structure, reinforcement is necessary. This is applied in Brigade as reinforcement layer in the elements [27]. Reinforcement for the whole structure is chosen to ∅12 with a spacing of 0.1 m in vertical and horizontal alignment. The reinforcement have a Young's modulus and Poissons ratio of 0.2[30]. The density of the reinforcement is in Brigade/Plus selected to 0 kg/m^3 due to that the density of concrete is selected as a density of reinforced concrete according to [30].

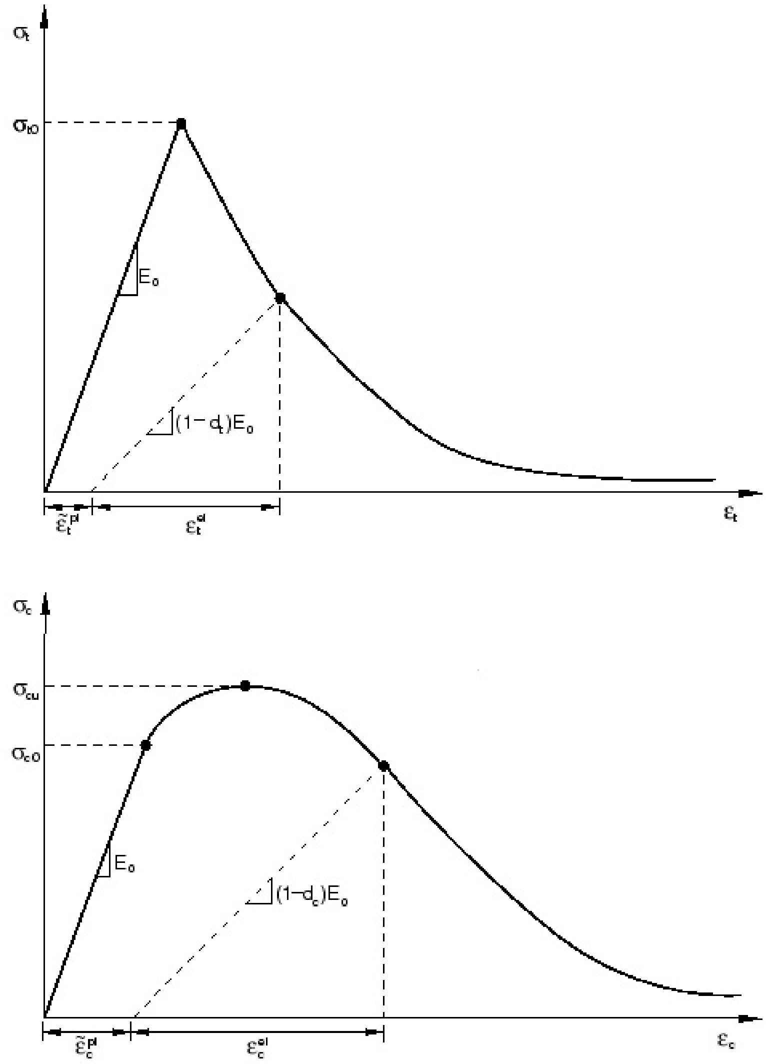


Figure 3.13: Damaged plasticity of concrete

3.8.3 Assembly module

The model is assembled into one big part. This affects the connection between structural elements. More on the interaction between structural element is given in subsection 3.8.5 [27].

3.8.4 Step module

The model is analyzed with only small deformations. The model will not be analyzed for stabilization.

3.8.5 Interaction module

The connections between the beams and the body of the building are tied together and automatically make the global displacement and rotation equal at two nodes and the connection in the body between structural elements will be rigid. This rigid connection in the building makes it very sturdy [27].

3.8.6 Load module

The design loads used in the calculation are shown in table 3.3. There are two types of wind loads that are analyzed. One when wind is applied to the gable and one when it is applied to the long side [27]. When applying the wind load to the long side an extra line load has to be added because of the area by the columns are not included in the pressure load. The first slab is applied with an extra load according to Appendix 10.2:

$$Wind_{bot} = \frac{1.052kN \cdot length\ of\ building,\ m}{number\ of\ nodes\ on\ longside\ of\ slab}$$

$$Wind_{bot} = \frac{1.052 \cdot 36}{72} = 0.526\,kN$$

3.8.7 Mesh module

When meshing the model, 'structured' technique and quadrilateral elements are used. Quadrilateral elements are defined as a four sided polygon. Structured meshing contains a pre-established mesh pattern to particular model topologies. This gives a high control over the meshing of the model. The element size is approximately 0.5·0.5 meter depending on how well the mesh is created. Figure 3.14 below illustrates the meshed structure [27].

The beam elements applied to the columns are shear flexible. Shear flexible is also known as Timoshenko beams. For more background information about Timoshenko beam theory see subsection 3.7.1.

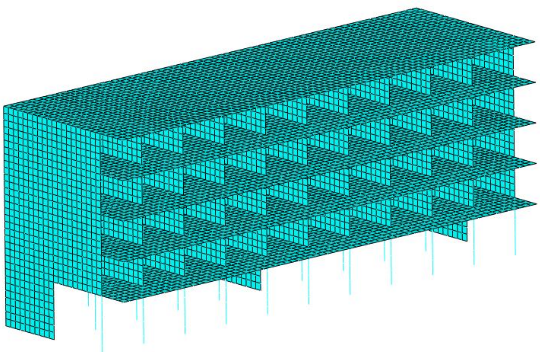


Figure 3.14: Meshed model

The model will be calculated with quadratic geometric order instead of linear geometric order. This means that, on each side of the element and in the middle of the element, an extra node is added. This results in a more advanced calculation, but at the same time it also provides a more accurate result, see Figure 3.11 for comparison [27].

3.8.8 Comparison of boundary conditions

The analysis of a pinned connection between structural elements in Brigade/Plus is going to be neglected. Since Brigade/Plus is a far more complex program to define geometry, in the Brigade/Plus model will only be analyzed using rigid condition in the connection between structural elements. To illustrate that this is a valid assumption it is necessary to do a comparison of results from Fem-design and Brigade/Plus. In test number 1 according to table 3.9, Brigade/Plus and Fem-Design obtain reaction forces of the columns and the edge wall according to table 3.23.

Table 3.23: Difference between Fem-design and Brigade/plus reaction forces

	Fem-design (kN)	Brigade/plus (kN)	Difference
Ra	618	644	-26
Rb	1167	1172	-5
Rc	1239	1252	-13
A	1794	1840	-46
B	2796	2870	-74
C	1794	1830	-36

The difference is small in comparison with the reaction force and therefore it is reasonable to only use Fem-Design with the pinned boundary condition between structural elements.

3.8.9 Result of Brigade/Plus 3D-model

In the table 3.24 and table 3.25 the results from the Brigade/Plus 3D-model is assembled.

Table 3.24: Parametric study 1 - 3D model with various input- Wind load applied at long side

#	Non-rigid motion of ground	Connection between structural elements. rigid/pinned	Connection ground. rigid/ pinned/non-rigid	Reaction columnrow (kN) (nonlinear)	Reaction shellwall (kN) (nonlinear)
1	No	rigid	rigid	Ra=644(644) Rb=1172(1173) Rc=1250(1252) $\sum$3066(3069)	A=1840(1839) B=2870(2870) C=1853(1853) $\sum$6563(6562)
2	No	rigid	pinned	Ra=647(647) Rb=1174(1174) Rc=1247(1246) $\sum$3067(3068)	A=1817(1816) B=2809(2808) C=1830(1830) $\sum$6456(6454)
3	No	pinned	rigid	Ra=- Rb=- Rc=- $\sum$	A=- B=- C=- $\sum$
4	No	pinned	pinned	Ra=- Rb=- Rc=- $\sum$	A=- B=- C=- $\sum$
5	type 1-2-2	rigid	pinned	Ra=639(639) Rb=1183(1183) Rc=1170(1170) $\sum$2992(2992)	A=1399(1399) B=1908(1908) C=1408(1408) $\sum$4715(4715)
6	type 1-2-2	pinned	pinned	Ra=- Rb = - Rc=- $\sum$	A=- B= - C=- $\sum$
7	type 2-2-2	rigid	pinned	Ra=736(736) Rb =1059(1059) Rc =1191(1191) $\sum$2986(3009)	A=1372(1372) B=1940(1940) C=1383(1383) $\sum$4695(4695)
8	type 2-2-2	pinned	pinned	Ra=- Rb=- Rc=- $\sum$	A=- B=- C=- $\sum$

Table 3.25: Parametric study 1 - 3D model with various input - Wind load applied at gable

#	Non-rigid motion of ground	Connection between structural elements. rigid/pinned	Connection ground. rigid/ pinned/non-rigid	Reaction columnrow (kN) (nonlinear)	Reaction shellwall (kN) (nonlinear)
1	No	rigid	rigid	Ra=651(651) Rb=1172(1172) Rc=1244(1244) $\sum$ 3067(3067)	A=1900(1899) B=2887(2886) C=1830(1830) $\sum$ 6617(6615)
2	No	rigid	pinned	Ra=654(654) Rb=1174(1174) Rc=1239(1234) $\sum$ 3067(3067)	A=1797(1796) B=2825(2825) C=1887(1886) $\sum$ 6509(6507)
3	No	pinned	rigid	Ra=- Rb=- Rc=- $\sum$	A=- B=- C=- $\sum$
4	No	pinned	pinned	Ra=- Rb=- Rc=- $\sum$	A=- B=- C=- $\sum$
5	type 1-2-2	rigid	pinned	Ra=647(647) Rb=1184(1184) Rc=1160(1160) $\sum$ 2991(2991)	A=1388(1388) B=1908(1919) C=1408(1444) $\sum$ 4704(4751
6	type 1-2-2	pinned	pinned	Ra=- Rb=- Rc=- $\sum$	A=- B=- C=- $\sum$
7	type 2-2-2	rigid	pinned	Ra=736(736) Rb =1059(1059) Rc =1191(1191) $\sum$ 2986(2986)	A=1372(1372) B=1940(1940) C=1383(1383) $\sum$ 4695(4695)
8	type 2-2-2	pinned	pinned	Ra =- Rb=- Rc=- $\sum$	A=- B=- C=- $\sum$

By analysing table 3.24 and 3.25 conclusion can be drawn that nonlinear calculations equal the linear calculations. Other conclusion is that the reaction forces does not significantly change between the different boundary conditions. When non-rigid supports are analysed the edge walls will carry less load and the columns will automatically carry more load.

3.9 Comparison of 3D-model results

In this step the Fem-Design models have been compared with the Brigade/Plus simulated models in four steps. First the linear simulation is compared, and then the non-linear and last the non-rigid motion simulations that have been carried out in the two software have been compared. The difference in pinned alternative comparison between Fem-design and Brigade/Plus are based on a percentage difference from the result obtained in table 3.27. The percentage difference in test 3 is based on test number one according to comparison of boundary condition section 3.8.8, table 3.23.

Table 3.26: Comparison of load cases between Fem-Design and Brigade/Plus

	Fem-Design (kN)	Brigade/Plus (kN)
Self-weight	26520	26514
Live load	4460	4460
Snow load	560	560
Sum	31540	31534

Table 3.27: Comparison between 3D-model in Fem-design and brigade according to numbering in table 3.9

#	Fem-design supports	Brigade/plus support	Difference	Fem-design Egde wall	Brigade/plus Egde wall	Difference
1	Ra=618(603) Rb=1167(1181) Rc=1239(1237)	Ra=644(644) Rb=1172(1173) Rc=1250(1252)	-26(-41) -5(8) -11(-15)	A=1794(1803) B=2796(2821 C=1794(1796)	A=1840(1839) B=2870(2870) C=1853(1853)	-46(-36) -74(-49) -59 (-57)
2	Ra=618(609) Rb=1166(1181) Rc=1238(1236)	Ra=647(647) Rb=1174(1174) Rc=1247(1246)	-29(-38) -8(7) -9(-10)	A=1787(1801) B=2785(2821) C=1792(1795)	A=1817(1816) B=2809(2808) C=1830(1830)	-30(-15) -24(-13) -38(-35)
5	Ra=646(645) Rb=1202(1207) Rc=1160(1154)	Ra=639(639) Rb=1183(1183) Rc=1170(1170)	7(6) 19(24) -10(-16)	A=1565(1569) B=2044(2056) C=1564(1567)	A=1399(1399) B=1908(1908) C=1408(1408)	166(170) 136(148) 156(159)
7	Ra=710(706) Rb=1080(1086) Rc =1217(1217)	Ra=736(736) Rb=1059(1059) Rc =1191(1191)	-26(-30) 21(27) 26(26)	A=1487(1492) B=2026(2041) C=1485(1489)	A=1372(1372) B=1940(1940) C=1383(1383)	115(120) 86(101) 102(106)

Chapter 4

Discussion and conclusions

The discussion and conclusion intends to give the reader an idea of how the models work and to give hands-on tips on what to consider when designing using FEM. In this chapter the results will be evaluated, discussed and explained. It ought to be made clear that the number of comparisons and the numerous parameters that changes in the parameter study are maybe too few in order to achieve any general conclusions. The conclusion that has been established should be taken as an indication of how a simulated model ought to be established in order to obtain how a model will behave in "reality"

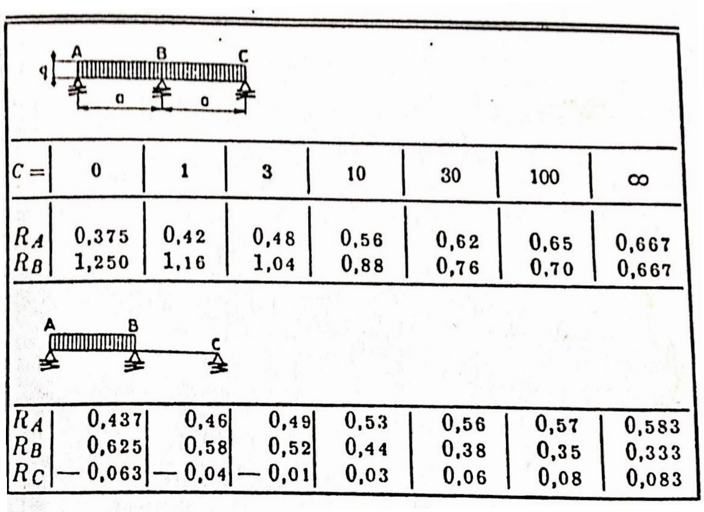
One main objective was to create a model, in all programs, that would simulate the responses of the structure set in the task both in 3D-models and 2D-models. To analyze a model that ought to have the same load calculation behaviour in the hand calculations as well as in the computer simulated models is not possible since the hand calculations do not take into account the entire structure.

4.1 2D-model

The results from the 2D-model without non-rigid motion indicate that the hand calculation very well coincides with the 2D-model from frame analysis and Fem-design. Although the result obtained from Brigade/Plus differs quite more than the other results it is still a good approximation and differs only with maximum 10 %. The hand calculation is a good method to quickly determine the maximum reaction forces which also correspond well to the FE-analysis. The hand calculations and the 2D-model with non-rigid motion 1-1-1 K_{column} coincide well with Frame analysis and Fem-Design. When analysing the comparison between the hand calculations and the 2D-model with non-rigid motion 1-1-1 K_{system} the result begin to differ between hand calculations and Frame analysis and Fem-Design. Though a comparison between the hand calculations and the non-rigid motion 1-2-2 K_{system} seem to coincide better than with 1-1-1 K_{column} and 1-1-1 K_{system}. The reason for why the obtained results differed more when a non-rigid motion was applied can be described through that the beam is now allowed to more or less rotate. Though the 1-2-2 K_{system} describes that the support Ra will gain less load since the support will appear less stiffer than the 1-1-1 K_{column} and 1-1-1 K_{system} which is distributed to the support Rb. The fact that Rc gets smaller is due to less rotation due to more stiffness in Rb and Rc.

A reason for why the difference from the obtained result are so small can be described by the chosen HEA1000 beam in the models. In reality the shell wall is much more stiffer than a HEA1000. Though when analysing the 3D-models one can see that the difference in the results still are small.

A significant annotation that was obtained between all system between table 3.12- 3.19 was that Ra and Rc carried more load when a spring stiffness was applied on each support. Unlike Ra and Rc, Rb decreased its reaction force. This relationship can be described by taking a closer look at figure 4.1. The figure illustrates the relationship C between the stiffness in the beam and the stiffness in the supports. In a scenario without a non-rigid motion system the stiffness in the supports are infinitely large compared to the stiffness in the beam which will lead to a certain distribution of load according to figure 4.1. When non-rigid motion is given and the motion is illustrated by a spring stiffness the relationship between the beam and the supports will change. The beam now have infinite large stiffness properties compared to the spring supports which will generate a different distribution of load, see figure 4.1. HEA1000 will have a C value of 2.9, see appendix 10.5. The situation that has been analyzed in the 2D-models in the thesis is similar to the case in figure 4.1 and one can therefore draw conclusion that the relationship can be applied in our example though without exact load distribution [41].



C =	0	1	3	10	30	100	∞
R_A	0,375	0,42	0,48	0,56	0,62	0,65	0,667
R_B	1,250	1,16	1,04	0,88	0,76	0,70	0,667
R_A	0,437	0,46	0,49	0,53	0,56	0,57	0,583
R_B	0,625	0,58	0,52	0,44	0,38	0,35	0,333
R_C	− 0,063	− 0,04	− 0,01	0,03	0,06	0,08	0,083

Figure 4.1: The relationship between the stiffness in the beam and the stiffness in the supports [41]

One should keep in mind that Frame analysis and Fem-Design is produced from the same software developer, Strusoft, and therefore the 2D-models calculation steps ought to be alike and the output obtained will be similar.

4.2 3D-model

The results achieved from the linear 3D-model analysis, test 1 -4, indicate that the Fem-Design model very well coincides with the Brigade/Plus model.

Though, when the non-rigid motion is analyzed in the different models there seem to appear a few changes in the results. Test number 5 with non-rigid motion 1-2-2 K_{system} still coincide between Fem-Design and Brigade/Plus though in this step the major differences in reaction forces are obtained. In test number 7 the reaction forces in the columns and edge walls between the software coincide better than in test number 5 though slight differences are gained. The reason for why the obtained results differed more in test number 5 compared to test number 7 when a non-rigid motion is applied is because in test number five 5 there is not an equal stiffness distribution applied though an equal stiffness distribution is applied in test number 7. The big difference in reaction forces in the edge walls between Fem-design and Brigade/Plus when studying the result with non-rigid motion are that in Brigade/Plus the edge walls will carry less load and the difference in load will be carried by the columns. The reader should keep in mind that not only the featured columns in table 3.27 are the ones that will carry the extra load but the extra load will be distributed and carried of all the 27 columns.

Another important aspect to study in the result in test number 5 and 7 is the differences in distribution between the reaction forces in the columns and the edge walls from the software's. One should keep in mind that our simulation refers for a specific chosen column row and can differ when analyzing other column rows. However the important fact is that the total reaction forces obtained from the different software's are equal according to table 3.26. The results from the model when the non-rigid motion of the ground was taken into account makes it possible for us to establish that the reaction forces in the edge walls are significantly smaller compared to without non-rigid motion. This will contribute that the extra load will be carried and distributed to the columns.

The nonlinear analysis is more complex to study though the applied plastic- and elasto-plastic material behaviour differ between the software's. In Fem-Design it is very easy to carry out a nonlinear analyze just by allowing cracking in the concrete. But in Brigade/Plus the user have to attach several material properties which can lead to major differences in the outcome. This is an important aspect when analyzing uncertainties. The easy to use property in Fem-Design makes it more or less impossible for a standard user to control the different steps that will lead to the final results. Whereas Brigade/Plus's nonlinear material module is very advanced for a standard user. In the analysed structure the results illustrates that linear and nonlinear models coincide well. Since the nonlinear effect can be neglected there will not be any significant cracking in the concrete.

The reason for why an equal result is obtained in the linear and nonlinear analysis in Brigade/Plus can be described by the large attached fracture energy in the material step for the concrete properties. Smaller fracture energy could be chosen though when we attached smaller fracture energy several areas in the model could not converge due to large plasticity in the material. With chosen fracture energy with the value of 96 N/mm the model could only converge up to a load of 70 percent of the total load. Therefore we chose to use bigger aggregate size in the concrete and chose higher fracture energy with the value of 182 N/mm.

The pinned connection properties between structural elements seem to change the distribution of load. When the structure is modeled with a pinned property the wind load will influence the outcome of the reaction forces in the columns as much as when an rigid property is chosen but it is harder to spot in the analysis though the distribution of load will appear different. This statement can be described by studying figure 4.2.

When analyzing the wind load applied on the gable the reaction forces on the right half of the structure according to figure 3.6 appear larger than the ones on the opposite side due to the moment created by the wind load. An important aspect to discuss is the fact that the Rc column in majority of the test have the largest reaction force. An explanation to this behaviour can be described by assuming the whole system as a rigid

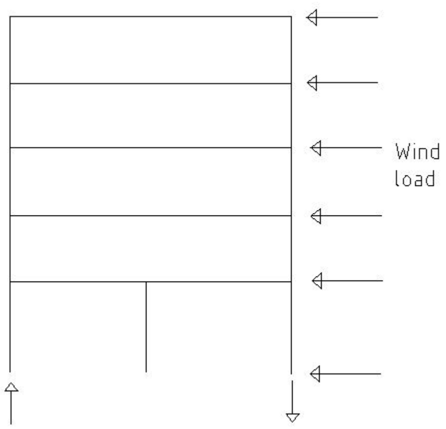


Figure 4.2: Schematic section illustrating the moment created by the wind load when a rigid connection between structural elements is chosen.

body motion. Since that the center of gravity of the entire structure is located illustrated in figure 4.3 a moment will develop due to the eccentricity which will lead to that Rc carry a greater load.

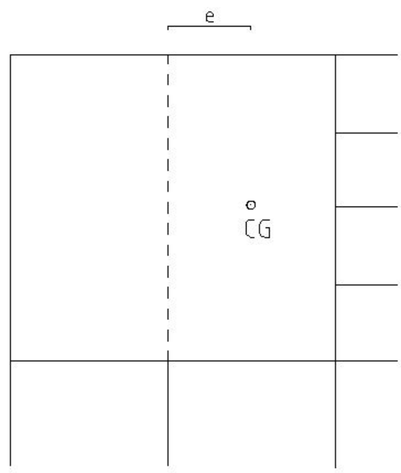


Figure 4.3: Schematic section illustrating the center of gravity

4.3 Comparison of model

A big difference between the 2D-model and the 3D-model is the distribution of the load. In the 2D-model Rb will carry the largest load compared to the other two columns while the reaction force will be largest in Rc in the 3D-model when analyzing rigid connection between structural elements. The main reason for the difference in the distribution is the chosen HEA1000 beam since the stiffness of a HEA1000 beam is much smaller than the stiffness of the concrete shell wall. If a beam with the properties of a 1000 times HEA1000 was chosen the distribution of load will behave much more similar to the distribution of load obtained in the 3D-model. Figure 4.4 illustrates the issue, the Young's modulus manipulated to about 1000 times the ordinary Young's modulus for steel.

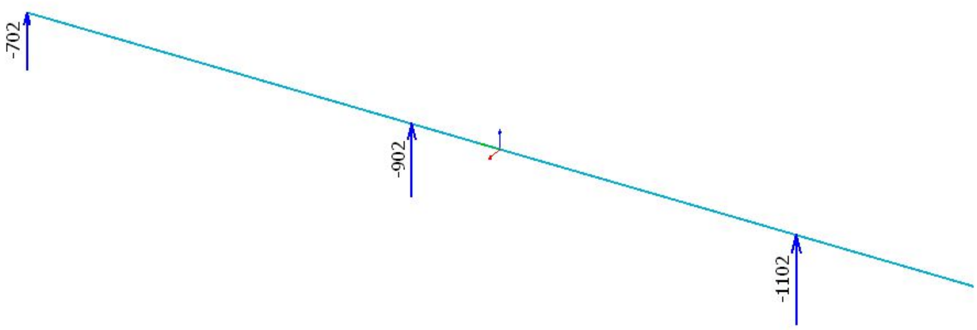


Figure 4.4: 2D-model with a HEA1000 with a scaled up Young's modulus

The reader is reminded of that these analyses have been carried out without taking regard to the stability analyzes of the structure. The structure ought to collapse since there is no stabilization lengthwise. This issue could be handled through installing a horizontal bracing. A big problem when analysing with regard of stability is that the model can be analysed and result can be obtained without a proper stabilization system. In that way one can analyse a model where there is a risk for collapse.

4.4 General notes when modeling

When modeling 2D- and 3D-models it is of importance to consider a few fundamental though very important issues. Some of these issues are listed below.

- Connection between structural elements. When modeling rigid or pinned connection the structure will behave differently in terms of load distribution though the reaction forces does not significantly change between the different boundary conditions. Pinned connections do not consider wind loads.
- When non-rigid supports are analysed the edge walls will carry less load and the columns will automatically carry more load.
- Connections between structural elements will never behave 100 percent rigid or pinned in reality. When constructing a connection in reality this issue have to be taken into account.
- When modeling 2D-models in aspect of non-rigid motion of the support and increasing the spring stiffness proportional in all supports the load distribution change insignificantly.
- The choice of material model will determine the behavior of the structure for example whether the concrete will crack or not. One can see in the result an insignificantly difference in reaction forces between linear and nonlinear behavior.
- It is of importance to choose the most appropriate software for the specific problem that ought to be analysed.

This is the end of the FE-study and the beginning of the quality control process

Part II

Quality Control

Chapter 5

Quality control: Background and basic concepts

In this chapter the reader is introduced to the master thesis quality control part. Background and basic concepts are described. The quality control process has been divided into two parts. The first part contains literature studies and the second part contains interviews with people in the industry that work with simulation or quality control on a daily basis.

5.1 Quality control in construction

All businesses have a minimum set of standards and quality that their product or service has to encounter. Quality control in construction is a set of procedures that is used and followed to confirm that the required level of quality in a service are met. It can contain whatever actions a company considers is essential to provide for the control and confirmation of certain characteristics of service. Mostly it includes in detail examining and testing the quality of the outcomes of services. The fundamental aim of this procedure is to confirm that the services that are delivered encounter specific requirements and characteristics, for instance being reliable, satisfactory and safe [46].

The aim of quality control is to find the services that do not fulfill specified criteria of quality, e.g drawings. [46]. Quality control is often mixed up with quality assurance. The two concepts are similar but there are some basic differences. Quality control is the method when examining the service, the end result. Quality assurance is the method when examining the process that leads to the end result.

A business would use quality assurance to guarantee that a service is executed in the right way, in that way reducing or excluding possible problems with the quality of the end result [46]. Quality control is important since it confirms that the required quality is maintained during the entire process [40].

5.1.1 Assuring good quality control

Documentation is also known as good record keeping. This is a key in assuring good quality control and the inspector should always remember that. Some factors that can affect documentation are societies expectation of direct response and gratification and that students learn through education that a deadline must be held regardless if you have done a quality check or not. Other factors that can have a significant role are the fact that the construction business has unrealistic expectations on architects and engineers regarding the time needed to produce correct drawings and specifications. Haste in planning and lack of time to carry out a document check adequately will generate problems throughout the bidding and construction phases. A group needs time to check, cross-check and organize all aspects of the drawings including architectural, structural, civil, landscape architecture, interior design, mechanical, electrical, technology and safety of the drawings [24].

5.1.2 History

Until 1986, Sweden had a different building regulation from today. The first version of the current planning and building act was introduced in 1987. Many changes were made since then in terms of inspection and control of buildings which has resulted in a great deal of impact on how the design and planning is made today [45].

Sweden had once a different set of building regulations with so called municipality building inspections. Companies had to submit their documents to a building committee and building inspectors verified the documents. The documents submitted were reported according to a certain standard, depending on the case it referred to. Building inspectors acted as an external inspector who examined both drawings and calculations made by the designer. Building inspectors had no responsibility that the documents were accurate, that was the structural engineer responsible of. However the inspector would let the structural engineer know if something was not approved so that the mistake could be corrected [45].

The process could differ from municipality to municipality. For example, Helsingborg executed every step of the examination process very carefully and it was extremely difficult for the engineering firms to have their documents approved without changes, while for instance in Falkenberg most cases were approved without further investigation [45].

In late 1986 the first version of the current planning and building act emerged. This indicated clearly that the client had responsibility for the entire project. The mandatory external municipal inspection was deregulated [45].

Later in 1995 so called quality assurance managers were established. The quality assurance manager assigned under the planning and building act is not expected to monitor the overall quality of the project, but only the part of the contractor's control system designed to ensure that the essential requirements are met. The quality assurance manager assignments do not manage or supervise the work itself but on the clients behalf oversee the enforcement of society's requirements [45].

5.1.3 Quality assurance manager

In the construction business in Sweden there is a system of certified quality assurance managers. Their role is to help the client to establish a proposed quality assurance plan and ensure that the quality assurance plans are followed. According to the Swedish current building codes, a certified quality assurance manager is responsible for many construction-demolition and land operations. Small changes in for example one-or two store -family houses, and the measures in respect to outdoor buildings like garages and other small structures do not require a quality assurance manager, unless otherwise is decided by the local housing committee. Recently the quality assurance manager has been replaced by a control assurance manager [3].

5.1.4 Requirements for a control assurance manager

The requirements concerning former quality assurance managers have recently undergone some changes. The Swedish National Board of Housing has e.g. proposed that the competence requirement will also include consumer and contracting laws.According to the new planning and building act, PBL (2010:900) the control assurance manager must be certified. This is stated in Chapter 10. § 9 of the new Planning and building act [3].

5.1.5 ISO 9001

ISO 9001:2008 sets out the conditions for a quality management system. It is the only standard in the ISO 9000 family that one can be certified to, although it is not compulsory. Any kind of organization can become certified, large or small, regardless of its field of activity. The goal of the quality management system is continuous improvements i.e. the business must constantly strive to become more efficient [10].

5.1.6 Other countries view on quality assurance

Germany has so called Prüfstatiker performing thorough inspections of building documents. The inspector must have completed a number of operations such as having experience and having participated in advanced projects to earn the title and the inspector must control everything during the process. The examination process is divided into two parts. During the first examination only the main drawings without any detailing are inspected. The first examination must be approved before the Structural Engineer can move on with the work. The second part of the examination verifies that construction documents are correct and that approvals from the first examination have been followed. Construction cannot take off until the second control is approved. Denmark has a similar system as in Germany. The documentation papers must be presented and approved by a Building Committee before construction can start [45].

5.1.7 Projects gone wrong

In Sweden there have been a number of collapses in the last years due to flaws in the design, among them the Kista mall and a building in Ystad. The collapse in Kista mall occurred at the construction site in July 15th, 2008. Several concrete slabs fell down because the beam holding up the concrete plates, collapsed due to that the web of the beam was too thin. The beam was supposed to have a web of 25 millimeters but the dimension was changed to 7 millimeters. One construction worker was killed and a motorist passing by was buried under the wreckage [15].

The same year that the Kista case occurred, a formwork for a bridge construction at Botniabanan collapsed and two people were killed. During the snowy winters of 2009/10 and 2010/11 in the southern and central Sweden, hundreds of roofs collapsed. Many laymen believe that the collapses occurred due to the extreme amount of snow. But in virtually all cases, the errors in the structures were revealed when the snow load suddenly became high [6].

On May 25th 2012 a nearly finished building in Ystad collapsed. Preliminary research of the collapse showed that three steel columns on the ground floor were too slender. The collapse occurred primarily due to the column closest to the facade reached its ultimate load [1].

In today's construction and planning of buildings and in the process that this is done, it is easy for many errors of more or less serious nature to occur. There are a great number of parties involved in the building process and in many cases there is no one with overall control of the technical aspects. The current planning and building act states clearly need for carrying capacity, stability and durability (chapter 3, 7§), but without requirements of control or sanctions from the society. This is a result of a far-reaching deregulation of society's control and supervision of building, which began more than 20 years ago. The idea of the deregulation was to enable innovative thinking and new ideas but the consequence is that it compromises peoples safety [6].

It is necessary to introduce more strict exercise of authority to gain control of the building process. Planning and building acts and its rules must be defined in a way so that clear and substantial demands in areas such as documentation of stability, independent technical control and follow-up can be made. Documentation of stability and load-carrying capacity for buildings should be fully recorded. To minimize serious mistakes a qualified third person ought to review the documentation. It is needed to institute clear requirements for systematic follow-up of already occurred collapses and more serious damages [6].

5.2 Documentation

When companies want to improve their business processes it is crucial that the company document the process and any improvements made to it. A decent documentation a consultant can perform is to document both the "As-Is Process" as well as the "To-Be Process" [39]. The process usually starts with the analysis of the current 'As-Is' situation, see Figure 5.1. The goal is to reach a better process in terms of inter alia quality, efficiency. Often the hardest part is actually to achieve a change. However, without an accurate picture of the 'As-is' process the business cannot effectively measure the progress.

There are a lot of important reasons why companies ought to document the process. Documentation helps the organization gain long term primary and secondary benefits [23].

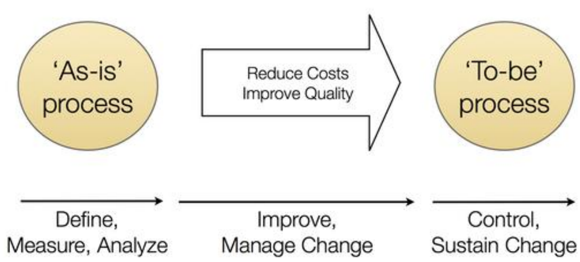


Figure 5.1: Process improvement [23]

5.2.1 Why is documentation of importance?

There are a lot of important reasons why companies ought to document the process. Documenting helps the organization gain long term primary and secondary benefits. Below are some examples of the primary benefits [23].

Documenting a process helps reducing operational ambiguity. Since a detailed documentation contains information about the entire operation, misunderstandings can therefore be sorted out. These documents function as the store of cooperative organizational knowledge regarding the processes and can be read by anyone in times of need. Another fact that is good to consider when documenting is that it [39] enables the possibility to use processes that have been carried out earlier [23]. Documentation can also help ease the learning process when new resources join the group and will probably move up the learning curve faster [39]. Documentation also assures continuity of process and, for that reason, the continuity of quality that the process represents. The procedure makes it possible to consciously examine processes to improve them, and can make it possible for an organization to learn from the past [18].

5.2.2 Secondary Benefits

Certain secondary benefits can help the company analyze and improve its process continuously. Once a process changes and the change is documented in a detailed manner it will become available for analysis when required. This helps the management in understanding the knowledge that was used when designing the most suitable practices that have been applied. If a company documents their processes accurately the process improvements can be tracked version to version. In that way the management will be able to study their performance along with the current process and performance. In that way it becomes possible to analyze which specific changes are responsible for which specific outcomes [39].

Chapter 6

Method

The aim of this chapters is to let the reader get familiar with the method used in order to acheive the result of the quality control part.

6.1 Quality control

6.1.1 Interview

The persons that have been interviewed have different background and work for different companies. A total of eight interviews have been carried out with the total of ten attendants and the questions that were asked were adapted after the person who was interviewed. All ten person(s) has agreed to publish their opinion in this thesis.

The interviewees were:

Roberto Crocetti, Professor, department of Structural Engineering, The faculty of engineering, Lund University [4]
Thomas Kamrad, Structural Engineering in bridges, Centerlöf och Holmberg [12]
Martin Larsson,Structural engineer NCC [13]
Jan Nord, Regional Director, Sweco [16]
Mikael Rosengren, Structural engineer, Structor AB [22]
David Persson, Structural engineer, Centerlöf och Holmberg [20]
Mats Persson, Business Director, Tyréns [21]
Mats Skällenäs, Head of department, Industry, Centerlöf och Holmberg [29]
Kristian Widén, Tekn Dr at the division of Construction Management, Lund University and CEO at SKF[44]
Jan Wikström, civ ing SVR, EUR ING, vice president of ”Kontrollansvarigas riksförening”, KARF.[45]

The questions asked during the interviews were the following:

- How does your company assure that uncertainties are minimized?
- If uncertainties occur, how does your company deal with them?
- What do you think is the critical factor that generates uncertainties?
- How do you manage quality control/quality assurance?
- Do you have a quality system that you use?
- Is your business ISO 9001 certified?
- Do you think that the quality control process has become better or worse during the years?
- What is your opinion about FEM programs?
- Are most employees educated first hand in the software program they use on a daily basis or do they learn over time?
- How do you think the FE-programs have changed and evolved over the years, has the focus changed?
- Which software program do you mostly use?
- What limitations do you believe the software programs you use have?
- How do you ensure that your FE models are correct?
- How do you handle documentation?
- Do you feel that there is a gap between seniors and juniors in this industry?
- Do you consider Eurocode being user friendly?
- The way to learn today compared to 20 years ago, has the education changed or the students?

Chapter 7

Compilation of interviews

This chapter is based on a compilation of the interviews that have been carried out.

7.1 Quality control

All attendants agreed that society today has a problem with an ongoing gap, that seems to get clearer by the years, between seniors and juniors. This has in turn caused valuable information losses. The graduated structural engineer relies blindly on technical 3D simulations while an older more experienced structural engineer depend on 2D models and hand calculations. The consequence of the age gap could result in losses of information containing the art of performing a simple hand calculation and that uncertainties in the calculations might occur.

"A structural engineer's status is very low in Sweden compared to, for example Italy" expressed Roberto Crocetti. He believes that this fact has a great role in the gap issue that the structural engineer profession has today and that it is becoming more evident. Roberto Crocetti wants to emphasize that the universities play a big part in the question since the education is more focused on matrix-based methods rather than hand calculations. Consequently this will result in that the understanding of how the structure behaves fades away. Furthermore he resembles hand calculation with learning a language, *"It is easier to learn a language when you are ten years old than when you are 40."* However Roberto Crocetti considers that the education in Sweden generally offers a good standard but that the possibility of deeper specialization is missing. The education has no basis in reality according to Jan Nordh. He considers that the teachers are a fundamental reason for this issue. Since they often have not been working in the construction business at all they will not have anything to compare to. In Mats Persson's opinion the problem why the structural engineering industry nowadays has a gap between seniors and juniors has to do with the structural engineering students. Nowadays students have bad attitudes towards the education. The structural engineering profession something students today start working with in the beginning of their career. They work as a structural engineer for a couple of years and then later takes on another career choice instead of wholeheartedly go in for the profession. Roberto agrees with this statement. Roberto Crocetti wants to underline that a lot of students today are studying because the norm more or less says that one should, while 20 years ago one had thoroughly thought through the study option before commencing. Mats Persson agrees on the claim. Most participants agree on that the FE-programs have become more accessible and user-friendly today compared to ten

years ago. They also agreed that the design process has become much easier to obtain from computer software today. Thomas Kamrad points out that as the FEM software is becoming increasingly user-friendly, the possibility to simulate most advanced calculation performed by the utmost inexperienced structural engineer exists, which in turn can have devastating consequences.

All interviewed parties believe that reasonable assessments are gained over time through experience. The constraints that the FE-software programs have are material properties, says Thomas Kamrad. He wants to highlight that a simulated model is based on linear elastic behavior though the material is not isotropic. Another inadequacy that FE-programs have, according to Thomas Kamrad and David Persson, is the insufficient possibility to analyze with respect to shear when shear forces are of interest to examine.

Although the different companies agreed on that there is an issue with quality control today, they all had different views on what the crucial factors are that contribute to the occurrence of uncertainties. They also differed in the way they worked in order to minimize uncertainties from arising. Jan Nordh is convinced that time pressure is the main reason why uncertainties occur. According to him the stress depends on two factors. First the demands on the staff that has to be available at all times and secondly the financial limits that has to be held within the project. Jan Nordh points out that today's society build far more complex buildings to be performed at lower projection costs. Roberto Crocetti, Mats Persson, Kristian Widén, Mikael Roesengren and Martin Larsson agree with Jan Nordh in his statement. Roberto Crocetti also notes that structural engineers must adapt the right model to the proper construction purposes. *"It is not possible to apply the same method of detailing for small structures as for large ones, this can lead to serious consequences,"* he says. Kristian Widén also believes that uncertainties arise when too much thought is put into optimization of calculations. He develops his statement by saying that companies need to find a balance between good enough and excellent, *"adapt the optimization of the quality control process in relation to the cost and to the purpose of the project."* Mats Persson also wants to emphasize that the main reason to why uncertainties arise is because structural engineers do not always understand the theory behind the physics. *"The structural engineer controls how the reality will appear"* expressed Mats Persson. He develops his statement by stating that in the real world there is no right or wrong. There are physical laws that theoretically describe how objects will behave, but it is up to the designer to apply the right model for the right intention in order to gain the desired result. Roles within the project are another important factor that Jan Nordh wants to enlighten. *"It is important that everyone in the project knows their role and that the group has just the right size in order to obtain an optimal dynamic group,"* he expressed. Furthermore Jan Nordh continued to explain his thought in the issue, *"Serve as an advisor and influence during the early stages,"*. By this statement Jan Nordh explains the importance of the structural engineer being part of and able to influence early in the project. This could minimize numerous confusions and hopefully lead to a better performed project.

Martin Larsson wants to underline that the code-change that occurred from BKR to Eurocode and inexperience are two other factors that play a crucial role. David Persson agrees that the code-change has resulted in considerable uncertainty since there is no experience in Eurocode on the workspace as the situation is today. *"Eurocode is more adapted for lawyers than for engineers,"* uttered Roberto Crocetti. He believes that Eurocode is not easily adapted and because of that it could lead to uncertainties in the interpretation of it. Roberto Crocetti would like a Swedish building regulation that were more educational and that would not control technology as much as Eurocode does. The easiest design requires several Eurocodes which contributes to confusion, continues Roberto Crocetti. This claim is something Mats Skällenäs have had experience in and he agrees with Roberto Crocetti. Mats Skällenäs wants to emphasize that the human factor plays a significant role to why uncertainties occur since the understanding of technology has become more uncertain today due to the more user-friendly software available. He would also like to point out that stress in itself is not a critical factor to why uncertainties arise. The significant reason to why uncertainties occur is more due to how one handles the stress being perceived. Roberto Crocetti agrees with Mats Skällenäs thoughts about technique. *"There is a general overconfidence in computer software. However in order to obtain as accurate results as possible, the input given in the simulated model by the structural engineer has to be correct, regardless of how excellent the software is"*, expressed Roberto Crocetti.

Thomas Kamrad emphasized that boundary conditions are a factor of importance to master in order to minimize uncertainties in modeling. Martin Larsson agrees and also points out that the connections between the elements are difficult to model. Martin Larsson simulates in Fem-Design to get an overall picture of the structure but then he designs the different elements with other programs. According to Martin Larsson this provides both a holistic and an extra assurance that the calculations are correct. Reasonable assessments in the form of rough estimates as easy static hand calculations and load calculations, is something all participants perform to ensure that uncertainties are minimized and the majority of all participants agree that this is the best way to ensure that the model works as intended in reality. Thomas Kamrad mentioned that he always follows the rule of thumb that his hand calculation should not differ by more than 20 percent from the simulated model to prevent further investigation. Boundary conditions and parameter study is something Thomas Kamrad also always implements.

Another factor that everyone agreed on was the fact that one should not be afraid to ask for help from more experienced colleagues or external experienced personnel when uncertainties arise, since it is always good to have a second opinion. *"See me as a colleague rather than your boss"* expressed Jan Nordh several times. Mats Persson also want to emphasize that each individual must realize ones restraints and admit ones inadequacies. All the participants also turn to the support of the program software if applicable.

Other factors that companies among themselves differed in were the way they handled quality assurance, what the quality assurance systems they used looked like and how they handled the documentation of their calculations. Checklists that have been developed by the company are something Structor, Sweco, Centerlöf and Holmberg and Tyréns use. Centerlöf and Holmberg have general checklists that are adapted after the specific project that will be carried out. While in NCC one make one's own checklists, with help from more experienced colleagues, needed for the aimed project. Sweco, Centerlöf and Holmberg, NCC and Tyréns are also certified in ISO 9001. Mats Skällenäs wants to point out that the certification in itself has not guaranteed that the company's knowledge of quality assurance has increased. It is up to the individual company's employee/worker to take responsibility. Thomas Kamrad mentioned that the certification mostly causes administrative processes rather than improved quality assurance. Jan Nordh states that in Sweco they are working on standardizing all the different documentation routines so that the documentation can be readily transparent and easy to use for anyone within the company. All companies document their calculations and store them for at least ten years. Another thing all companies do is self-monitoring. Kristian Widén emphasizes that companies need to figure out why documentation is necessary for their business and how this ought to be carried out. He believes that only necessary papers should be documented in order for the company to have a use of them. All parties/interviewees also agreed that each individual must take responsibility in order to archive as good results as possible. *"Proper communication is the key for almost every process to be carried out in the best possible way"* stated Kristian Widén.

A comparison of whether the participants felt that the quality assurance process has become better now compared to before the external control was deregulated, indicate that the majority agrees that deterioration has occurred over time. *"We have a coordination problem today,"* Jan Wikström pointed out. Mats Skällenäs and Mats Persson agree and note that as turnkey contracts are becoming more common, more deficiencies occur as a result. Since a turnkey contractor buy different parts of the project from different subcontractors, the outcome will result in that there is no person who understands the overall picture of the entire project and there is no one who knows how the various subcontractors interact. Mats Person would want to have a main structural engineer that would have coordination responsibilities involved in more complex and greater projects in order to ensure that the interaction between the different subcontractors works out in order to execute the project correctly. This desire is also one of Jan Wikström's and Mats Skällenäse's.

Mats Persson also points out the importance of contractors and structural engineers need to show mutual respect for each other in order to maintain a well functioning interaction. Jan Nordh, Jan Wikstrom, Roberto Crocetti and Mikael Rosengren would want the government to impose a similar system as the municipal building inspection or introduce some kind of requirement for external control. Mats Persson think that the quality assurance process has declined over the years. However he wants to underline that the reason for this problem is far more complex and not only due to the deregulation of external controls. He develops his statement by mentioning the importance of correct and accurate conditions that must be given to everybody involved in a project so that the end result can turn out flawless. Mats Skällenäs thinks the municipal building inspection had many flaws, but that the idea was better than the current way of controlling the quality standards. He clarifies the statement by commenting that the main role of a control assurance manager nowadays is to control that a documentation of the execution is available. Whether it is right or wrong is not of importance. Roberto Crocetti shares a valuable knowledge gained from his experiences from Switzerland that could be a good solution to ensure that the quality of a project is holding worthy standards. He develops his idea by saying that in the design mission of greater items, two firms ought to be involved. One firm would have the main task to design and another firm would have an examination responsibility. However both parties would work parallel together through the ongoing period of the project for the same goal, to get the best suitable solution as possible for the specific project. Roberto Crocetti means that such a solution would not only mean that uncertainties would minimize, but could also lead to maximum optimization of the building so that the economic cost could decrease.

Martin Larsson does not agree that any form of municipal building inspection would be the solution to the problem, thinking it would be too time-consuming and entail excessive additional cost. Kristian Widén agrees with Martin Larsson, he would also like to emphasize that a municipal building inspection would become very inconvenient. Smaller municipals would have difficulty keeping up with current rules that applies for the building industry. Furthermore they have limited resources to take responsibility points Kristian

Widén out. Mats Skällenäs agrees with Kristian Widéns statement in the difficulties the municipals would face, however he thinks it is a far better system than the system today. "Prerequisites cause companies to take responsibility and to improve on performing quality assurance, however, it may not always exist conditions to ensure that this is done in practice" would Kristian Widén wants to highlight. He believes that the consulting industry has become tougher over the years as they are increasingly time-pressured nowadays.

Kristian Widén gave a different angle of the quality assurance issue. He emphasizes that the essence of a worthy quality assurance program begins when companies know why they should implement a quality assurance plan. Most companies acquire quality assurance systems today without knowing why, Mats Persson agree with this statement. The consequence is that businesses cannot develop and focus on improvement within this area expresses Kristian Widén. *"The question companies must ask themselves is why should our company implement a quality assurance system? What is the main problem? What do we need to change and why? ". "Companies usually relieves the symptoms of the problem without analyzing what the actual cause of the symptoms is"*, Kristian Widén wants to compare the problem with a metaphor in which a person often take headache tablets to ease the headache instead of investigating the real source that is causing the headache.

Chapter 8

Discussion and conclusions

The reader is introduced to how the quality assurance process should work in reality according to the authors.

By taking note of all the valuable information received from the interviews and by absorbing the theory we have come to realize that a well-structured and accurate quality control system is a process every organization ought to invest in. Since a complete quality control plan, that include the entire process, clearly should minimize the possible uncertainties that can arise in a project.

What are the factors that can have a decisive role to ensure that uncertainties due to negligence are minimized? In our opinion there are a certain number of steps that a business should take into account. The first one is to let everyone in the business get involved and let them know the reason why it is of importance to invest in a well-planned quality control system for the company.

An external control system ought to be developed and established in the construction of greater more complex buildings. The person who shall execute the work must be educated within the subject that shall be inspected. For instance one should know the physical laws when examining the work a structural engineer have performed. Furthermore the external control should have the same standards and roles over the entire country. The controller also ought to examine and control the coordination between all structural engineers from different subcontractors.

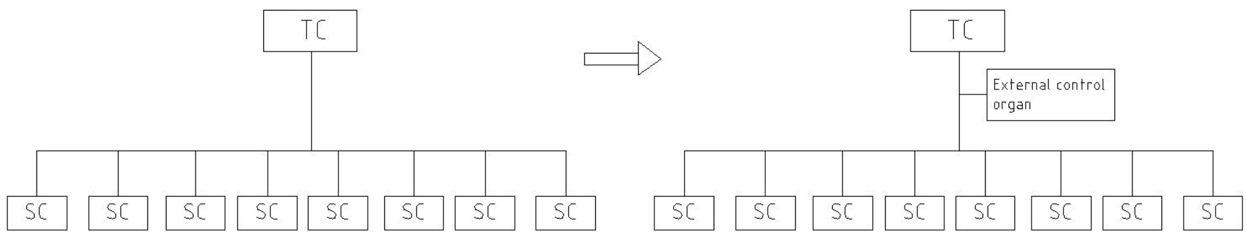


Figure 8.1: Implementation of external control organ

Documentation is another factor that has an important role in the process. In order for a documentation to be in any use for the company it should fulfill a number of requirements. Documentation ought to be well structured in order to maintain an easy to follow calculation part and to assure the possibilities of understanding the documented documentation when desired. Another aspect that the government should take into account is implementation of a general documentation system that ought to be followed by every structural engineering company in the country. This system should be able to be adjusted so that it will meet the purpose of the project. Easy adjustments such as implementing the same heading in all documentation files regardless of the business can make big differences in the process of understanding one another's documentation. This will enable inspection of the documented project as well as lead to a much more organized structure. Furthermore it will hopefully facilitate the possibility to understand similar documented projects that could help prevent mistakes in new projects. Checklists should be developed in all businesses and follow a certain structure. The checklists should only contain the most necessary items so that the list is easy to follow so that they will be performed. However the checklists ought to adapt after the specific project. In this way one can assure that a step in the process has not been neglected.

The universities have a significant role from an educational point of view. The universities ought to have a wider range of specialization courses. Thus more students get the opportunity to master a subject that interests them and will therefore become better on their topic. To know ones theory is probably the key in preventing mistakes from occurring in the first place. Another subject the universities should take into account is the possibility to implement practical topics within the education. Whether one shall work as a structural engineer or within a contractor firm it is always important that one knows how the theory works in practice, in the "real world", or else there is a big chance one loses basis in reality. Some things one has to learn by time through experience, while other fundamental skills has to be understood and known understand and know before stepping into the working life.

Another significant fact to implement in a project in order to succeed is to define the different roles within the group clearly. Everyone must know what is expected of them. It creates safety and commitment. Additionally each person within the group gets to know their responsibilities and limitations. Descriptions of the work that is to be executed ought to be discussed with the person who will execute the work so that everyone in the group is aware of what is expected of them. The right conditions must be met in the right way in order for the outcome to become as planned. Everyone ought to understand the difference between a role and a person. A person can have multiple roles while several people can have the same role. Right flow of information, both within the group and between the providers, creates the right conditions for the specific goal. Each person in the group has an important role, *"A chain is only as strong as its weakest link."* An analogy can be made with a cog wheel (figure 8.2), in order for the machinery to function as intended all cog wheel's need to spin.

Since a majority of the cost as well as the conditions are defined in the early stages it is of great importance for a structural engineer to be a part of this stage and to be able to influence. In that way a lot of misunderstandings between for instance architects and structural engineers can be minimized as well as the final cost could be reduced. Figure 8.3 illustrates the issue.

Another problem that we think has a crucial role in the structural engineer industry nowadays is the fact that students have enormous opportunities. One day one can work as a structural engineer, the other day as a head manager at a global IT company. The consequence of this range can have caused the age gap that we are experiencing in the structural engineering industry today. There are no more restraints in the subject that

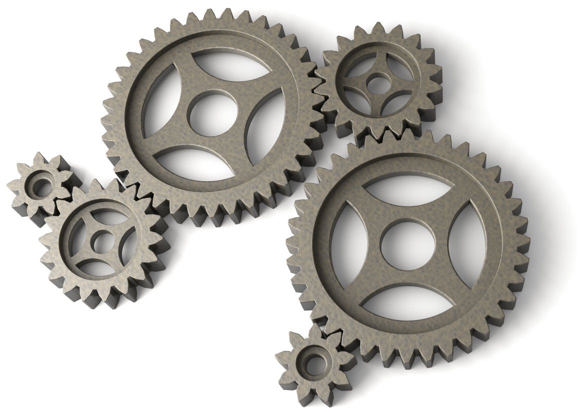


Figure 8.2: A system of cog weal's [11]

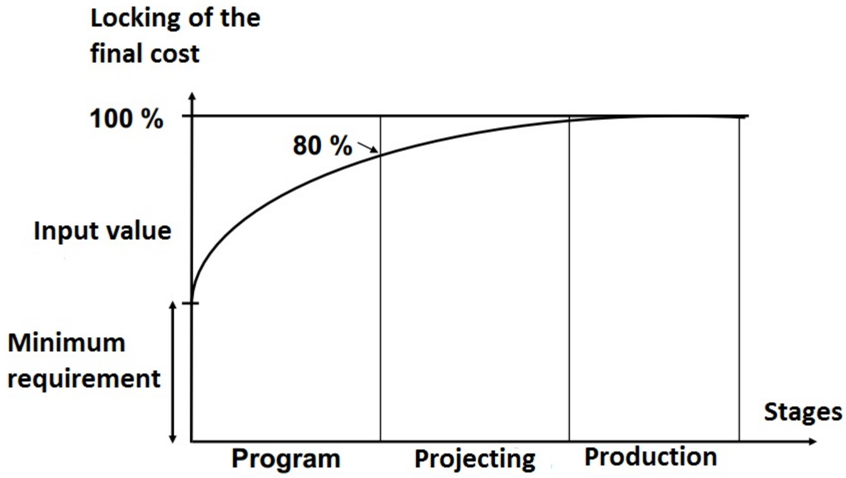


Figure 8.3: Early stages [17]

"I shall work within the educational choice I made," nowadays you can create your own profession through a range of options. For instance through social networks such as LinkedIn or through creating your own education through studying a range of subjects. Therefore one can graduate as a structural engineer, work within this profession in five years and then change to another career choice as a project manager in for instance a contractor company. In that way one can more or less earn more money by taking on more responsibilities in a larger faster growing business. The fact that todays students have more possibilities is both positive and negative but when analyzing why there is an ongoing growing gap between seniors and juniors it is more negative than positive. Another fact that can have an important role in the gap issue is the work environment a consultant has to put up with. It is a very time dependent and strenuous work place and at the same time a structural engineer has to take big responsibility in the work they carry out.

To connect the quality assurance aspect with the FE-analysis one has to consider that a person with slight knowledge within FEM has the possibility to more or less manipulate a model in a FE-program in order to obtain a desired result. The obtained result does not have to be the problem but the result has to fulfill the correct behaviuor in reality. Therefore the design of the building in reality must match with the simulated model in order to achieve the desired behaviour of the building. For that reason it is of great importance to gain experience, knowledge and to carry out accurate documentation.

Chapter 9

Suggestions for future thesis

Several suggestions for possible future thesis are described in this chapter.

The interaction between the foundation and the structure is essential to take into account in simulations. Therefore it is of great importance to further analyze this issue. A future master thesis could handle this subject.

To say that uncertainties arise in the building industry due to poorly made quality control in the planning phase is not enough to determine how well this statement integrates in real life. To be able to assure that uncertainties are minimized a company should at least improve their quality control system through the entire process, from the planning stage to the operation and maintenance phase. A future thesis could deal with and how quality control is handled through the entire project in real life and how the companies deal with uncertainties that arise along the project.

Another object that could be further examined is the possibility to introduce a main structural engineer that would have coordination responsibilities. The main structural engineer should be involved in more complex and greater project in order to ensure that the interaction between the different subcontractors, under a turnkey contractor, functions in order to execute the project correct.

The importance of that entrepreneurs and structural engineers need to have a mutual respect for each other in order to maintain a well functioned interaction is another aspect that can be of interest to investigate further. A future master thesis could be to study how this relationship works in the industry and what kind of shortcomings there is that can contribute to the occurrence of uncertainties.

Documentation is a significant part in the quality assurance process therefore it would be of great interest to examine this issue further. A future master thesis could be to analyze a more systematic and flexible way to document and the possibility to implement a similar documentation system in the entire country in order to achieve well-structured and easy to follow documentations.

Bibliography

[1] M. Bergström. *Feldimensionerade pelare kollapsade*, 2012. http://www.byggvarlden.se/nyheter/dimensioneringsfel-orsakade-husras (130207).

[2] S. Betongforeningen. *Svenska Betongforeningens handbok till Eurokod 2*, volume 1. 2010.

[3] Boverket. *2010:4 - om certifierade kontrollansvariga i nya plan- och bygglagen*, 2010. http://www.boverket.se/Om-Boverket/Nyhetsbrev/Boverket-informerar/Ar-2010/2010—om-certifierade-kontrollansvariga-i-nya-plan–och-bygglagen/ (130207).

[4] R. Crocetti. Professor, department of structural engineering the faculty of engineering,. Lund University. Interview, 2013. (130205).

[5] O. Dahlblom and K.-G. Olsson. *Strukturmekanik - Modellering och analys av ramar och fackverk*, volume 1. 2010.

[6] L. Elfgren, K. Gylltoft, H. Sundquist, and S. Thelandersson. *Byggnader som rasar växande problem i Sverige*, 2012. http://www.dn.se/debatt/byggnader-som-rasar-vaxande-problem-i-sverige.

[7] B. Engstrom. *Design and Analysis of Deep Beams, Plates and Other Discontinuity Regions*, volume 6. 2011.

[8] Fib. *Structural Concrete - Textbook on behaviour, design and performance, vol. 1*, volume I. 2009.

[9] S. Heyden, O. Dahlblom, A. Olsson, and G. Sandberg. *Introduktion till Strukturmekaniken*, volume 4. Studentlitteratur, 2008.

[10] ISO. Kvalitetsansvarig enligt pbl. 2013.

[11] Isoft. 2013. http://www.isoft.se/situation/kugghjul.jpg (130220).

[12] T. Kamrad. Structural engineering in bridges. Centerlöf and Holmberg. Interview, 2013. (130129).

[13] M. Larsson. Structural engineer. NCC. Interview, 2013. (130129).

[14] M. Nielsen and L. Hoang. *Limit Analysis and Concrete Plasticity.* 2010. Edition 3.

[15] L. Nohrstedt. *Ingenjören: Därför rasade bron i Kista*, 2009. http://www.nyteknik.se/nyheter/bygg/byggartiklar/article265636.ece (130205).

[16] J. Nordh. Regional director. Sweco. Interview, 2013. (130125).

[17] S. Olander. Projektkalkylering. 2010.

[18] D. Orr. *Better Business Communication: Tips and Techniques*, 2006. http://tipsbuscom.blogspot.se/2006/08/why-document-processes.html (130221).

[19] N. S. Ottosen and H. Petersson. *Introduction to the finite element method.* Prentice Hall, 1992.

[20] D. Persson. Structural engineer. Centerlöf och Holmberg. Interview, 2013. (130204).

[21] M. Persson. Business director. Tyréns. Interview, 2013. (130207).

[22] M. Rosengren. Structural engineering. Structor AB. Interview, 2013. (130123).

[23] A. Rozinat. *How to Reduce Waste with Process Mining*, 2011. http://fluxicon.com/blog/2011/10/how-to-reduce-waste-with-process-mining/ (130221).

[24] J. Rydeen. *Facility Planning: Quality Control.* 2004. http://asumag.com/mag/facility-planning-quality-control (130205).

[25] Scanscot. Homepage, 2012. http://www.scanscot.com/products/brigadeplus/product-overview/ - (121204).

[26] J. Schlaich, K. Shafer, and M. Jennewein. *Towards a Consistent Design of Structural Concrete.* 1987.

[27] Simulia. *Abaqus 6.10: Analysis User's Manual*, volume IV. 2010.

[28] Simulia. *Abaqus 6.10: Keywords reference Manual*, volume IV. 2010.

[29] M. Skällenäs. Head of department industry. Centerlöf och Holmberg. Interview, 2013. (130204).

[30] SSI. *Eurocode 2: Design of concrete structures-Part 1-1:General rules and rules for buildnings.* 1 edition, 2008.

[31] SSI. *Eurocode 1: Actions on structures-Part 1-1: General actions-Snow loads.* 1 edition, 2009.

[32] SSI. *Eurocode 0: Basic of structural design.* 1 edition, 2010.

[33] SSI. *Eurocode 1: Actions on structures-Part 1-1: General actions-Densities, self-weight, imposed loads for buildings.* 1 edition, 2011.

[34] SSI. *Eurocode 1: Actions on structures-Part 1-1: General actions-Densities, self-weight, imposed loads for buildings.* 1 edition, 2011.

[35] S. S. I. SSI. *Eurocode 1: Actions on structures–Part 1-4: General actions–Wind actions.* 1 edition, 2008.

[36] Strusoft. *FEM-design - Applied theory and design.* 2010.

[37] Strusoft. *FEM-design - User manual.* 2010.

[38] Strusoft. Homepage, 2012. http://www.strusoft.com/index.php/en/products/fem-design - (121204).

[39] M. Studyguide. *Documenting a Process - Importance and Its Benefits*, 2008. http://www.managementstudyguide.com/documentation-of-process.htm (130221).

[40] M. Techniques. *Why is quality control important?*, 2011. http://group3-qualitymanagementtechniques.blogspot.se/2011/02/why-is-quality-control-important.html (130207).

[41] W. Tell. *Bygg handbok för hus-, väg- och vattenbyggnad*, volume 3. 1961.

[42] Tyréns. *Geoteknik för konstruktörer - geoteknikutbildning för bro- och byggnadskonstruktörer*, volume I. 2010.

[43] C. Wang. *Timoshenko beam-bending solutions in terms of Euler-Bernoulli solutions.* Journal of engineering mechanics, 1995.

[44] K. Widén. Tekn dr at the construction management department and president at swedish council quality. SKF. Interview, 2013. (130130).

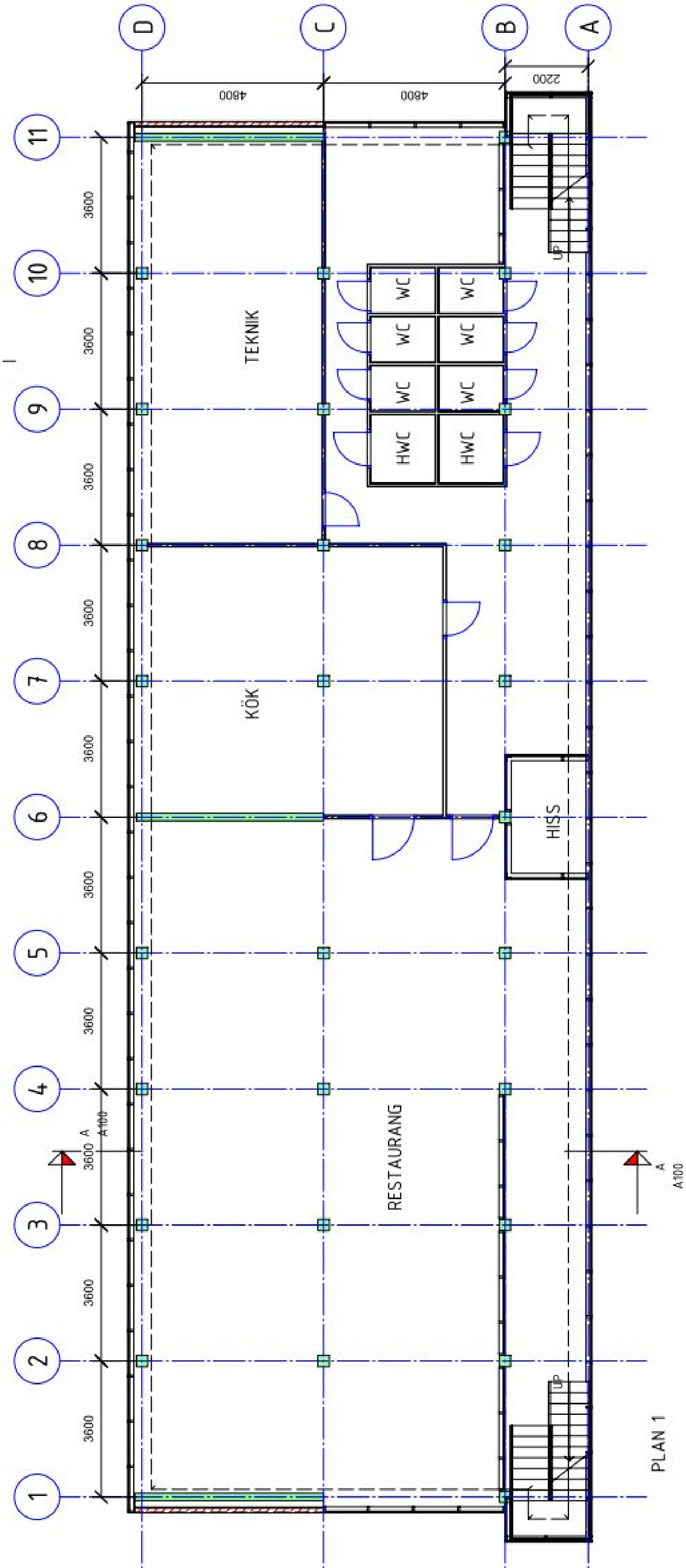
[45] J. Wikström. Civil engineer svr, eur ing, vice president of 'kontrollansvarigas riksförening', karf. Interview, 2013. (130131).

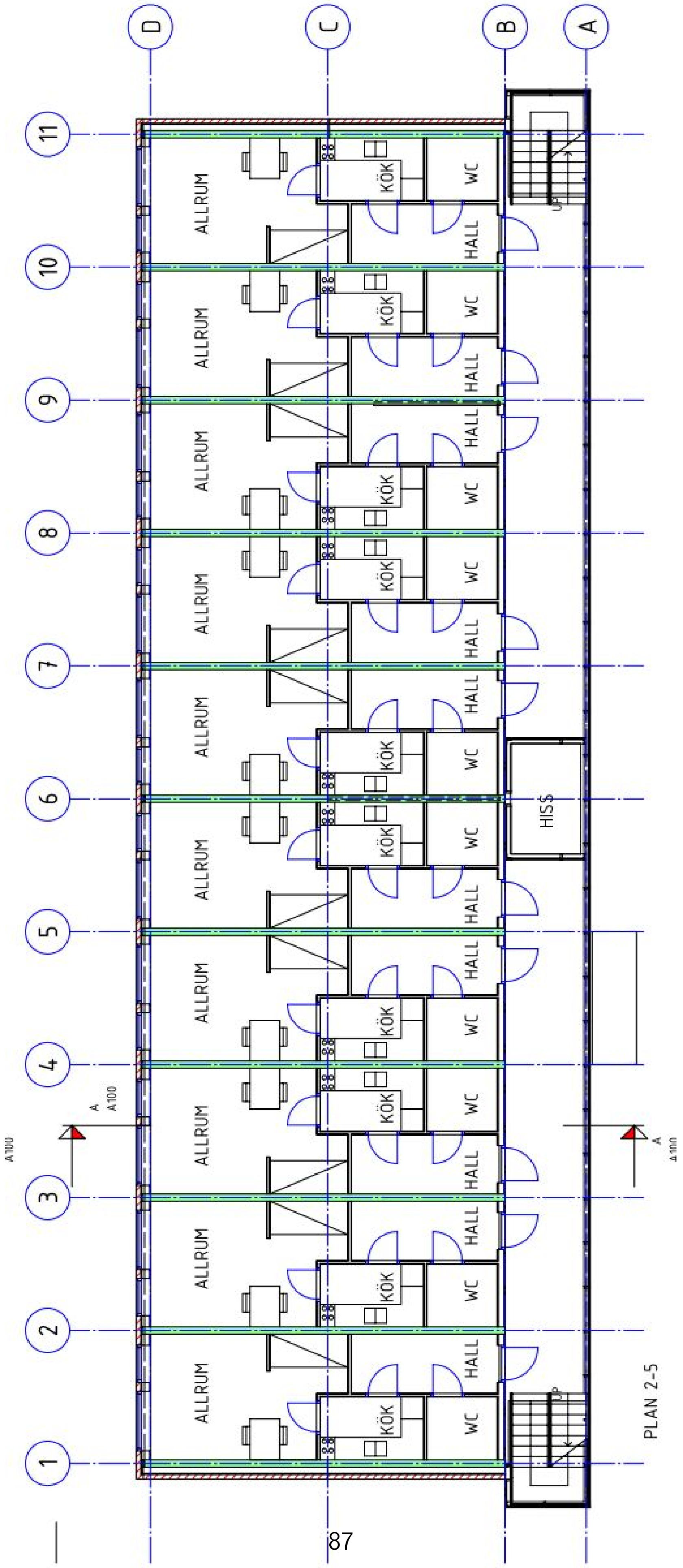
[46] Wisegeek. *What is quality control?*, 2008. http://www.wisegeek.org/what-is-quality-control.htm (130105).

Chapter 10

Appendix

10.1 View over the building


D
C
B
A
4800
4800
2200
11
10
9
8
7
6
5
4
3
2
1
3600
TEKNIK
WC
HWC
KÖK
HISS
RESTAURANG
UP
A
A100
PLAN 1


A
B
C
D
1
2
3
4
5
6
7
8
9
10
11
ALLRUM
KÖK
WC
HALL
HISS
UP
A
A100
PLAN 2-5

10.2 Load Calculations

Design loads

Martin Hedström & Noor Alsuhairi

$kNm := 1kN \cdot 1m$

safety class

$\gamma_d := 1$

Self-weight:

$\gamma_{con} := 25 \frac{kN}{m^3}$

$\gamma_{brick} := 19 \frac{kN}{m^3}$

$\gamma_{plaster} := 15 \frac{kN}{m^3}$

$\gamma_{minerite} := 21 \frac{kN}{m^3}$

SS-EN 1991-1-1

$\gamma_{iso} := 1.4 \frac{kN}{m^3}$

$t_{slab} := (0.220)m$

$t_{wall} := 0.2 \cdot m$

$h_{wall} := 2.53m$

$h_{outerwall} := 11m$

$h_{pillar} := 3.68m$

$G_{slab} := t_{slab} \cdot \gamma_{con} = 5.5 \cdot \frac{kN}{m^2}$

$G_{shellwall} := t_{wall} \cdot \gamma_{con} \cdot h_{wall} = 12.65 \cdot \frac{kN}{m}$

$G_{outerwall} := \begin{pmatrix} 0.12m \cdot \gamma_{brick} + 0.005 \cdot m \cdot \gamma_{minerite} + 0.013 \cdot m \cdot \gamma_{plaster} \cdots \\ + 0.195m \cdot \gamma_{iso} \end{pmatrix} \cdot h_{outerwall} = 31.383 \cdot \frac{kN}{m}$

$G_{outerwallperfloor} := \frac{G_{outerwall}}{4} = 7.846 \frac{kN}{m}$

$G_{conpillar} := 0.3 \cdot 0.3m^2 \cdot \gamma_{con} \cdot h_{pillar} = 8.28 \cdot kN$

$influens_{area} := 3.6m$

$G_{tot4floor} := G_{slab} \cdot 5 \cdot influens_{area} = 99 \cdot \frac{kN}{m}$

$G_{shellwalltotal} := G_{shellwall} \cdot 4 = 50.6 \cdot \frac{kN}{m}$

Snow.

$s_k := 1 \frac{kN}{m^2}$

$\psi_{snow0} := 0.6$

$\mu_1 := 0.8$

Formfactor roof <5 grader

$C_e := 1.0$

$C_t := 1.0$

$Snow := s_k \cdot \mu_1 \cdot C_e \cdot C_t = 0.8 \cdot \frac{kN}{m^2}$

Drifted snow

$\alpha := 16$

$h_{roofbuilding} := 3m$

$l_s := 2 \cdot h_{roofbuilding} = 6\,m$

$b_1 := 5m$

$b_2 := 2.4m$

$l_{ss} := l_s - b_2 = 3.6\,m$

$\mu_s := 0.5 \cdot 0.8 = 0.4$

$\mu_w := \frac{(b_1 + b_2)}{2 \cdot h_{roofbuilding}} = 1.233$

Interpolation

$\mu_3 := 1.298$

$\mu_m := \frac{[(\mu_w + \mu_s) + \mu_3]}{2} = 1.466$

$s_{ficka} := \mu_m \cdot s_k = 1.466 \cdot \frac{kN}{m^2}$

Wind

$v_b := 26 \frac{m}{s}$

$\psi_{wind0} := 0.3$

Terrain. 0

$q_p := 1.27 \frac{kN}{m^2}$ worst case scenario, cpe10 = 1.0, zon D

$w_{wall} := q_p$ Invändig vindlast

Live load

$q_k := 2.0 \frac{kN}{m^2}$ $\psi_{liveload0} := 0.7$

$N_{inst} := 0.5 \frac{kN}{m^2}$ SS-EN 1991-1-1

$q_{liveloadtot} := q_k + N_{inst} = 2.5 \cdot \frac{kN}{m^2}$

Design of loads ekq 6.10a

$G_{walldim1} := \gamma_d \cdot 1.35 \cdot G_{outerwall} \cdot influens_{area} = 152.521 \cdot kN$

$Q_{shellwall1} := \gamma_d \cdot 1.35 \cdot G_{shellwalltotal}$ $Q_{shellwall1} = 68.31 \cdot \frac{kN}{m}$

$$Q1 := \gamma_d \cdot 1.35 \cdot G_{tot4floor} + \gamma_d \cdot 1.5 \cdot \psi_{snow0} \cdot snö_{ficka} \cdot influens_{area} \dots + \gamma_d \cdot 1.5 \cdot \psi_{liveload0} \cdot q_{liveloadtot} \cdot influens_{area} = 147.849 \cdot \frac{kN}{m}$$

$Q_{wind} := 1.5 \cdot \psi_{wind0} \cdot w_{wall} \cdot influens_{area} = 2.057 \cdot \frac{kN}{m}$

$M_{wind} := Q_{wind} \cdot \frac{(18m)^2}{8} = 83.325 \cdot kNm$

$forcecouple := \frac{M_{wind}}{9.6m} = 8.68 \cdot kN$

$Q_{conpillar1} := 1.35 \cdot 0.3 \cdot 0.3m^2 \cdot \gamma_{con} \cdot h_{pillar} = 11.178 \cdot kN$

Design of loads ekq 6.10a

$$Q_{conpillar2} := 1.2 \cdot 0.3 \cdot 0.3 m^2 \cdot \gamma_{con} \cdot h_{pillar} = 9.936 \cdot kN$$

$$G_{walldim2} := \gamma_d \cdot 1.2 \cdot G_{outerwall} \cdot influens_{area} = 135.575 \, m \cdot \frac{kN}{m}$$

$$Q_{shellwall2} := \gamma_d \cdot 1.2 \cdot G_{shellwalltotal} \qquad Q_{shellwall2} = 60.72 \cdot \frac{kN}{m}$$

Snow main load

$$Q2 := \gamma_d \cdot 1.2 \cdot G_{tot4floor} + \gamma_d \cdot 1.5 \cdot snö_{ficka} \cdot influens_{area} \dots + \gamma_d \cdot 1.5 \cdot \psi_{liveload0} \cdot q_{liveloadtot} \cdot influens_{area} = 136.165 \cdot \frac{kN}{m}$$

$$Q_{wind2} := 1.5 \cdot \psi_{wind0} \cdot w_{wall} \cdot influens_{area} = 2.057 \cdot \frac{kN}{m}$$

$$M_{wind2} := Q_{wind2} \cdot \frac{(18m)^2}{8} = 83.325 \cdot kNm$$

$$forcecouple2 := \frac{M_{wind2}}{9.6m} = 8.68 \cdot kN$$

Wind main load

$$Q3 := \gamma_d \cdot 1.2 \cdot G_{tot4floor} + \gamma_d \cdot 1.5 \cdot snö_{ficka} \cdot influens_{area} \cdot \psi_{snow0} \dots + \gamma_d \cdot 1.5 \cdot \psi_{liveload0} \cdot q_{liveloadtot} \cdot influens_{area} = 132.999 \cdot \frac{kN}{m}$$

$$Q_{wind3} := 1.5 \cdot w_{wall} \cdot influens_{area} = 6.858 \cdot \frac{kN}{m}$$

$$M_{wind3} := Q_{wind3} \cdot \frac{(18m)^2}{8} = 277.749 \cdot kNm$$

$$forcecouple3 := \frac{M_{wind3}}{9.6m} = 28.932 \cdot kN$$

Live Load main load

$$Q4 := \gamma_d \cdot 1.2 \cdot G_{tot4floor} + \gamma_d \cdot 1.5 \cdot snö_{ficka} \cdot influens_{area} \cdot \psi_{snow0} \dots + \gamma_d \cdot 1.5 \cdot q_{liveloadtot} \cdot influens_{area} = 137.049 \cdot \frac{kN}{m}$$

$$Q_{wind4} := 1.5 \cdot \psi_{wind0} \cdot w_{wall} \cdot influens_{area} = 2.057 \cdot \frac{kN}{m}$$

$$M_{wind4} := Q_{wind4} \cdot \frac{(18m)^2}{8} = 83.325 \cdot kNm$$

$$forcecouple4 := \frac{M_{wind4}}{9.6m} = 8.68 \cdot kN$$

Load - Design values

$Design_{shellwall} := Q_{shellwall1} = 68.31 \cdot \frac{kN}{m}$

$Design_Q := Q1 = 147.849 \cdot \frac{kN}{m}$

$Design_P := G_{walldim1} = 152.521\, m \cdot \frac{kN}{m}$

$Service := \gamma_d \cdot 1 \cdot G_{tot4floor} + \gamma_d \cdot 1.5 \cdot s_{ficka} \cdot \psi_{snow0} \cdot influens_{area} \ldots = 113.199 \cdot \frac{kN}{m}$
$+ \gamma_d \cdot 1.5 \cdot \psi_{liveload0} \cdot q_{liveloadtot} \cdot influens_{area}$

$Service_{shellwall} := G_{shellwalltotal} = 50.6 \cdot \frac{kN}{m}$

$Service_p := G_{outerwall} \cdot influens_{area} = 112.979 \cdot kN$

$ServiceQ_{conpillar} := 1 \cdot 0.3 \cdot 0.3m^2 \cdot \gamma_{con} \cdot h_{pillar} = 8.28 \cdot kN$

Brigade and FEM-design

$Q1aNL1 := \gamma_d \cdot 1.5 \cdot \psi_{liveload0} \cdot q_{liveloadtot} = 2.625 \cdot \frac{kN}{m^2}$

$Q2aS1 := \gamma_d \cdot 1.5 \cdot \psi_{snow0} \cdot s_{ficka} = 1.319 \cdot \frac{kN}{m^2}$

$Q3aV1 := 1.5 \cdot \psi_{wind0} \cdot w_{wall} = 0.571 \frac{1}{m} \cdot \frac{kN}{m}$

$Q_{conpillar} := 1.35 \cdot 0.3 \cdot 0.3m^2 \cdot \gamma_{con} \cdot h_{pillar} = 11.178 \cdot kN$

$Wind_{middle} := Q3aV \cdot \frac{h_{pillar}}{2} = 1.052 \cdot \frac{kN}{m}$

$Wind_{bot} := Q3aV \cdot \left(\frac{h_{pillar}}{2} + \frac{h_{wall}}{2} \right) = 1.775 \cdot \frac{kN}{m}$

$wind_{top} := Q3aV \cdot \frac{h_{wall}}{2} = 0.723 \cdot \frac{kN}{m}$

10.3 Appendix Hand - and Strut and Tie Calculations

10.3.1 Reaction forces

According to figure 3.1 the reaction force is calculated with the support angle method and equilibrium equations. q_1 and q_2 are equal to q_{d1} and q_{d2} in section 3.1.

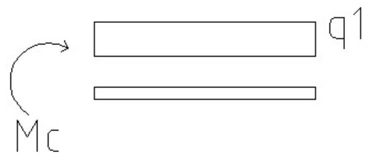


Figure 10.1: Cut in the third support

$$\circlearrowleft -M_c - q_1 \cdot \frac{2.2^2}{2} = 0 \Longrightarrow M_c = -358kN$$

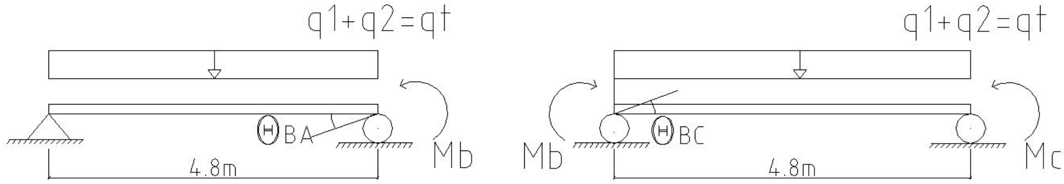


Figure 10.2: Beam cut in two sections with cantilever part replaced with an end moment

$$q_t = q_1 + q_2 = 216.1kN/m$$

$$\theta_{BA} = \frac{q_t L^3}{24EI} + \frac{2M_B L}{6EI}$$

$$\theta_{BC} = -\frac{q_t L^3}{24EI} + \frac{L(-2M_B - M_c)}{6EI}$$

$$\theta_{BC} = \theta_{BA}$$

$$2 \cdot \frac{q_t \cdot 4.8^3}{24} + 4 \cdot \frac{M_b \cdot 4.8}{6} + \frac{4.8 \cdot M_c}{6} = 0$$

$$2 \cdot \frac{216.1kN/m \cdot 4.8^3}{24} + 4 \cdot \frac{M_b \cdot 4.8}{6} + \frac{4.8 \cdot -358kN}{6} = 0 \Longrightarrow M_b = -533kNm$$

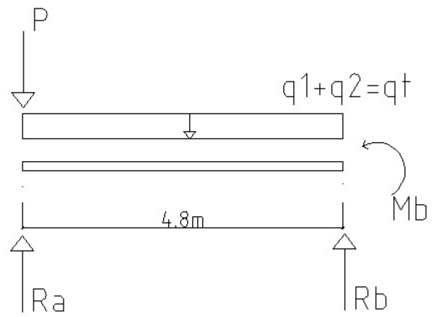


Figure 10.3: Beam cut in the second support, analyzing the left part of the structure

$$\circlearrowleft b : -R_a \cdot 4.8 + M_b + q_t \cdot \frac{4.8^2}{2} + 4.8 \cdot P = \ 0 \Longrightarrow R_a = 560kN$$

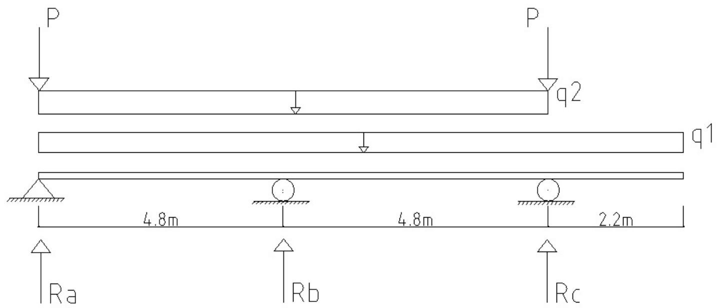


Figure 10.4: Section of the entire system

$$\circlearrowleft c : -q_2 \cdot 9.6 \cdot 4.8 + 9.6 \cdot P + q_1 \cdot 11.8 * 3.7 - R_a * 9.6 - R_b * 4.8 = 0 \Longrightarrow R_b = 1185kN$$

$$\uparrow) - q_2 \cdot q_1 \cdot 11.8 - 2 \cdot P + R_a + R_b + R_c \Longrightarrow R_c = 960kN$$

The self-weight of the columns are according to Appendix 10.2 8.3 kN which add 8.3 kN on each reaction force. The wind load on the long side gives rise to a moment in the structure. This moment will result in a larger reaction force in R_a and a smaller reaction force in R_c.

$$M_{wind} = Q_{wind} \cdot \frac{18^2}{8} = 83.3kNm$$

$$F = \frac{83.3kNm}{9.6m} \ = \ 8.7kN$$

The resulting reaction force is when the self-weight of the columns and wind load is taken into account.

$$R_a = \ 560 \ + \ 8.3 \ - \ 8.7 \ = 560kN$$
$$R_b = \ 1185 \ + \ 8.3 = \ 1193kN$$
$$R_c = \ 960 \ + \ 8.3 \ + \ 8.7 \ = 977kN$$

10.3.2 Reinforcement calculation according to Strut and Tie

Figure 3.2 illustrates the selected struts and ties for the simplified structure. α_1 is chosen to 70° according to recommendation in section 2.2.9 . Since length between the where the force acts and the egde is known, the height h and the angles in node 2,3 and 4 can be calculated.

$$h = \tan 70° \cdot 0.95 = 2.61m$$

$$\alpha_2 = \arctan \tfrac{2.61}{1.45} = 61°$$

$$\alpha_3 = \arctan \tfrac{2.61}{1.3} = 63.5°$$

$$\alpha_4 = \arctan \tfrac{2.61}{1.1} = 67°$$

As noted in section 2.2.9 the node and the compression in the nodes should be controlled. The concrete quality and reinforcement size for this structure is selected to C45 and ∅12. With these material properties the stress limit in the node and compression in the node is calculated to:

$$v = 1 - \frac{45}{250} = 0.82$$

$$\sigma_{rd,max,node} = 0.85 \cdot 0.82 \cdot 30 \cdot 10^6 = 20.9MPa$$

$$\sigma_{rd,max,brace} = 0.6 \cdot 0.82 \cdot 30 \cdot 10^6 = 14.8MPa$$

The dimension of reinforcement determines the concrete cover according to section 4.4.1 in [30].

$$c_{nom} = 12 + 10 + 6 = 28mm$$

The distance u is determined by $u = 2 \cdot s_o$ [7].

$$u = 2 \cdot 28 = 56mm$$

Figure 2.10 With angles in every important node one can begin to calculate forces, reinforcement and control stresses. The width that the compression on the node is acting on is determined by equation 2.8. Beginning with node 1, see figure 10.5:

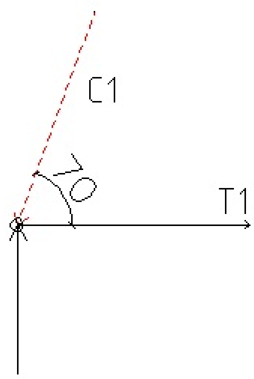


Figure 10.5: Node 1

$$C_1 = \frac{R_a}{\sin 70} = \frac{577kN}{\sin 70} = 614kN$$

$$T_1 = \frac{R_a}{\tan 70} = \frac{577kN}{\tan 70} = 210kN$$

$$a_2 = 0.3 \cdot \sin 70 + 0.056 \cdot \cos 70 = 0.301m$$

$$\sigma_{CCT1} = \frac{577kN}{0.3^2} = 6.4MPa$$

$$\sigma_{Brace1} = \frac{614kN}{0.3 \cdot 0.301} = 6.8MPa$$

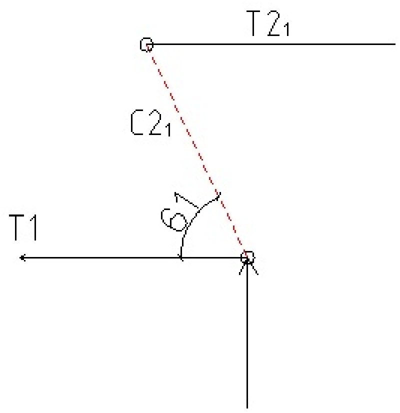


Figure 10.6: Node 2

Because of the difference in reaction force R_{b1} and R_{b2} the support in the middle is split by two.

$$C_{21} = \frac{R_{b1}}{\sin 61} = \frac{596kN}{\sin 61} = 681kN$$

$$T_{21} = \frac{R_{b1}}{\tan 61} = \frac{596kN}{\tan 61} = 330kN$$

$$l_{b1} = 0.3 \cdot \frac{596}{1193} = 0.15m$$

$$l_{b2} = 0.3 \cdot \frac{596}{1193} = 0.15m$$

$$a_2 = 0.15 \cdot \sin 61 + 0.056 \cdot \cos 61 = 0.158m$$

$$\sigma_{CCT21} = \frac{596kN}{0.3 \cdot 0.15} = 13.2MPa$$

$$\sigma_{Brace21} = \frac{681kN}{0.3 \cdot 0.158} = 14.4MPa$$

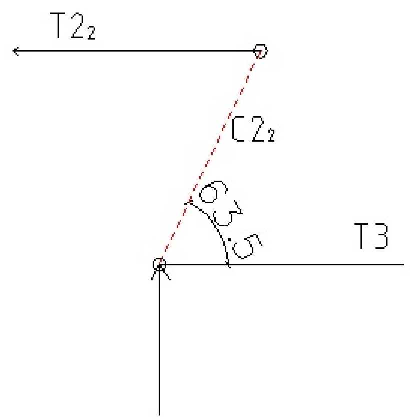


Figure 10.7: Node 3

$$C_{22} = \frac{R_{b2}}{\sin 63.5} = \frac{596kN}{\sin 63.5} = 666kN$$

$$T_{22} = \frac{R_{b2}}{\tan 63.5} = \frac{596kN}{\tan 63.5} = 297kN$$

$$a_2 = 0.15 \cdot \sin 63.5 + 0.056 \cdot \cos 63.5 = 0.159m$$

$$\sigma_{CCT22} = \frac{596kN}{0.3 \cdot 0.15} = 13.2MPa$$

$$\sigma_{Brace22} = \frac{666kN}{0.3 \cdot 0.159} = 14MPa$$

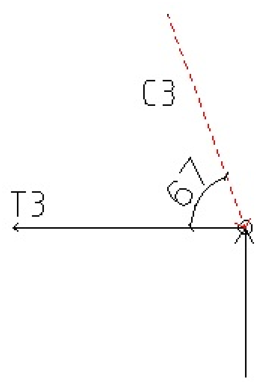


Figure 10.8: Node 4

$$C_3 = \frac{R_c}{\sin 67} = \frac{960kN}{\sin 67} = 1042kN$$

$$T_3 = \frac{R_c}{\tan 67} = \frac{960kN}{\tan 67} = 407kN$$

$$a_2 = 0.3 \cdot \sin 67 + 0.056 \cdot \cos 67 = 0.298m$$

$$\sigma_{CCT3} = \frac{960kN}{0.3^2} = 10.7MPa$$

$$\sigma_{Brace3} = \frac{1042kN}{0.3 \cdot 0.298} = 11.7MPa$$

Concrete quality C45 will withstand the compression stresses. When all quality control is done the amount of reinforcement is calculated according to equation 2.7 and that the reinforcements design strength is 435 MPa.

-T_1

$$\frac{210kN}{435MPa} \cdot 10^6 = 483mm^2$$

$$\frac{483mm^2}{6^2 \cdot \pi} = 4.3 \longrightarrow 5\varnothing 12$$

-T_{21}

$$\frac{330kN}{435MPa} \cdot 10^6 = 758mm^2$$

$$\frac{758mm^2}{6^2 \cdot \pi} = 6.68 \longrightarrow 7\varnothing 12$$

-T_3

$$\frac{407kN}{435MPa} \cdot 10^6 = 936mm^2$$

$$\frac{936mm^2}{6^2 \cdot \pi} = 8.27 \longrightarrow 9\varnothing 12$$

10.4 Design of concrete column

Concrete C45
Reinforcement ϕ12 B500B
SK 3 $\gamma_d = 1$
<u>Strenght</u>

C45 ⇒ $f_{ck} = 45$ MPa, $E_{cm} = 36$ GPa
$\varepsilon_{cu} = \varepsilon_{cu3} = 3.5$ ‰
Ultimate limit state ⇒ $\gamma_C = 1.5$

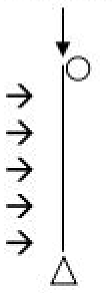
$$f_{cd} = \frac{f_{ck}}{\gamma_C} = \frac{45}{1.5} = 30\ MPa$$

B500B ⇒ $f_{yk} = 500$ MPa, $E_s = 200$ GPa
Ultimate limit state ⇒ $\gamma_S = 1.15$

$$f_{yd} = \frac{f_{yk}}{\gamma_S} = \frac{500}{1.15} = 435\ MPa$$

The pillars buckling length = 3.68 m.

1350 kN



0.571kN/m^2

N_d=1350 kN

$$M_d = \frac{ql^2}{8} = \frac{2.06 \cdot 3.68^2}{8} = 3.48\ kNm$$

Imperfections

$$\theta_i = \theta_0 \alpha_h \alpha_m$$

$$\theta_0 = 0.005$$

$$\alpha_h = 2/\sqrt{l} = 2/\sqrt{3.68} = 1.04$$

$$\alpha_m = \sqrt{0.5\left(1 + 1/27\right)} = 0.72$$

$$\theta_i = 0.005 \cdot 1.04 \cdot 0.72 = 0.00374$$

$$M_d = N_d \cdot (e + a) = 1350 \cdot (0.00387 \cdot 3.68 / 2) = 9.32 \ kNm$$

Dimensions for M_d = 9.32+3.48=12.80 kNm

Reinforcement dimensions ϕ12

d = 300 – (12+10) -12/2 = 272 mm

$$i = \frac{300}{\sqrt{12}} = 86.6 \, mm$$

$$\lambda = \frac{L_{cr}}{i} = \frac{3.68}{0.0866} = 42.5$$

2 order effect?

$$\lambda_{\lim} = \frac{20 \cdot A \cdot B \cdot C}{\sqrt{n}}$$

$$\Phi_{ef} = \varphi(\infty, t_o) \cdot \frac{M_{oEqp}}{M_{oEd}}$$

t_0=28 days
h_0=150 mm
N-curve

→ $\varphi(\infty, t_o) = 2.3$
$\Phi_{ef} = 1.675$

A = 1/(1 + 0.2·1.675) = 0.749

B = 1.1 (ω unknown)

$r_m \approx 1$ → C = 0.7

n = $1350 \cdot 10^{-3}/(0.3 \cdot 0.3 \cdot 30) = 0.5$

$$\lambda_{\lim} = \frac{20 \cdot 0.75 \cdot 1.1 \cdot 0.7}{\sqrt{0.5}} = 16.33 < 42.5 \Rightarrow$$ 2:a order must be considered

Assume $M_{Sd} = 1.5 \cdot M_{0Sd} = 1.5 \cdot 12.8 = 19.2$ kNm

Interaction diagram

$$\left.\begin{aligned}\frac{N_d}{bdf_{cd}} &= \frac{1350\cdot 10^3}{0.4\cdot 0.272\cdot 30\cdot 10^6} = 0.55\\ \frac{M_d}{bd^2 f_{cd}} &= \frac{19.2\cdot 10^3}{0.4\cdot 0.272^2\cdot 30\cdot 10^6} = 0.0288\end{aligned}\right\} \Rightarrow \omega = 0.03$$

$$A_s = A_s' = 0.03\cdot 300\cdot 272\cdot\frac{30}{435} = 168.8\,mm^2 \rightarrow 2\emptyset 12$$

Control

$$\rho = \frac{4\cdot\frac{12^2\pi}{4}}{300\cdot 300} = 0.00503 > 0.002 \rightarrow \begin{cases} K_s = 1\\ K_c = k_1 k_2/(1+\varphi_{ef})\end{cases}$$

$$k_1 = \sqrt{\frac{f_{ck}}{20}} = 1.5$$

$$k_2 = n\frac{\lambda}{170}0.125$$

$$\rightarrow K_c = 0.07$$

$$I_c = \frac{0.3\cdot 0.3^3}{12} = 6.75\cdot 10^{-4}\,m^4$$

$$I_s = 4\left(\frac{\pi\cdot 0.012^4}{64} + \frac{0.012^2\pi}{4}\cdot\left(\frac{0.3}{2} - 0.028\right)^2\right) = 6.73\cdot 10^{-6} m^4$$

$$EI = K_c E_{cd} I_c + K_s E_s I_s = 0.07\cdot\frac{36\cdot 10^9}{1.2}\cdot 6.75\cdot 10^{-4} + 1\cdot 200\cdot 10^9\cdot 6.73\cdot 10^{-6}$$
$$= 2.76\,MNm^2$$

$$N_B = \frac{\pi^2\cdot EI}{L_c^2} = \frac{\pi^2\cdot 2.76\cdot 10^6}{3.68^2} = 2014\,kN$$

$$\beta = \frac{\pi^2}{9.6} = 1.028$$

$$M_{Sd} = M_{0d}\left[1 + \frac{\beta}{\left({}^{N_B}/_{N_{Ed}}\right) - 1}\right] = 12.8\left[1 + \frac{1.028}{\left({}^{2014}/_{1350}\right) - 1}\right] = 39.5\,kNm$$

$$\left.\begin{aligned}\frac{N_{d.}}{bdf_{cd}} &= \frac{1350\cdot 10^3}{0.3\cdot 0.272\cdot 30\cdot 10^6} = 0.55\\ \frac{M_d}{bd^2 f_{cd}} &= \frac{39.5\cdot 10^3}{0.3\cdot 0.272^2\cdot 30\cdot 10^6} = 0.059\end{aligned}\right\} \Rightarrow\ \omega = 0.075$$

$A_s = A_s' = \ 0.075\cdot 300\cdot 272\cdot \frac{30}{435} = 422\ mm^2$ ger 4ϕ25 per sida

Control

$$\rho = \ \frac{8\cdot\frac{12^2\pi}{4}}{300\cdot 300} = 0.01053 > 0.01 \rightarrow \ \begin{cases} K_s = 0 \\ K_c = \dfrac{0{,}3}{(1+0{,}5\varphi_{ef})}\end{cases}$$

$$K_c = \ \frac{0{,}3}{(1+0{,}5\cdot 1.675)} = 0.163$$

$$I_c = \frac{0.3\cdot 0.3^3}{12} = 6.75\cdot 10^{-4}\ m^4$$

$$I_s = 8\left(\frac{\pi\cdot 0.012^4}{64} + \frac{0.012^2\pi}{4}\cdot\left(\frac{0.3}{2} - \ 0.028\right)^2\right) = 1.35\cdot 10^{-5} m^4$$

$$\begin{aligned}EI = \ K_c E_{cd} I_c + K_s E_s I_s &= 0.2\cdot\frac{36\cdot 10^9}{1.2}\cdot 6.75\cdot 10^{-4} + \ 0{,}0\cdot 200\cdot 10^9\cdot 1.35\cdot 10^{-5}\\ &= 3.3\ MNm^2\end{aligned}$$

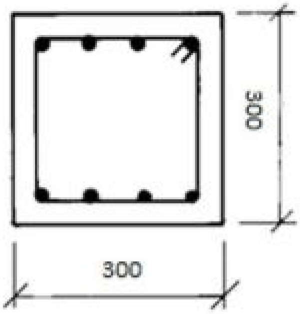
$$N_B = \frac{\pi^2\cdot EI}{L_c^2} = \ \frac{\pi^2\cdot 3.3\cdot 10^6}{3.68^2} = 2405.6\ kN$$

$$\beta = \ \frac{\pi^2}{9.6} = 1.028$$

$$\begin{aligned}M_{Sd} = \ M_{0d}\left[1 + \frac{\beta}{\left({N_B}/{N_{Ed}}\right) - 1}\right] &= 12.8\left[1 + \frac{1.028}{\left({2405.6}/{1350}\right) - 1}\right]\\ &= 29.63\ kNm\end{aligned}$$

$$\left.\begin{aligned} \frac{N_{d,}}{bdf_{cd}} &= \frac{1350 \cdot 10^3}{0.3 \cdot 0.272 \cdot 30 \cdot 10^6} = 0.55 \\ \frac{M_d}{bd^2 f_{cd}} &= \frac{29.63 \cdot 10^3}{0.3 \cdot 0.272^2 \cdot 30 \cdot 10^6} = 0.044 \end{aligned}\right\} \Rightarrow \omega = 0.065$$

$A_s = A_s' = 0.065 \cdot 300 \cdot 272 \cdot \frac{30}{435} = 365.8\, mm^2$ ger 4ϕ25 per sida



A total of 8 rebar's are required, 4 on each side. Stirrups ought to be constructed.

Control of compressive strength

$$\sigma = \frac{N}{A} + \frac{M}{I} y = 15 + 8.77 = 28.78\, MPa < 30\, MPA\ \ OK!$$

Though

15> 8.77 → no tensile occurs

10.5 Calculation of the relationship C

$$\mu = \frac{48EI}{L^3 \cdot S} \ N/m \tag{10.1}$$

Where $S = 175MN/m$ according to $K_{system} = 175MN/m$ in section 3.4. $L = 4.8m$, $E_{steel} = 210\ GPa$ and $I_{HEA1000} = 5538 \cdot 10^{-6} m^4$

$$\mu = \frac{48 \cdot 210 \cdot 10^9 \cdot 5538 \cdot 10^{-6}}{4.8^3 \cdot 175 \cdot 10^6} = 2.9 \tag{10.2}$$